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Development of sediment-infilled rock joint models and implications on rockmass stability

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Abstract

Soil-infilled discontinuities adversely influence the stability of rockmass, because, the infill materials especially when saturated, drastically reduce the shear strength. The angle of shearing resistance of a discontinuity decreases significantly for increasing infill thickness. Once it reaches a critical thickness, the shear strength of the discontinuity will be governed only by the infill material and the rock-walls effect becomes negligible. Owing to the lack of research on the shear behavior of infilled rock joints, it has been common practice to assume that the shear strength of the joint is that of the infill material alone. This assumption can often lead to significant underestimation of the joint strength. This paper provides a critical analysis of selected soil-infilled joint models and the peak shear strengths were compared with experiental data. For the Interference zone, the modified shear displacement model better predicts the peak shear strength of the joint. For the Non-Interference zone all the models predict peak shear strength accurately.

1 INTRODUCTION

The presence of various discontinuities such as joints, faults and fractures will reduce the strength of a rock mass. If these discontinuities are filled with soft material, the strength will be even more reduced. The infill material may be fine sediments transported by tectonically crushed rock material or the product of rock joint weathering of joints.

Many studies have been carried out to investigate the behavior of clean joints (Ohnishi & Dharmaratne, 1990; Indraratna et al., 1999) but only limited studies have been conducted for infilled joints (Kanji, 1975; Ladanyi & Archambault, 1977; Lama, 1978). Shear strength of infill joint is often assumed as the strength of the infill alone. This assumption may eventually lead to the prediction of joint strength to be underestimated as the rock to rock contact is neglected. Some infilled joints gain strength over time due to bonding and consolidation. However, these joints may be weakened again upon subsequent joint movement (Indraratna et al., 2008).

Most laboratory testing on infilled joints have been carried under constant normal load (CNL) or zero normal stiffness conditions (e.g. Lama, 1978; Pereira, 1990; Phien-wej et al., 1990; de Toledo & de Freitas, 1993). Even though CNL testing has been preferred until recent times, constant normal stiffness (CNS) conditions are more likely to be representative of non-planar joints in which dilation takes place as a result of shearing, and the surrounding rock mass inhibits some of this dilation. CNS conditions are more appropriate for jointed slopes and underground excavations, and some researchers have identified the importance of CNS testing over CNL testing (e.g. Ohnishi & Dharmaratne, 1990; Haberfield & Johnston, 1994).

The infill thickness also plays a major role in shear strength of an infilled joint. It has been observed that the shear strength of a joint varies from the clean joint strength to that of the infill alone, when the infill thickness increases (Indraratna et al., 2005). The role of infill thickness is often modeled by the thickness to asperity height ratio (t/a). Once the thickness reaches a critical value the joint strength approaches the shear strength of infill. Owing to the lack of a generalized model that correctly simulates the behavior of soil-infilled joints under CNS, especially under mining and underground tunneling conditions, limited attempts to capture numerous factors affecting infilled joints have been made in recent years (Indraratna et al., 2005, 2008, 2010).

2 EXSISTING MODELS FOR SEDIMENT-INFILLED ROCK JOINTS

2.1 Models Based on Shear Strength Drop Due to Sediment Infill

An empirical model based on experimental results was proposed by Phien-wej et al. (1990) under CNL conditions. They observed that for low asperity angles ($i = 15^o$) the peak shear strength envelope is linear for all infill thicknesses. For rougher surfaces ($i = 30^o$) the strength envelopes were bilinear for a thin infill layer, while they again became linear for a relatively thick infill. They also observed three zones depending on the (t/a) ratio. Interlocking condition were observed when (t/a) ratio was less than 0.5 while non-interfering condition was observed when (t/a) was greater than 2.

$$\frac{\tau_p}{\sigma_n} = \frac{\tau_0}{\sigma_n} - \frac{k_1}{\sigma_n}(t/a)\exp[k_2(t/a)] \tag{1}$$

where τ_p is the peak shear strength of the infilled joint; τ_0 is the peak shear strength of the clean joint at same normal stress; σ_n is the normal stress; and k_1 and k_2 are constants that vary with surface roughness of joint and normal stress. The equation has been formed by subtracting the drop in peak shear strength due to the presence of infill.

