University of Wollongong

Research Online

Faculty of Engineering and Information
Sciences - Papers: Part A

Faculty of Engineering and Information
Sciences

1-1-2014

Shear strength of rock joints influenced by compacted infill

Buddhima Indraratna

University of Wollongong, indra@uow.edu.au

Wuditha N. Premadasa

University of Wollongong, wnp517@uowmail.edu.au

Edwin T. Brown

Golder Associates Pty Ltd

Antonio Gens

Universitat Politecnica de Catalunya, SPAIN

Ana Heitor

University of Wollongong, aheitor@uow.edu.au

Follow this and additional works at: <https://ro.uow.edu.au/eispapers>

Part of the [Engineering Commons](#), and the [Science and Technology Studies Commons](#)

Research Online is the open access institutional repository for the University of Wollongong. For further information contact the UOW Library: research-pubs@uow.edu.au

Shear strength of rock joints influenced by compacted infill

Abstract

Discontinuities such as fault planes, joints and bedding planes in a rock mass may be filled with different types of fine-grained material that are either transported or accumulated as gouge due to weathering or joint shearing. Previous laboratory studies have mainly examined the role of saturated infill that exhibits the minimum shear strength. However, in practice, the infill materials are often partially saturated generating matric suction within the joint that can contribute to increased shear strength. To the authors' knowledge this is the first study to examine the influence of compacted (unsaturated) infill on the joint shear strength. A series of laboratory triaxial tests on idealised model joints and imprinted natural joint profiles was carried out, with constant water contents of the infill being maintained. From the laboratory results, it is observed that the peak shear strength of infilled joints increased with the decrease of degree of saturation from 85% to 35% for both idealised joints and replicated natural joints. Based on the laboratory observations an empirical model for describing the infilled joint shear strength was developed.

Keywords

infill, joints, compacted, influenced, rock, strength, shear

Disciplines

Engineering | Science and Technology Studies

Publication Details

Indraratna, B., Premadasa, W., Brown, E. T., Gens, A. & Heitor, A. (2014). Shear strength of rock joints influenced by compacted infill. *International Journal of Rock Mechanics and Mining Sciences*, 70 296-307.

SHEAR STRENGTH OF ROCK JOINTS INFLUENCED BY COMPACTED INFILL

Buddhima Indraratna†

Professor of Civil Engineering, Director, Centre for Geomechanics and Railway Engineering,
Faculty of Engineering, University of Wollongong, Wollongong, NSW 2522, Australia

Wuditha Premadasa

PhD Candidate, Centre for Geomechanics and Railway Engineering, Faculty of Engineering,
University of Wollongong, Wollongong, NSW 2522, Australia

Edwin T. Brown

Emeritus Professor, University of Queensland and Senior Consultant, Golder Associates Pty
Ltd, PO Box 1734, Milton, Q 4064, Australia

Antonio Gens

Professor of Geotechnical Engineering Department of Geotechnical Engineering and
Geosciences, Universitat Politècnica de Catalunya, Jordi Girona 1-3, Edifici D-2, 08034
Barcelona, Spain

Ana Heitor

Lecturer, Centre for Geomechanics and Railway Engineering, Faculty of Engineering,
University of Wollongong, Wollongong, NSW 2522, Australia

†Author for correspondence:

Prof. Buddhima Indraratna

Professor of Civil Engineering and Research director

Centre for Geomechanics and Railway Engineering, University of Wollongong
Wollongong, NSW 2522, AUSTRALIA

Ph: +61 2 4221 3046

Fax: +61 2 4221 3238

Email: indra@uow.edu.au

To be submitted to: International Journal of Rock Mechanics and Mining Sciences

SHEAR STRENGTH OF ROCK JOINTS INFLUENCED BY COMPACTED INFILL

Buddhima Indraratna¹, Wuditha Premadasa,² Edwin T. Brown³, Antonio Gens⁴, Ana Heitor⁵

ABSTRACT

Discontinuities such as fault planes, joints and bedding planes in a rock mass may be filled with different types of fine-grained material that are either transported or accumulated as gouge due to weathering or joint shearing. Depending on the degree of saturation and the thickness of the infill, the shear strength of the rock mass can be adversely affected. Previous laboratory studies have mainly examined the role of saturated infill that exhibits the minimum shear strength. However, in practice, the infill materials are often partially saturated generating matric suction within the joint that can contribute to increased shear strength. To the authors' knowledge this is the first study to examine the influence of compacted (unsaturated) infill on the joint shear strength. A series of laboratory triaxial tests on idealised model joints and imprinted natural joint profiles was carried out, with constant water contents of the infill being maintained. From the laboratory results, it is observed that the peak shear strength of infilled joints increased with the decrease of degree of saturation from 85% to 35% for both idealised joints and replicated natural joints. Based on the laboratory observations an empirical model for describing the infilled joint shear strength was. As no current standards are available for such joint testing, it is anticipated that this laboratory attempt will provide a useful platform towards establishing a more accurate estimation of infilled joint strength in rock masses.

KEYWORDS: compacted infill; rock joint; rock mechanics; shear strength, Unsaturation

¹ Professor of Civil Engineering and Research director, Centre for Geomechanics and Railway Engineering; University of Wollongong, Wollongong, NSW 2522, Australia (corresponding Author); email: indra@uow.edu.au

² PhD candidate, Centre for Geomechanics and Railway Engineering; University of Wollongong, Wollongong, NSW 2522, Australia; email: wnp517@uowmail.edu.au

³ Emeritus Professor, University of Queensland and Senior Consultant, Golder Associates Pty Ltd, PO Box 1734, Milton, Q 4064, Australia; email: tbrown@golder.com.au

⁴ Professor of Geotechnical Engineering Department of Geotechnical Engineering and Geosciences, Universitat Politècnica de Catalunya, Jordi Girona 1-3, Edifici D-2, 08034 Barcelona, Spain; email: antonio.gens@upc.edu

⁵ Lecturer, School of Civil, Mining and Environmental Engineering, Faculty of Engineering, University of Wollongong, Wollongong, NSW 2522, Australia; email: aheitor@uow.edu.au

1. INTRODUCTION

Generally rock masses present in nature are characterised by discontinuities such as joints, fractures and other planes of weakness. Discontinuities that are infilled with fine-grained material which is either transported or appears as a result of weathering or joint shearing, will adversely affect the behaviour of the rock mass. These fine infill materials may drastically reduce the shear strength of the rock joints compared to an unfilled or clean joint, because they may prevent the walls of the rock joint from coming into contact during shear.