Another model based on CNS testing was proposed by Indraratna et al. (1999) capturing the shear drop due to the presence of infill. They introduced a term called Normalised shear drop (NSD) which is the drop of shear strength due to the infill material normalized by the initial normal stress.

$$(\tau_p)_{\text{infilled}} = (\tau_p)_{\text{unfilled}} - \sigma_{n0}\frac{(t/a)}{\alpha(t/a)+\beta} \tag{2}$$

where, $(\tau_p)_{\text{infilled}}$ is the peak shear strength of infilled joint; $(\tau_p)_{\text{unfilled}}$ is the peak shear strength of clean joint; α and β are empirical constants depending on initial normal stress and surface roughness.

2.2 Models Based on Cumulative Shear Strength of Rock Interface and Infill Components

A model to predict shear behavior of clay filled joints under CNL conditions was proposed by Ladanyi & Archambault (1977). In their analytical investigation, two equations based on breakage of irregularities were proposed. When there is insignificant breakage, the equation becomes the same as the basic governing equation introduced by Patton (1996). For no breakage, Ladanyi & Archambault (1977) propose;

$$\tau_p = \frac{c_u}{(1-\tan i\cdot\tan\phi_b)} + \sigma_n\tan(\phi_b + i) \tag{3}$$

where, τ_p is the peak shear strength; c_u is the undrained shear strength of the infill; σ_n is the normal stress; ϕ_b is the basic friction angle of the rock interface; i_o is the initial asperity angle; i is the peak dilation angle estimated by $\tan i = m\tan(i_0)$; m is an empirical reduction factor varying from 0 to 1, presented as:

$$m = \left(1-\frac{2}{3}\left(\frac{t}{a}\right)\right)^2 \quad \text{for } (t/a) \le 1.5$$
$$m = 0 \quad \text{for } (t/a) > 1.5 \tag{4}$$

where t/a is the thickness of infill to asperity height ratio.

For breakage of irregularities during shear Ladanyi & Archambault (1977) has proposed another equation based on the following principle: "*The strength of a filled joint, in which the filling and the irregularities are sheared simultaneously, is located between the strength of an empty joint and that of the filling alone and varies with the thickness of filling and the normal pressure*". This can be mathematically expressed by:

$$S = m(R - C) + C \tag{5}$$

where, S is the shear strength of the infilled joint; R is the shear strength of the clean joint given by $R = \sigma_n\tan(\phi_b + i)$; C is the shear strength of the infill given by $C = c_u + \sigma_n\tan\phi_u$; ϕ_u is the undrained friction angle of the infill material; and i is the peak dilation angle for the clean joint given by:

$$\tan i = \left[1-(\sigma_n/\sigma_c)^{\frac{1}{4}}\right]\tan i_0 \tag{6}$$

where σ_c is the compressive strength of the intact rock.

An empirical relation was proposed by Papaliangas et al. (1990) to predict the peak shear strength of infilled joints. From laboratory experiments they have shown that peak shear strength varies between maximum and minimum limits. These limits vary with the infill thickness (t), type of infill, roughness of the rock surface and normal stress.

$$\mu = \mu_{min} + (\mu_{max} - \mu_{min})^n \tag{7}$$

where, $\mu = (\tau/\sigma)\times 100$, $\mu_{max} = (\tau_{max}/\sigma)\times 100$, $\mu_{min} = (\tau_{min}/\sigma)\times 100$, and n is a function of the infill thickness given by:

$$n = \left[1-\frac{1}{c}(t/a)\right]^m \tag{8}$$

In the above, c and m are empirical constant. The constant c is defined as the ratio t/a at which the minimum shear strength is attained. The constant m needs to be evaluated in advance for various t/a ratios. This model lacks some of the major parameters involved in shear strength theories such as the friction angle of joint surface, shearing resistance of the infill and the dilation angle.

2.3 Normalised Peak Shear Strength Model

The conceptual development of a semi-empirical model is shown in Figure 1. The model is based on two algebraic functions A and B. Function A is introduced to model the decrease in the influence of the shear strength contribution from the rock surface with increasing (t/a) ratio, which is equal to $\tan(\phi_b + i)$ as proposed by Patton (1966). The function B increases with the t/a ratio until the critical t/a ratio is reached. When the t/a ratio reaches its critical value,