The degree of saturation of the infill is a governing parameter of the shear strength of a filled joint, and it can vary noticeably, depending on the groundwater and climate patterns. Barton [1] carried out an extensive study of filled discontinuities in rock in which the in-situ water content of the infill was found to be a principal parameter controlling the shear strength of a filled joint. Furthermore, for adverse climatic conditions, i.e. heavy precipitation and long periods of rainfall, Barton [1] has reported that the joints may act as conduits of water, leaving the fine infill material basically in near saturated conditions. Most rock masses contain complex, interconnected networks of joints filled with gouge material and because of their high transmissivity, joints are often conduits for fluid flow [2]. Laboratory testing of infill materials from a rock mass failure site at Kangaroo Valley, New South Wales, Australia, confirmed that the soil can reach more than 95% of saturation after a period of heavy rainfall. During dry seasons, the infill saturation will gradually decrease, contributing to an increase in the overall shear strength of the jointed rock mass. While studies have been carried out to investigate the behaviour of infilled rock joints (e.g., Ladanyi & Archambault [3], Lama [4], Pereira[5], de Toledo & de Freitas [6], Indraratna *et al.* [7], [8], [9]), the majority have considered either a fully saturated infill condition or a specified saturation level.. Recently, Alonso *et al.* [10], and Zandarin *et al.* [11] conducted a study on partially saturated bare rock joints, but the role of infill within the joint was not considered. From a practical perspective, most infill materials will likely be compressed over time and remain typically in an unsaturated state, unless the joints are submerged by groundwater which may happen in the event of groundwater inflows occurring through specific discontinuities. In this instance, grouting of the joints may be considered as a method to prevent the infill materials from reaching full saturation, thus reducing the probability of catastrophic rock slides occurring.

The shear strength of a filled joint is often assumed to be that of the infill material

alone. While this assumption may be acceptable if the infill thickness is higher than a certain critical value, for smaller values of infill thickness in relation to the joint roughness or asperities it neglects the possibility of rock-to-rock contact taking place. In these conditions, the rock-to-rock contact influence becomes increasingly prominent. In contrast, the effect of infill saturation can be distinctly observed for a thicker infill where the strength is governed by the infill alone. This effect decreases as the infill becomes thinner, because in such conditions the shear strength of the joint is basically governed by the shear behaviour of the surfaces of the rock.

In this study, a series of constant water content undrained triaxial tests on idealised models of rock joints and replicated natural joints has been conducted to investigate the effect of infill saturation on the shear strength of filled joints. Although the shear strength of soil at constant water conditions has been studied in the past (e.g., Thu *et al.* [12], Hamid & Miller [13]), no literature is available on infilled joints tested under unsaturated infill conditions. This study proposes a novel approach for laboratory testing of infilled joints under different initial degree of saturation of compacted infill.

2. LABORATORY INVESTIGATION

2.1 Specimen preparation

Calibration of any shear strength model requires a series of identical joint specimens, and this surely restricts the use of real joint surfaces. Therefore, for reasons of simplicity and reproducibility, idealised model rock joints with regular saw toothed surfaces were cast with a mean dip angle of 60° [8]. The jointed specimens were 54 mm in diameter with an asperity height of 2 mm and an initial asperity angle of 18° . Fig. 1 shows the mould used to prepare the specimens and a specimen obtained after casting. Indraratna [14] proposed the use of gypsum cement ($\text{CaSO}_4 \cdot \text{H}_2\text{O}$ hemihydrates, 98%) to model soft sedimentary rocks and to prepare idealised joints. This material is readily available and relatively inexpensive and can be moulded into any shape when mixed with water. The unconfined compressive strength is independent of time once the chemical hydration is complete. The properties of the material depend on the gypsum cement to water ratio used to mix it. A consistent unconfined compressive strength (σ_c) of 65-70 MPa was obtained for a plaster to water ratio of 7:2 by weight after two weeks of curing. After being removed from the mould, the plaster specimens were cured for two weeks in an oven at a controlled temperature of $40^\circ - 45^\circ\text{C}$. The plaster specimens were immersed in water for at least 72 hours, and subsequently an organic

waterproof sealant was applied over the surfaces of the joints to ensure that the infill water content remained constant throughout shearing. Indraratna *et al.* [8] has shown that this thin layer of waterproofing sealant has negligible effect on the shearing resistance of the joint surface. Additional specimens replicating the roughness profiles of natural joints were also prepared. The details of the preparation method are given in the subsequent sections.

A silty clay (25% fine sand and 75% kaolinite) with a liquid limit of 39 and a plastic limit of 20 was used as the infill material. The shear strength behaviour of the infill material under saturated conditions was studied using a direct shear box apparatus (AS 1289.6.2.2 [15]), and a friction angle of $\phi' = 21^\circ$ and a cohesion intercept (c') of 13.4 kPa were obtained. The infill material was mixed in the laboratory to known moisture contents and then spread over the surfaces of the joints with a spatula. The joints were then statically compacted to a given infill thickness to asperity height (t/a) ratio with an infill dry density of 1250 kg/m³. Note that despite having infill specimens prepared with varying (t/a) ratios, the initial dry unit weight remained the same, and thus different water content resulted in initial degree of saturations varying from 35% to 85%, corresponding to a suction varying from 860 kPa to 165 kPa (see soil water characteristic curve later). An example of the final joint profile obtained once the infill was spread to a certain thickness is shown in Fig. 1c. Although the behaviour of the joints with infill is studied for a wide range of the infill initial degree of saturation, fully saturated conditions were not considered. This is owing to the difficulty in preparing viable specimens using this procedure, that is, because the infill became slurry-like when it approached saturation. After assembly the infilled joint specimens were wrapped in a thin neoprene (impervious) membrane. During testing, the sealant applied on the surfaces of the joint and the impervious membrane ensured that the clay infilled joints maintained constant water content conditions. Furthermore, as the permeability of the model rock is much lower than that of the infill ($k_{infill}/k_{rock} > 1000$), it could be assumed that undrained conditions would still prevail.

2.2 Testing procedure

The high-pressure two-phase triaxial apparatus (Fig. 2) developed at the University of Wollongong [16] was customised for this study. The cylindrical chamber can accommodate samples of 54 mm and 60 mm in diameter with a height-to-diameter ratio up to 2.0. Silicon oil was used as the confining fluid as it does not react with the steel cell or with the latex membrane. A variable constant strain mechanical driving system was installed to apply a

constant strain rate. Vertical displacement was measured using a Linear Variable Differential Transformer (LVDT) with an accuracy of 0.001 mm while a system with laser beams with accuracy of 0.001 mm was used to measure the horizontal movement of the joint. All the measuring devices were connected to a data-logger which was then connected to a computer for continuous data logging.

Although the testing was carried out adopting undrained conditions for the water phase, the air phase remained drained throughout the test. The specimen of infill was tested at its initial water content. While the change in suction could not be monitored during the compression stage, its value at the beginning of the shearing stage could be captured following the procedure outlined by Oloo & Fredlund [17]. A strain rate of 0.01 millimetres per minute was adopted for the test series. This shearing rate is consistent with those adopted for constant water conditions in the previous studies (i.e. Fredlund *et al.* [18] and Thu *et al.* [12]).

The infilled jointed specimen with known moisture content (or initial degree of saturation) was assembled inside the cell and the confining pressure was applied. Three different confining pressures of 300 kPa, 500 kPa and 900 kPa were used in this study to investigate the effect of normal stress (or the confining pressure) on shear strength. As shown in Fig. 1c, the mean joint plane is inclined at 60° to the horizontal plane, and the normal stress acting on the joint plane varies during shearing even though the confining pressure is kept constant. A series of tests was carried out for (t/a) ratios of 0.5, 1.0 and 3.0 and a starting infill saturation of 35%, 50%, 60%, 70% and 85% corresponding to water contents of 15%, 20%, 25%, 30%, and 35%, respectively. The stress-strain and dilation responses were observed and the peak shear strength was considered as the maximum shear strength obtained on the stress-strain curve. Dilation of the joint was calculated normal to the joint surface using independent measurements of axial and horizontal displacements. At the end of each test, the moisture content of the infill was measured and compared with the initial value. The difference between the two measurements was typically less than 0.1, which indicates that constant water conditions were attained.

To adequately describe the hydraulic properties of a material under unsaturated conditions, it is necessary to establish a relationship between suction and the amount of water present, which can be expressed in terms of either water content also known as gravimetric water content (mass), volumetric water content, or degree of saturation (volume). This relationship is often referred as the soil-water characteristic curve or SWCC. The soil-water characteristic curve for the infill material prepared at the same dry unit weight was developed

using test data obtained from the pressure plate apparatus and chilled mirror hygrometer, according to the standard procedure described in ASTM [19]. Two distinct data sets obtained from the pressure plate and chilled mirror hygrometer are shown in Fig. 3 for a wide range of matric suction that defines the SWCC. Pressure plate apparatus was used to obtain data under matric suctions up to 1000 kPa and the chilled mirror hygrometer was used for the range above 1000 kPa. The pressure plate extractor consists of a saturated high air entry value (HAEV) ceramic disk contained in an air pressure chamber. In this study, two ceramic disks with HAEVs of 500 kPa and 1500 kPa were used. The HAEV ceramic disk was saturated and was in contact with water in a compartment below the disk that was connected to an exterior burette. The infill material was then prepared in a retaining cell to a dry density of 1250 kg/m³ and was saturated by inundation in a tray. After saturation the sample weight was measured and it was placed inside the pressure chamber. Then the desired suction was applied to the system by the pressure differential of air and water phases (i.e. $u_a - u_w$; air pressure was increased while the water pressure was kept at atmospheric pressure) that was maintained until the specimens reached equilibrium. The equilibrium state can be identified by observing the air-water interface of the burette attached to the water compartment of the pressure chamber. Once the equilibrium state was established (typically after seven days or when the air-water interface changed less than 1 ml over a period of 48 hours), the pressure chamber was opened and the specimens and their retaining rings weighed to determine the corresponding water content. The specimens were then placed back on the porous plate and the next increment of suction was applied by increasing the air pressure. The process was continued until the required matric suction increment was reached.

For the matric suction values exceeding the air entry values of the pressure plate ceramic disk, alternative techniques such as the chilled mirror hygrometer can be used. Note that while the chilled mirror hygrometer apparatus measures total suction, the osmotic component should be negligible as there are no salts present in the infill material. The chilled mirror hygrometer consists of sealed chamber with a fan, a mirror, a photoelectric cell, and an infrared thermometer. In this case, the infill material is placed in a plastic container with a diameter of 40 mm, which is then placed on a tray and inserted in to a temperature controlled chamber at which point, the infill material reaches equilibrium with the chamber environment in a matter of minutes. Inside the chamber the temperature is reduced until it the material reaches its dew-point. At this time, the chilled mirror hygrometer computes the total suction based on the vapour pressures and temperature inside the chamber using Kelvin's equation. Prior to the suction measurements the device was calibrated using a solution of 0.5 M KCL

provided by the manufacturer (Decagon Devices).

The data obtained by the two methods (i.e. pressure plate and chilled mirror hygrometer) was interpolated using the well-known van Genuchten [20] relationship fitted to the experimental data using the least square method. The interpolated best fit parameters are shown in Fig. 3 along with the experimental data.

The silty clay seam inside the rock joint that was tested varied from 1 mm to 6 mm in thickness (corresponding to t/a ratios of 0.5 to 3.0). Since this was a relatively thin seam, it was not feasible to install a pore water pressure or a suction probe inside the joint to measure the negative pore pressures, as there was a distinct possibility of it being damaged during shearing. It was therefore not feasible to monitor the change in the matric suction while the joint was being sheared. The direct shear tests conducted on statically compacted kaolin reported by Tarantino & Tombolato [21] show that for applied vertical stresses of 300 and 600 kPa the changes in suction monitored during direct shearing were generally small. Limited variations of suction during the shear stage under constant water content conditions have also been reported by Thu *et al.* [12] and Rahardjo *et al.* [22] for tests of the same material. Therefore, the value of suction at the start of testing incorporated in the current shear strength analysis is reasonable. In addition, during the shearing process, the broken asperities commonly contaminate the infill material due to asperity overriding in thin infill seams. In this instance, the final value of suction computed using methods such as the filter paper technique may not be realistic. Therefore, in subsequent sections of this paper the shear strength behaviour of the infilled joints is modelled considering the initial suction of the infill computed at the start of test as a reference value.

3. RESULTS AND DISCUSSION

Forty eight undrained triaxial tests were carried out on idealised saw-tooth joints for confining pressures of 300, 500 and 900 kPa, and at initial degrees of saturation of 35%, 50%, 60%, 70%, 85% with the t/a ratios of 0.5, 1.0 and 3.0. Test results for 63 triaxial tests on replicated natural joint surfaces were also carried out and will be discussed later in this paper. An example of the test results obtained from the undrained triaxial testing of infilled, idealised rock joints is presented and discussed here. Fig. 4 illustrates the selected plots of deviator stress and dilation against axial strain for varying values of infill thickness and initial degrees of saturation for the specimens tested under a confining pressure of 900 kPa for t/a ratios varying from 0.5 to 3.0. For the purpose of brevity, only the typical results for the

degree of saturation of 50% and 85% are shown in Figs. 4 (a) and (b). The associated dilation was measured as a function of the normal displacement of the joint with respect to the shear plane, considering both the horizontal and vertical components of the joint displacement vector, as presented in Figs. 4(c) and (d). The shear stress of an infilled joint depends mainly on the thickness of the infill. When the infill is comparatively thin (i.e., $t/a = 0.5, 1.0$), the stress - strain plot exhibits two distinct stages of change in curvature (Indraratna *et al.* [8]). The first stage corresponds to the yielding of the soil infill. Beyond this stage asperity interference starts to prevail causing a significant increase in dilation. This is particularly evident in the $t/a = 0.5$ data shown in Fig. 4 for an axial strain of approximately 0.5%. The second stage corresponding to the peak deviator stress is largely influenced by rock-rock contact (albeit some effect of the infill is still present). This double-stage phenomenon becomes less pronounced as the infill increases in thickness. When t/a reaches a value of 3.0, there is no noticeable double-stage response in deviator stress, which implies that the shear behaviour is largely controlled by the soil infill alone. This is also confirmed by the dilation response since fully compressional behaviour was observed with thicker infill. When the normal stress or confining pressure increased from 300 kPa to 900 kPa, the deviator stress increased as expected showing a higher peak shear stress.

When the degree of saturation of the infill increased from 50% to 85%, the deviator stress was reduced and the axial strain required to reach peak deviator stress increased slightly (Figs. 4, 5). In addition, the volumetric behaviour observed is mainly compressive when the degree of saturation of the infill is relatively high whereas specimens with a lower degree of saturation exhibit predominantly dilative behaviour. This is owing to the fact that when matric suction increases, the infill becomes stiffer (compressibility reduced) as 'bonding' of infill material particles is enhanced. Similar experimental observations have been made in constant water content tests of residual soil [12]. The predominately dilative behaviour observed after the initial compression is due to both asperity interference (i.e., local sliding and overriding) and some asperity degradation.