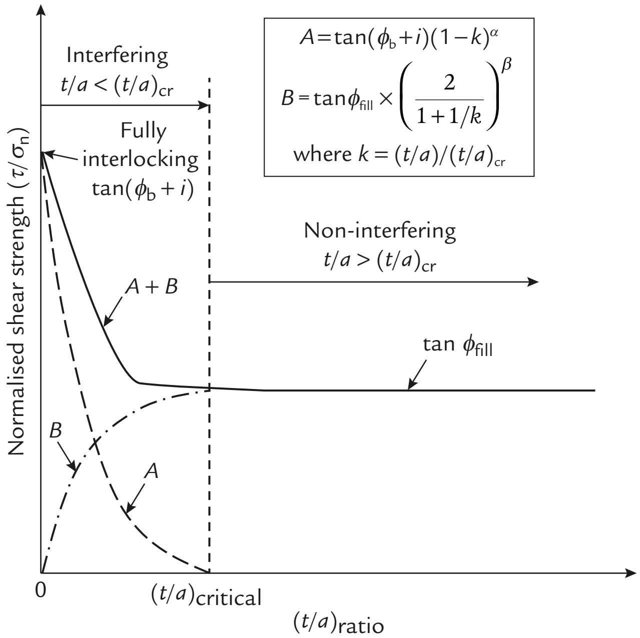

Figure 1 Conceptual normalized peak shear strength model for sediment-infilled discontinuities (modified from Indraratna et al., 2005).

function A becomes zero indicating the absence of rock to rock contact. At the critical infill thickness the function B reaches its optimum value and remains same for higher (t/a) ratios. Indraratna et al. (2005) introduced two distinct zones which are called interference zone $(t/a<(t/a)_{cr})$ and non-interference zone $(t/a>(t/a)_{cr})$.

For $t/a<(t/a)_{cr}$ – interference zone:

$$A=\tan(\phi_b+i)\times(1-k)^{\alpha} \tag{9}$$

$$B=\tan\phi_{fill}\times\left(\frac{2}{1+1/k}\right)^{\beta} \tag{10}$$

$$\frac{\tau_s}{\sigma_n}=A+B$$

$$=\tan(\phi_b+i)\times(1-k)^{\alpha}+\tan\phi_{fill}\times\left(\frac{2}{1+1/k}\right)^{\beta} \tag{11}$$

where, ϕ_b = basic friction angle of rock; i = initial asperity angle; ϕ_{fill} = friction angle of the soil infill; $k=(t/a)/(t/a)_{cr}$; and α and β are empirical constants defining the geometric locus of the functions A and B.

For $t/a>(t/a)_{cr}$ — non-inteference zone:

$$\frac{\tau_s}{\sigma_n}=\tan\phi_{fill} \tag{12}$$

In the above equations the cohesion of the infill is ignored. Because under saturated condition, normally consolidated clays shows very little cohesion.

This model was extended to take into account the effect of overconsolidated infill materials (Indraratna et al., 2008). An expression for the normalized shear strength of overconsolidated infilled joint $(\tau_p/\sigma_n)_{oc,n}$ was proposed as;

$$\left(\frac{\tau_p}{\sigma'_n}\right)_{oc,n}=P+Q=\tan(\phi_b+i_0)\times(1-k_{oc,n})^{a_n}$$
$$+\tan\phi_{fill}\times OCR^{\alpha}\times\left(\frac{2}{1+1/k_{oc,n}}\right)^{b_n} \tag{13}$$

where $(\tau_p/\sigma_n)_{oc,1}$ is the normalized shear strength of a normally consolidated infilled joint ($OCR=1$), OCR is the overconsolidation ratio; $(t/a)_{oc,n}$ is the t/a ratio of a given infilled joint with an OCR of n.

$$k_{oc,n}=[(t/a)_{oc,n}/(t/a)_{cr,n}]; \tag{14}$$

$(t/a)_{cr,n}$ = critical t/a ratio of an infilled joint with an OCR of n; α, a_n and b_n are empirical constants.

2.4 Shear Displacement Criterion for Soil-Infilled Rock Joints

This revised model describes the shear displacement behavior of a soil-infilled discontinuity capturing the effect of infill squeezing during the shearing process. The soil-infilled model developed by Indraratna et al. (2010) is described by:

$$\tau=\sigma_n\left\{(1-\eta)\left[\frac{\tan\phi_b+\tan i_d}{1-\tan\phi_b\tan i}\right]+\eta\tan\phi_r\right\} \tag{15a}$$

where,

$$\eta=\exp\left(\frac{-u_s\,JRC}{100c_1a(t/a)}\right) \tag{15b}$$

$$i_d=(i_0-i)\exp\left[-\frac{(u_s-u_{peak})^2JRC}{100(c_2a)^2}\right]+i \tag{15c}$$