To better understand the shear strength behaviour of the infill material alone, a series of undrained triaxial tests was conducted on specimens compacted with different initial water contents yielding the same dry densities and initial degrees of saturation adopted for the joint testing. As expected, the peak shear stress of the silty-clay infill material was observed to decrease with an increase in the initial degree of saturation (S_0). In Fig. 5 the behaviour of infill alone is contrasted to that of the infilled joints. It is observed that the ductile infill specimens continue to shear at high axial strains to attain the peak deviator stresses (>15%),

in comparison with the relatively brittle infilled joints ($t/a = 0.5$ and 1.0) that attain their peak deviator stresses at axial strains of less than 5%. For the infilled joint specimens, large post-peak strains could not be accommodated in the high pressure triaxial equipment in order to prevent damage to the thin latex membrane. As shown in Fig. 5 for $t/a = 3.0$, the infilled joints behave in a ductile manner similar to that of the infill alone. However, the influence of the rock interfaces is still present producing higher peak stresses than for the infill alone. The large difference in size between the cylindrical triaxial specimens of infill material and the relatively thin seam of infill sheared between the joint walls within narrow and constrained boundaries could contribute to this difference. In addition, the specimens of infill alone showed significant bulging without producing a distinct slip surface or shear band.

4. THEORETICAL BACKGROUND

The thickness of the infill is the most important parameter controlling the shear strength of an infilled joint. Several approaches have been taken to investigate the effect of the infill thickness. In most of these studies the infill thickness (t) is usually normalised by the height of the joint asperity (a), giving the t/a ratio to account for the effect of joint roughness. These models are based on a concept of a reduction in the shear strength with an increasing t/a ratio ((Ladanyi & Archambault [3], Phien-wej *et al.* [23], de Toledo & de Freitas [6], Papaliangas *et al.* [24], Indraratna *et al.* [7-9, 16]). Indraratna *et al.* [7] proposed a conceptual model to describe the peak shear strength of a soil-infilled joint (Fig. 6). According to this model, the peak shear strength of an infilled joint is the sum of two algebraic functions A and B which are assumed to provide a normalised shear strength for ratios of infill thickness to asperity height (t/a) that are less than the critical value of the ratio of infill thickness to asperity height $(t/a)_{cr}$. The relevant shear strength parameters are given by:

$$A = \tan(\phi_b + i)(1 - k)^\alpha \quad (1)$$

$$B = \tan \phi_{fill} \left(\frac{2}{1 + 1/k} \right)^\beta \quad (2)$$

$$\frac{\tau_p}{\sigma_n} = A + B = \tan(\phi_b + i)(1 - k)^\alpha + \tan \phi_{fill} \left(\frac{2}{1 + 1/k} \right)^\beta \quad (3)$$

where, ϕ_b is the basic friction angle of the joint surfaces, i is the initial asperity angle of the undulations, ϕ_{fill} is the friction angle of the infill, $k = (t/a)/(t/a)_{cr}$, $(t/a)_{cr}$ is the critical t/a

ratio of the joint, σ_n is the normal stress and α and β are empirical constants defining geometric loci of the functions A and B .

For rough joints without infill, the normalised shear strength is equivalent to $(\tan(\phi_b + i))$, as proposed by Patton [25] for clean joints. Function A models the decrease in the influence of rock-to-rock contact with an increasing (t/a) ratio, while Function B gradually increases the frictional component of the infill strength until the critical thickness to asperity height ratio $(t/a)_{cr}$ is reached [7]. Depending on the critical t/a ratio, two main regions can be identified in the model: the ‘zone of interference’ ($t/a < (t/a)_{cr}$) and the ‘zone of non-interference’ ($t/a > (t/a)_{cr}$) (Fig. 6). In the non-interference zone the shear strength is governed by infill only as no rock-to-rock contact occurs during shearing. Therefore, the normalised shear strength is given by the peak friction angle of the infill $(\tan \phi_{fill})$ in that zone. In this model, the infill was not considered to be partially saturated in which case the matric suction plays a significant role. Moreover, the cohesion of the infill was ignored by Indraratna *et al.* [7]. The model proposed in the present study considers both the cohesion and the effect of matric suction for unsaturated infill material.

5. SHEAR STRENGTH MODEL WITH UNSATURATED INFILL

Even though the shear strength of an infilled joint is primarily governed by the infill thickness, the infill properties such as the degree of saturation can also be expected to have an influence on its strength. It is possible that the dilation of the joint and the critical t/a ratio will also be influenced by the extent to which the infill has been saturated. To incorporate the effect of infill saturation, the peak shear strength model proposed earlier by Indraratna *et al.* [7] is modified in this study. For a general soil-infilled joint, the peak unsaturated shear strength ($\tau_{p,unsat}$) can be expressed as:

$$\tau_{p,unsat} = A' + B' \quad (4)$$

where, A' is the strength contribution of the joint surfaces and B' is the strength contribution of the infill material.

Function A' will have an optimum value equivalent to the shear strength of a clean joint when there is no infill in the joint, and a minimum value of zero where there is no contact with the rock wall. Function B' increases from zero to its maximum value (i.e., the shear strength of the infill alone) with infill thickness increasing up to the critical t/a ratio (Fig. 7). The change in degree of saturation of the infill only affects the shear strength of the infill material, and therefore will not impact on function A' which will remain exactly the

same as in the conceptual model (Eq. 1) of Indraratna *et al.* [7]. In contrast, function B' needs to be modified as it models the variation in the shear strength of the infill material which will vary with the degree of infill saturation. The shear strength of a partially saturated soil includes a strength contribution from the matric suction in addition to the strength of a soil in a fully saturated state. The shear strengths of partially saturated soils have been studied in the past (e.g., Bishop *et al.* [26], Fredlund *et al.* [18], Vanapalli *et al.* [27], Sheng *et al.* [28]), and these studies have proposed a range of models. A comprehensive review of unsaturated soil models has been presented by Gens [29]. The model proposed by Vanapalli *et al.* [27] predicts the shear strength using the soil-water characteristic curve. The revised model capturing the degree of saturation can be expressed by:

$$\tau_{p,unsat} = c' + (\sigma_n - u_a) \tan \phi' + (u_a - u_w) \left[(\tan \phi') \left(\frac{S_r - S_{res}}{100 - S_{res}} \right) \right] \quad (5)$$

where $\tau_{p,unsat}$ is the shear strength of an unsaturated soil, c' is the effective cohesion of saturated soil, $(\sigma_n - u_a)$ is the net normal stress, ϕ' is the effective angle of shearing resistance for a saturated soil, $(u_a - u_w)$ is matric suction of the soil, S_r is the degree of saturation of the soil, and S_{res} is the residual degree of saturation of the soil.

The conceptual development of the shear strength model for partially saturated infilled joints is illustrated in Fig. 7. This model defines two different zones distinguished by the thickness of the infill, the ‘interference zone’ and the ‘non-interference zone’. For convenience in modelling, a ratio k_s is introduced so that the boundaries of the zone can be clearly expressed, i.e., $k_s = (t/a)_s / (t/a)_{cr,s}$, where $(t/a)_{cr,s}$ is the critical t/a ratio of an infilled joint with a degree of saturation of s , and $(t/a)_s$ is the t/a ratio of a given infilled joint with an infill saturation of s .