$$i=\tan^{-1}(\partial u_n/\partial u_s) \tag{15d}$$

$$u_n=\frac{a_0}{2}+\sum_{n=1}^{N_h}L_f[a_n\cos(2\pi nu_s/T)+b_n\sin(2\pi nu_s/T)] \tag{15e}$$

$$u_{peak}=\frac{a}{\tan(i_0)}\sigma_{n0}^{-c_3} \tag{15f}$$

$$L_f=1 \text{ if } n\pi/N_h=0,\ =(N_h/n\pi)\sin(n\pi/N_h) \text{ otherwise} \tag{15g}$$

where τ is the shear stress; σ_n is the normal stress; σ_{n0} is the initial normal stress; ϕ_b is the basic friction angle of the rock joint; ϕ_r is the residual friction angle; i is the dilation angle at a given shear displacement, u_s; i_0 is the initial asperity angle; u_n is the normal displacement; u_s is the accumulated shear displacement; u_{peak} is the shear displacement at peak stress ratio (t/σ_n); η is the squeezing factor; c_1 and c_2 are empirical constants which control the rate of infill squeezing and asperity degradation respectively; c_3 is a fitting constant; a is the asperity amplitude; a_0, a_n, and b_n are Fourier series coefficients, T is the

Fourier period; N_h is the number of harmonics; L_f is the Lanczos sigma factor; *JRC* is the joint roughness coefficient; and *t/a* is the infill thickness to asperity amplitude ratio. Empirical constant c_2 is found by best fit regression while c_1 can be calculated from:

$$c_1 = \frac{-u_{peak} JRC}{100a(t/a)\ln(A_1/A_2)} \tag{16a}$$

where,

$$A_1 = \tan\phi_{peak} - A_3$$
$$A_2 = \tan\phi_r - A_3 \tag{16b}$$
$$A_3 = \frac{\tan\phi_b + \tan i_0}{1 - \tan\phi_b \tan i}$$

This model incorporates three distinct zones in its shearing mechanism. The strength of the infill governs the behavior of the first zone. The role of the rock is to set the boundary limits for the soil failure surfaces, which are defined by the geometry or roughness of the joint. In the second phase, as the shearing proceeds, the infill present above the sliding surface has to squeeze out to fill the space generated on the unloaded side of the joint. After the infill is squeezed out asperities come into contact, and the shear behavior will then be governed by the strength of the rock (Fig. 2).

2.5 Computational Procedure

The proposed method for capturing dilation in the shear displacement model is by means of the Fourier series analysis. The Fourier predictions (Equation 15e)

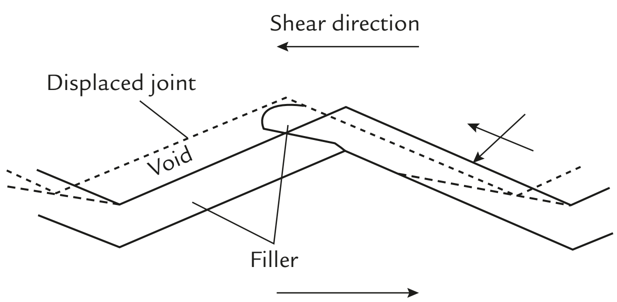

Figure 2 Mechanism of infill failure for small thickness (modified from de Toledo & de Freitas, 1993).

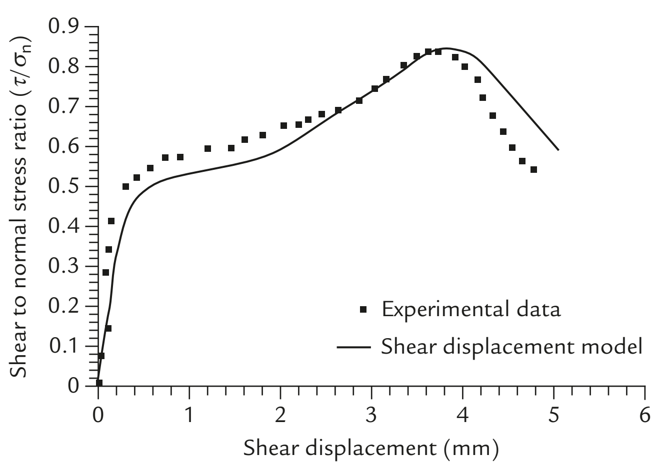

Figure 3 Shear displacement criterion for soil-infilled joint.

are then fitted to the normal displacements obtained experimentally. The corresponding Fourier constants a_n and b_n are determined by performing a conventional harmonic analysis using MATLAB. Either Microsoft Excel spreadsheets or MATALAB can be used to determine the shear stress corresponding to a given shear displacement, and then to plot this relationship (Fig. 3). The shear displacement equation, Equation (15a) can be differentiated to obtain the peak shear strength of the joint (Equations 17–18). This rigorous mathematical process has been conveniently programmed using MATHCAD.