For $k_s < 1$ in the ‘interference zone’, the function A' based on rock joint surfaces is given by:

$$A' = \{\sigma_n \tan(\phi_b + i)\} (1 - k_s)^\alpha = \sigma_n A \quad (6)$$

where α is an empirical constant and A is the original parameter from Eq. (1).

The unsaturated shear strength equation given by Eq. (5) is used to develop the function B' . The algebraic component, $c' + (u_a - u_w) \left[(\tan \phi') \left(\frac{S_r - S_{res}}{100 - S_{res}} \right) \right]$ of Eq. (5) is a constant for a given initial degree of saturation of the infill according to the soil water characteristic curve. This will be referred to here as the total cohesion intercept (c_t) given by:

$$c_t = c' + (u_a - u_w) \left[(\tan \phi') \left(\frac{S_r - S_{res}}{100 - S_{res}} \right) \right] \quad (7)$$

Equation (5) can then be re-written as:

$$\tau_{p,unsat} = c_t + (\sigma_n - u_a) \tan \phi' \quad (8)$$

The second function B' , influenced by infill is given by:

$$B' = c_t + [(\sigma_n - u_a) \tan \phi'] \left(\frac{2}{1 + 1/k_s} \right)^\beta \quad (9)$$

If the infill is saturated, $B' = \sigma_n \cdot B + c_t$, where B is the original parameter in Equation (2).

For $k_s = 0$, there is no infill material in the joint and the equation is simplified to the shear strength of a clean joint,

$$\tau = \sigma_n \cdot \tan(\phi_b + i) \quad (10)$$

For the rock-infill interference zone ($0 < k_s < 1$),

$$\tau = \{(\sigma_n - u_a) \tan(\phi_b + i)\} (1 - k_s)^\alpha + c_t + [(\sigma_n - u_a) \tan \phi'] \left(\frac{2}{1 + 1/k_s} \right)^\beta \quad (11)$$

For the non-interference zone ($k_s > 1$), where the shear strength is governed only by the infill material,

$$\tau = c_t + [(\sigma_n - u_a) \tan \phi'] \quad (12)$$

The above equations can be simplified when the infill is fully saturated (i.e., $u_a - u_w = 0$). In this case Eqs. 11 and 12 revert back to those proposed earlier by Indraratna *et al.* [7], except for the addition of the cohesion term.

5.1 Verification of the shear strength model

The peak shear strength of a joint was taken as the point at which the maximum deviator stress was reached. It was noted that when the t/a ratio increased, the peak shear strength decreased rapidly until a critical t/a was reached, beyond which any further decline in strength was insignificant. This observation leads us to hypothesise that beyond this critical ratio $(t/a)_{cr}$ the shear strength is exclusively a function of the infill properties [8].

The results of 48 undrained triaxial tests conducted on silty clay-infilled saw toothed joints were used to validate the shear strength model presented earlier. Test data for five different initial degrees of saturation were collected for joints having infills that varied in thickness from 1 mm to 8 mm, corresponding to t/a ratios of 0.5 to 4.0. With close observation of the

shear strength data with corresponding t/a ratios, it was noted that the critical t/a differs with the tested water contents. The variation of the critical t/a ratio with the initial degree of saturation of the infill is illustrated in Fig. 8 for wetting conditions. These critical values were obtained from the diminishing trends of peak shear stress with increasing t/a ratio, for instance, as shown in Fig. 9. Idealised joints tested with critical t/a values of 1.7 to 2.5 gave a slightly non-linear relationship (3rd order polynomial) between the critical t/a ratio and the initial degree of saturation of the infill, for the range of t/a values tested.

As $(t/a)_{cr}$ varies with the initial degree of saturation for the infilled joint, the parameter k_s was introduced to better describe the interference and non-interference zones. Asperity interference occurs when the infill thickness ratio is less than the critical thickness ratio (i.e., $k_s < 1$). When $k_s > 1$, the shear behaviour is governed solely by the infill material. The decrease in peak shear strength with increasing t/a ratio is represented quite well by the ‘decay function’ introduced in Eqs. (6) and (9). As shown by Fig. 9, the laboratory test data for the silty clay-infilled saw toothed joints with different initial degrees of saturation verifies the mathematical model represented by Eqs. (10) to (12). The algebraic function A' represents the decay of the maximum joint friction, while function B' models the increasing role of the infill angle of friction. The empirical parameters α and β for different initial degrees of saturation determined using a regression analysis are tabulated with the respective critical t/a ratios in Table 1.

The proposed peak shear strength model allows the shear strength of a given combination of infill-joint profile at any t/a ratio and initial degree of saturation to be determined, as long as the empirical constants α and β in Eqs. (6) and (9) can be evaluated from the results of undrained laboratory tests. The required values of the basic friction angle of the joint surfaces and the properties of the infill material can be determined by laboratory tests, if estimates cannot be made from the available literature. The initial matric suction and residual degree of saturation can be obtained using the soil-water characteristic curve of the infill material.

6. SHEAR BEHAVIOUR OF REPLICATED NATURAL JOINTS AND MODEL APPLICATION

The idealised joint profile used to develop the shear strength model has obvious limitations when compared to natural joint behaviour. This section compares the relative differences briefly, and offers an extension of the idealised joint model that can be applied to

natural joint profiles having a joint roughness coefficient (JRC) of 10 – 12. These joints were obtained from the rock slide at Kangaroo Valley, New South Wales, and were replicated on to gypsum plaster using silicone rubber (Fig. 10). The samples were 54 mm in diameter (as were the idealised specimens) and a mean asperity amplitude (a) of 3.91 mm was measured from the joint surface. The roughness amplitude or asperity height of the natural joint was determined according to the ISRM [30] recommendations. Infill thickness (t) was measured at least at ten points along the surface to obtain the mean infill thickness. Infill thickness to asperity height ratio (t/a) was then determined using the mean values of asperity amplitude and infill thickness. A series of constant water content triaxial tests was conducted on infilled joints with natural roughness similar to that of the idealised filled joints at confining pressures of 300, 500 and 900 kPa for (t/a) ratios of from 0.26 to 2.05, and water content from 35% to 85%, giving a total of 63 tests. Since the shear strength and volumetric data for all confining pressures showed similar trends, only the results for a confining pressure of 900 kPa are given here as an example (Fig. 11).