3 COMPARISON BETWEEN THE SHEAR STRENGTH MODELS FOR INFILL JOINTS

The shear displacement model proposed by Indraratna et al. (2010) was differentiated with respect to shear displacement to obtain shear displacement at peak shear stress, thus

$$\frac{\partial \tau}{\partial u_s} = 0 \tag{17}$$

$$\frac{\partial\left(\sigma_n\left\{(1-\eta)\left[\frac{\tan\phi_b + \tan i_d}{1-\tan\phi_b \tan i}\right] + \eta\tan\phi_r\right\}\right)}{\partial u_s} = 0 \tag{18}$$

The peak shear strength is then found by substituting u_s into Equation (15). The peak shear strength data obtained by this method was compared with the model predictions of Indraratna et al. (2005), Papaliangas et al. (1990) model and experimental data (Fig. 4). The experimental data was obtained from CNS direct shear testing for idealized infilled joints with clayey sand infill.

The peak shear strength model proposed by Indraratna et al. (2005) slightly overestimates the shear strength for infill thicknesses less than its critical value whereas the model proposed by Indraratna

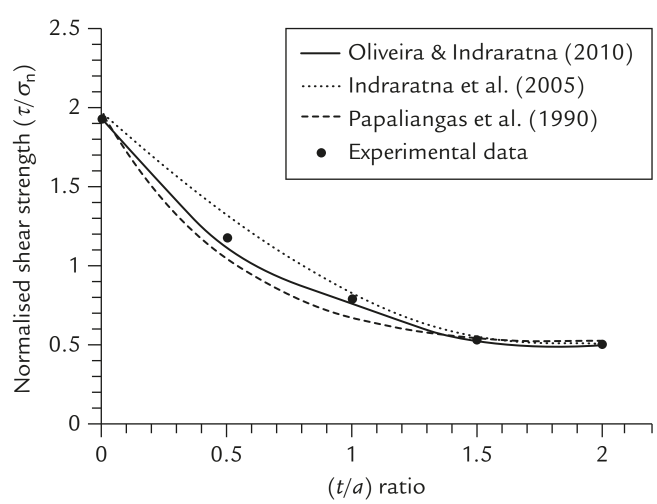

Figure 4 Comparison of model predictions with experimental data for clayey sand-infilled idealized saw-tooth joints.

et al. (2010) yield a better agreement with the experimental data overall. Papaliangas et al. (1990) model underestimates its predictions over the experimental data. All three models coincide with experimental data for relatively thick infills.

Figure 5 presents a study based on the experimental data obtained for bentonite-infilled idealized saw tooth joints and Figure 6 shows data for natural infilled joints filled with pulverized fuel ash (PFA). Models were compared over different types of infill materials in saw-tooth and natural joint profiles. The parameters used for predicting Indraratna et al. (2010) model is presented in Table 3.

In Papaliangas et al. (1990) model they have only used a constant which is defined as the (t/a) ratio at which the minimum shear strength is reached. Using this constant and a fitting parameter they have predicted the peak shear strength of the joint. For thicknesses greater than its critical value, they have simply used the minimum shear strength of the system. The empirical parameters used for calculations are presented in Table 1. Indraratna et al.

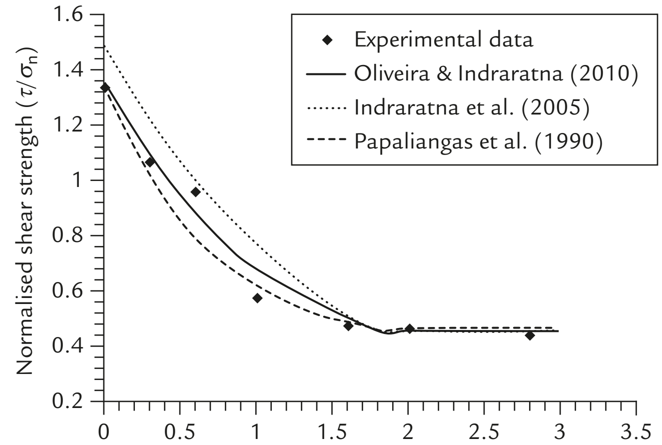

Figure 5 Comparison of model predictions with experimental data for bentonite-infilled idealized saw-tooth joints.