Compared to the idealised shear strength data, a key feature of the replicated natural joint testing is that the laboratory data does not show a distinct peak deviator stress even after 5% strain. This is because the shearing of the triangular asperities in the idealised joints corresponds to a noticeable drop in the shear stress, while the spatially distributed asperities of the natural joint profile do not show this marked strength drop. At lower normal stresses, the asperities can mobilize the peak strength by sliding over each other. When the steepest asperity is sheared off, the contacts will be moved to the next steepest asperity, and this process will continue until a condition is reached where both sliding and shearing of asperities occur simultaneously [31]. However, if the infill thickness is higher than a critical value, the natural joints follow a similar trend to idealised joints, when the shear behaviour is governed by the infill alone. For the natural joint with JRC = 10 -12, the (t/a)_{cr} values varied from 1.0 to 1.4 (Fig. 8). It is observed that the critical t/a ratios for both idealised joints and natural joints with JRC = 10 – 12 show a slightly non-linear relationship (3rd order polynomial) with the initial degree of saturation for the same infill material. At any given confining pressure, the critical t/a ratios for idealised and natural joint profiles are different. This is not surprising, because these ratios are governed by the joint characteristics, especially the heights and distributions of the asperities. Idealised saw-tooth joints consisted of an asperity height of 2 mm giving critical t/a ratios of from 1.7 to 2.5, corresponding to an infill thickness varying from about 3.4 to 5.0 mm. In contrast, the replicated natural joints of irregular profile had a mean asperity amplitude approaching 4 mm (i.e., almost double that of

idealised joints for similar infill thickness), corresponding to critical t/a ratios in the narrower range of 1.0 – 1.4. It is also noteworthy that for the silty-clay infill, the critical t/a ratios (Tables 1 and 2) are mainly dependent on the joint profile and the degree of infill saturation, and less dependent on the applied confining pressure.

The peak shear strength of a filled joint having a natural profile can be predicted using the model developed for idealised joints with a subtle extension. For a general soil infilled joint with partially saturated infill, the peak unsaturated shear strength ($\tau_{p,unsat}$) can be expressed as:

$$\tau_{p,unsat} = A'' + B'' \quad (13)$$

where, A'' is the strength contribution of the joint surfaces and B'' is the strength contribution of the infill material.

The function A' in the idealised model that reflects Patton's [25] equation for a clean idealised joint can be modified by introducing the joint roughness characteristics proposed by Barton & Choubey [32]. For an irregular clean joint Barton & Choubey [32] proposed a model to predict its peak shear strength using the Joint roughness coefficient:

$$\tau_p = \sigma_n \tan(\phi_b + JRC \log(JCS / \sigma_n)) \quad (14)$$

where, τ_p is the peak shear strength, σ_n is the normal stress, JRC is the joint roughness coefficient, JCS is the Joint wall compressive strength and ϕ_b is the basic friction angle. This equation gives a better application than to Patton's [25] equation as the term “ i ” in Patton's equation does not account for the irregularity of the joint profile.

For $k_s < 1$ in the ‘interference zone’, the function A' of the idealised model is extended to A'' to capture the natural joint profile as:

$$A'' = [\sigma_n \tan(\phi_b + JRC \log(JCS / \sigma_n))] (1 - k_s)^\alpha \quad (15)$$

where, JRC is the Joint roughness coefficient and JCS is the joint wall compressive strength.

Function B'' is only dependent on soil parameters and it is not dependent on the joint surface profile. Hence, the function B'' would be the same as function B' of the original model.

$$B'' = c_t + [(\sigma_n - u_a) \tan \phi'] \left(\frac{2}{1 + 1/k_s} \right)^\beta \quad (16)$$

where,

$$c_t = c' + (u_a - u_w) \left[(\tan \phi') \left(\frac{Sr - S_{res}}{100 - S_{res}} \right) \right] \quad (17)$$

$$\tau_{p,unsat} = [\sigma_n \tan(\phi_b + JRC \log(JCS / \sigma_n))] (1 - k_s)^\alpha + c_t + [(\sigma_n - u_a) \tan \phi'] \left(\frac{2}{1 + 1/k_s} \right)^\beta \quad (18)$$

For the non-interference zone ($k_s > 1$), where the shear strength is governed only by the infill material,

$$\tau_{p,unsat} = c_t + [(\sigma_n - u_a) \tan \phi'] \quad (19)$$

Fig. 12 shows the validation of the model over an irregular (natural) joint profile with JRC of 10 – 12 for different degrees of infill saturation. It can be observed that the proposed model (Eqs 18-19) gives a good agreement with the laboratory triaxial data obtained for natural joint replicas. The empirical parameters α and β for different degrees of saturation were determined by multiple regression analysis similar with the idealised joints and are presented in Table 2.

7. PRACTICAL IMPLICATIONS: JOINTED ROCK SLOPE STABILITY

The use of the proposed model for natural joint profiles in a practical situation is illustrated using a simplified slope analysis problem as presented in Fig. 13 (a), where a potentially unstable rock wedge from Kangaroo Valley, New South Wales, Australia, is considered with simplified boundaries. The rock strata consisted mainly of sandstone and the joint infill was primarily composed of sandy silt and/or sandy clay depending on the elevation above the ground level below the slope. The strata close to the ground level were significantly wetter than the upper slopes. More information on Kangaroo Valley jointed rock is given by Indraratna & Haque [33]. In the current analysis, an irregular rock joint of JRC = 11 defining a wedge of weight W (simplified to 2D plane strain) with a slope angle of λ and height H was considered, having a sediment-infilled joint at a dip angle of θ (Fig. 13a). For a unit length normal to the plane of Fig. 13 (a), the weight of the wedge W with unit weight γ can be simplified to:

$$W = 0.5\gamma H^2 [\cot(\theta) - \cot(\lambda)] \quad (20)$$

Considering a surcharge load F applied to the upper boundary of the wedge, the factor of safety (FS) for sliding can be found by limit equilibrium as:

$$FS = \frac{\tau [H / \sin(\theta)]}{W \sin(\theta) + F \sin(\theta)} \quad (21)$$

where, τ is the resisting shear stress along the joint described by Eq. 18. The geometry of this particular rock wedge is approximated by $H = 30.5$ m, $\gamma = 27.5$ kN/m³, $\lambda = 80^\circ$ and $\theta = 45^\circ$ [33]. As the length of the wedge is large compared to its width and height, the wedge stability

can be conveniently analysed assuming 2D (plane strain).

The clay-infilled joint model for a $t/a = 0.7$ with the corresponding model parameters given in Table 2 was used to calculate the FS for different infill saturation of the wedge subjected to a surcharge load $F = 245$ kN applied by the passage of a typical freight train. The value of FS increased significantly from 1.07 to 1.34 with the decrease of infill saturation from 85% to 35% (Fig. 13b). If the joint infill is fully saturated, the wedge is unstable as FS approaches unity, and the joint plane would require stabilisation, for example by rock bolting. In the case of relatively dry infill ($S_r < 40\%$), the wedge would have a $FS > 1.3$. Not surprisingly, the FS of the wedge becomes asymptotic between the upper bound at which the infill is totally dry and the lower bound where the infill is fully saturated. This simplified example of the Kangaroo Valley rock wedge demonstrates the benefits of unsaturated infill existing within joints compared to the conservative assumption of fully saturated joints that is often adopted in the design of jointed rock slopes. In actual practice, the sandy clay infill in Kangaroo Valley had degrees of saturation varying from 100% in the lower most terrain to 40% in joints at higher elevations of the sandstone rock stratum.

8. MODEL LIMITATIONS

- The use of idealised saw toothed joint profiles in the experimental program was justified by the need to carry out repetitive, reproducible tests with simple geometric profiles, in order to understand and formulate a conceptual model. Even though meaningful data could be obtained from these idealised profiles, they do not accurately represent the shear behaviour of natural joint surfaces that may be irregular and or wavy. Although some steps have been taken by the authors to replicate and test natural joint profiles, there are still limitations due to the narrow range of JRC (10-12) examined in this study. Therefore, further testing of different irregular joint profiles with different natural infills is required to validate the proposed models more comprehensively.
- Scale effects (the effects of changes in joint surface wave length and asperity height) were not studied.
- The effects of different infill material, parent rock type and the extent of joint weathering were not considered in this study.
- The roughness of the joints were characterised using JRC considering the 2D roughness only. An additional design parameter should be introduced for a comprehensive 3D shear analysis of rock joints

The joints with silty clay infill alone cannot represent the entire range of joint types and infill materials represented in nature. Despite the advances the proposed model has demonstrated over previously existing models of infilled joints by capturing the role of infill unsaturation, its application is still constrained by the limitations summarised above.

9. CONCLUSIONS

This paper has discussed the influence of the initial degree of saturation of the infill and the ratio of infill thickness to asperity height (t/a) on the shear strength of idealised saw toothed and natural silty clay-infilled joints. The experimental results obtained highlight how the t/a ratio contribute to a reduction of the shear strength from the maximum value associated with clean rough joints. The shear strength behaviour is governed by t/a up to a critical thickness after which there is no significant change in strength. Further, for infill thickness exceeding this critical thickness, the influence of asperities is suppressed and the shear behaviour is mainly governed by the infill.

The effect of initial degree of saturation of the infill material also has an important effect on the shear strength. Indeed, it was observed that as the degree of saturation increases, both the shear strength and the joint dilation decrease. The effect of the confining pressure is also shown by the test data obtained under three different confining pressures, 300 kPa, 500 kPa, and 900 kPa. There is an apparent increase in shear strength when the confining pressure is increased. When the thickness of the infill is very low ($k_s < 1$), the stress-strain behaviour is influenced by asperity interference and the peak shear strength is governed by rock-to-rock contact. Due to asperity interference, joint dilation can be observed in the interference zone. The ratio $(t/a)_{cr}$ varies with the initial degree of saturation of the infill. It ranges from 1.7 to 2.5 as the initial degree of saturation increases from 35% to 85%. For relatively thick infills ($k_s > 1$), the influence of the asperities is suppressed and shearing takes place only through the infill. Therefore, no dilation was observed in the laboratory for the non-interference zone. Additional tests were performed to evaluate the similarity of the shear strength data obtained with idealised saw-tooth joints to replicated natural joints ($JRC = 10-12$). The results indicated that similar trends are obtained for both types of joints except that a clear second stage is absent for the natural joints. This is attributed to the spatial variation of asperities over the natural profile. As for idealised joints, the peak shear strength of natural joints decreases with an increase in the initial degree of saturation. Also, for the same variation of degree of saturation, the critical (t/a) ratio varies from 1.0 to 1.4 in a similar manner to the

idealised joints. For both idealised and natural joints, critical (t/a) ratios show a slightly non-linear relationship with the degree of saturation. The current study proposed a conceptual model initially developed for idealised joints with varying degrees of saturation, that could be conveniently modified to suit the natural joint profile of $JRC = 10-12$. In addition, the stability study of potential rock wedge of Kangaroo Valley showed that the degree of saturation of the infill governs the stability, highlighting important practical implications of compacted infill rock joints. While this study provides insightful results of the behaviour of infill saturation on the overall rock joint shear strength behaviour further studies with different infill materials that may yield different water retention characteristics are recommended.

ACKNOWLEDGEMENT

The Authors wish to acknowledge the contributions of Alan Grant (Technical Officer, University of Wollongong) during the laboratory testing program. The second author would like to thank the Endeavour Postgraduate Awards program for their financial support. The previous research in this area conducted by former PhD candidates at University of Wollongong (Dr. A. Haque, Dr. H.S. Welideniya, Dr. M. Jayanathan and Dr. D.A.F. Oliveira) is gratefully acknowledged. Support from the ARC Centre of Excellence for Geotechnical Science and Engineering is also acknowledged.

REFERENCES

- [1] Barton N. A review of the shear strength of filled discontinuities in rock. Norwegian Geotechnical Institute Publication 1974.
- [2] Elsworth D. A model to evaluate the transient hydraulic response of three-dimensional sparsely fractured rock masses. *Water Resources Research* 1986; 22:1809-1819.
- [3] Ladanyi B, Archambault G. Shear strength and deformability of filled indented joints. *Proc International Symposium on the Geotechnics of Structurally Complex Formations. Capri* 1977; 317-326.
- [4] Lama RD. Influence of clay fillings on shear behaviour of joints. *Proc 3rd Int Assoc Engng Geol Madrid* 1978; . 27-34.
- [5] Pereira JP. Mechanics of filled discontinuities. *Proceedings of the International Conference on Mechanics of Jointed and Faulted Rock* 1990:375-380.
- [6] de Toledo PEC, de Freitas MH. Laboratory testing and parameters controlling the shear-strength of filled rock joints. *Géotechnique* 1993; 43:1-19.
- [7] Indraratna B, Welideniya HS, Brown ET. A shear strength model for idealised infilled joints under constant normal stiffness. *Géotechnique* 2005; 55:215-226.
- [8] Indraratna B, Jayanathan M, Brown ET. Shear strength model for overconsolidated clay-infilled idealised rock joints. *Géotechnique* 2008; 58:55-65.
- [9] Indraratna B, Oliveira DAF, Brown ET. A shear-displacement criterion for soil-infilled rock discontinuities. *Géotechnique* 2010; 60:623-633.
- [10] Alonso E, Zandarín M, Olivella S. Joints in unsaturated rocks: thermo-hydro-mechanical formulation and constitutive behaviour. *J Rock Mech and Geotech Eng* 2013; 5:200-213.
- [11] Zandarin MT, Alonso E, Olivella S. A constitutive law for rock joints considering the effects of suction and roughness on strength parameters. *Int J Rock Mech Min Sci* 2013; 60:333-344.
- [12] Thu TM, Rahardjo H, Leong E-C. Shear strength and pore-water pressure characteristics during constant water content triaxial tests. *J Geotech and Geoenviron Eng, ASCE* 2006; 132:411-419.
- [13] Hamid TB, Miller GA. Shear strength of unsaturated soil interfaces. *Can Geotech J* 2009; 46:595-606.
- [14] Indraratna B., Development and applications of a synthetic material to simulate soft sedimentary rocks. *Géotechnique* 1990; 40:189-200.
- [15] Standards Australia. Soil strength and consolidation tests - Determination of the shear strength of a soil - Direct shear test using a shear box. AS 12896221998.

- [16] Indraratna B, Haque A, Aziz NI. Shear behaviour of idealised infilled joints under constant normal stiffness. *Géotechnique* 1999; 49:331-355.
- [17] Oloo SY, Fredlund DG. A method for determination of $\phi(b)$ for statically compacted soils. *Can Geotech J* 1996; 33:272-280.
- [18] Fredlund DG, Rahardjo H, Fredlund MD. *Unsaturated soil mechanics in engineering practice*. 1st ed. New York: Wiley-Interscience; 2012.
- [19] ASTM. *Standard Test Methods for Determination of the Soil Water Characteristic Curve for Desorption Using a Hanging Column, Pressure Extractor, Chilled Mirror Hygrometer, and/or Centrifuge*. West Conshohocken, PA: ASTM International; 2008.
- [20] van Genuchten MT. A closed-form equation for predicting the hydraulic conductivity of unsaturated soils *Soil Sci Soc Am J* 1980; 44:892-898.
- [21] Tarantino A, Tombolato S. Coupling of hydraulic and mechanical behaviour in unsaturated compacted clay. *Géotechnique* 2005; 55:307-317.
- [22] Rahardjo H, Heng OB, Choon LE. Shear strength of a compacted residual soil from consolidated drained and constant water content triaxial tests. *Canadian Geotechnical Journal*. 2004; 41:421-436.
- [23] Phien-wej N, Shrestha U, Rantucci G. Effect of infill thickness on shear behaviour of rock joints. In: Barton NR, Stephansson O, editors. *Rock Joints: Proceedings of the international conference on rock joints*: Balkema; 1990: 289-294.
- [24] Papaliangas T, Hencher SR, Lumsden AC, Manolopoulou S. The effect of frictional fill thickness on the shear strength of rock discontinuities. *Int J Rock Mech Min Sci & Geomech Abstr* 1993; 30:81-91.
- [25] Patton FD. *Multiple modes of shear failure in rock and related materials*. PhD thesis, Urbana: University of Illinois; 1966.
- [26] Bishop AW, Alpan I, Blight GE, Donald IB. Factors controlling the shear strength of partly saturated cohesive soils. *Proceedings, ASCE research conference on shear strength of cohesive soils*. University of Colorado, Boulder, CO., 1960: 503-532.
- [27] Vanapalli SK, Fredlund DG, Pufahl DE. The relationship between the soil-water characteristic curve and the unsaturated shear strength of a compacted glacial till. *Geotech Testing J* 1996; 19:259-268.
- [28] Sheng D, Fredlund DG, Gens A. A new modelling approach for unsaturated soils using independent stress variables. *Can Geotech J* 2008; 45:511-534.
- [29] Gens, A. Soil-environment interaction in geotechnical engineering. *Géotechnique* 2010; 60: 3-74.

- [30] ISRM (International Society for Rock Mechanics). Suggested methods for the quantitative description of discontinuities in rock masses. *Rock characterisation, testing and monitoring – ISRM suggested methods*, E. T. Brown ed., Pergamon: Oxford; 1981: 319-368.
- [31] Kimura T, Esaki T. A new model for the shear strength of rock joints with irregular surfaces. In: Rossmann HP, editor. *Mechanics of jointed and faulted rock*. Rotterdam: Balkema; 1995: 133-137.
- [32] Barton N, Choubey V. The shear strength of rock joints in theory and practice. *Rock Mech Rock Eng* 1977; 10:1-54.
- [33] Indraratna B, Haque A. *Shear behaviour of rock joints*. Rotterdam: Balkema; 2000.

LIST OF TABLES

Table 1. Empirical constants and critical t/a ratios for different infill saturations for idealised saw-tooth joints

Table 2. Empirical constants and critical t/a ratios for different infill saturations for replicated natural joints

LIST OF FIGURES

Fig. 1. . (a) The mould and (b) simulated joint surface profile of the idealised saw toothed joint (c) with the infill material

Fig. 2. High Pressure Triaxial Apparatus for Rock Joint Shearing, designed and built at University of Wollongong

Fig.3. Soil-water characteristic curve for the infill material

Fig. 4. Shear behaviour of idealised infilled joints with different initial degrees of infill saturation under confining pressure of 900 kPa

Fig. 5. Comparison of shear behaviour of infilled rock joints and infill alone at a confining pressure of 900 kPa

Fig. 6. Conceptual normalised shear strength model for infilled rock joints (modified from Indraratna *et al.*, 2005)

Fig 7. Extended peak shear strength model including the effect of infill saturation

Fig. 8. Variation of critical (t/a) ratio with initial degree of saturation

Fig. 9. Verification of shear strength model with experimental data for infill saturations of (a) 50%, (b) 85%

Fig. 10 (a). Natural joint profile used for laboratory testing, (b) Section A-A: (Arrow indicates the shearing direction), (c) Section B-B

Fig. 11. Shear behaviour of replicated natural infilled joints with different initial degrees of infill saturation under confining pressure of 900 kPa

Fig. 12. Validation of shear strength model with experimental data for replicated natural joint profiles for infill saturations of (a) 50%, (b) 85%

Fig. 13 (a) Rock slope with a soil infilled discontinuity, (b) Factor of safety with the degree of saturation of the infill and initial matric suction

1

Table 1. Empirical constants and critical t/a ratios for different infill saturations for idealised saw-tooth joints

Degree of saturation (water content, %)	$(t/a)_{cr}$	α	β
35% (15%)	1.7	1.2	2.5
50% (20%)	1.9	1.4	2.2
60% (25%)	2.1	1.6	1.9
70% (30%)	2.3	1.8	1.7
85% (35%)	2.5	2.0	1.5

2

3

4

5

6

1 **Table 2. Empirical constants and critical t/a ratios for different infill saturations for**
2 **replicated natural joints**

Degree of saturation (water content, %)	$(t/a)_{cr}$	α	β
35% (15%)	1.0	1.3	3.0
50% (20%)	1.1	1.5	2.6
60% (25%)	1.2	1.7	2.3
70% (30%)	1.3	2.0	1.8
85% (35%)	1.4	2.2	1.5

3

4

5

Fig. 1. (a) The mould and (b) simulated joint surface profile of the idealised saw toothed joint (c) with the infill material.

1
2
3
4

5
6
7
8

Fig. 2. High Pressure Triaxial Apparatus for Rock Joint Shearing, designed and built at University of Wollongong

Fig.3. Soil-water characteristic curve for the infill material.

Fig. 4. Shear behaviour of idealised infilled joints with different initial degrees of infill saturation under confining pressure of 900 kPa.

Fig. 5. Comparison of shear behaviour of infilled rock joints and infill alone at a confining pressure of 900 kPa.

1

2

3 Fig. 6. Conceptual normalised shear strength model for infilled rock joints (modified from
 4 Indraratna et al., 2005).

5

6

1

2

3 Fig 7. Extended peak shear strength model including the effect of infill saturation.

4

5

6

Fig. 8. Variation of critical (t/a) ratio with initial degree of saturation

1

2

3
 4 Fig. 9. Verification of shear strength model with experimental data for infill saturations of (a)
 5 50%, (b) 85%

6

(a)

(b)

(c)

Fig. 10(a). Natural joint profile used for laboratory testing, (b) Section A-A: (Arrow indicates the shearing direction), (c) Section B-B

Fig. 11. Shear behaviour of replicated natural infilled joints with initial degrees of infill saturation of 50 and 85% under confining pressure of 900 kPa

3
 4
 5 Fig. 12. Validation of shear strength model with experimental data for replicated natural joint
 6 profiles for infill saturations of (a) 50%, (b) 85%

Fig. 13 (a) Rock slope with a soil infilled discontinuity

Fig. 13 (b) Factor of safety with the degree of saturation of the infill and initial matrix suction