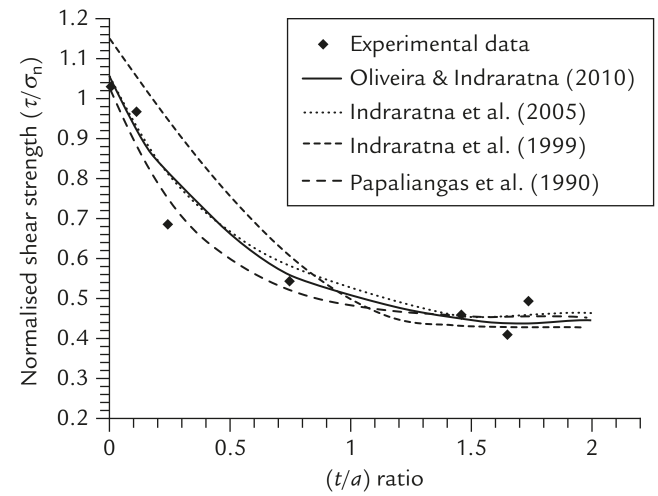

Figure 6 Comparison of model predictions with experimental data for natural joints filled with Pulverized fuel ash (PFA).

(1999) model also uses two empirical constants to predict the shear drop due to the infill. This model has its own method to predict the peak shear strength of a clean joint. They calculate the peak shear strength of the joint using the peak shear strength of the clean joint and normalized shear drop.

The conceptual model developed by Indraratna et al. (2005) also uses two empirical constants which vary with the joint type and infill material used. The shear strength of the joint is calculated by taking the strength of rock and infill separately and adding together. This model was able to capture the infill friction and predicts the shear strength of the infill material. In all these three models, shear strength of the filled joint converge to shear strength of the infill alone for t/a ratios greater than its critical value. The parameters used for the model calculations are presented in Table 2.

Indraratna et al. (2010) was able to capture more parameters over the other models discussed. In their model they have considered the effect of infill squeezing, joint roughness and variation of the dilation angle with the shearing. Since the model predictions were more accurate than the other models presented here. In the non-interfering zone the sliding surface does not touch the rock asperities.

Table 1 Empirical constants for Papaliangas et al. (1990) model

Infill type	c	m
Clayey sand	1.6	0.6
PFA	1.5	1
Bentonite	1.8	0.7

Table 2 Empirical constants for Indraratna et al. (2005) model

Joint type	$(t/a)_{cr}$	α	β
Clayey sand	1.6	1.1	4.4
PFA	1.5	1.9	1
Bentonite	1.8	1.1	3.1

Table 3 Soil-infilled joint parameters for Indraratna et al. (2010) model

Infill type	σ_n kPa	Φ_b	Φ_{fill}	(t/a)	c_1	c_2
Clayey sand	800	35.5°	27.5°	0.5	1.7	0.01
Clayey sand	800	35.5°	27.5°	1.0	3.0	0.01
Clayey sand	800	35.5°	27.5°	1.5	NA	NA
Clayey sand	800	35.5°	27.5°	2.0	NA	NA
Bentonite	300	37.5°	24.5°	0.3	3	0.05
Bentonite	300	37.5°	24.5°	0.6	4	0.10
Bentonite	300	37.5°	24.5°	1.0	7	0.15
PFA	75	31°	24°	0	10^{-5}	1.0
PFA	75	31°	24°	0.11	0.3	2.0
PFA	75	31°	24°	0.24	1.0	1.0
PFA	75	31°	24°	0.74	1.0	2.0
PFA	75	31°	24°	1.64	2	5

Therefore, the squeezing factor η is expected to reach zero when its empirical constants tend to high values. Also, no dilation is expected once rock-to-rock contact does not take place. There for this model also converge to the same equation as Indraratna et al. (2005) for non-interfering zone.

4 CONCLUSIONS

The peak shear stress models and the subsequent shear displacement model presented in this paper highlight the role of infill thickness in reducing the shear strength from the maximum value associated with the clean rough joints. The modified shear-displacement model better represents the peak shear behavior of clean, rough joints and presents a more practical approach capturing the role of the degradation factor and the infill squeezing factor. The proposed infill squeezing factor better demonstrates the influence of roughness and the t/a ratio on the shearing mechanism. The models describes by Indraratna et al. (1999, 2005) and Papaliangas et al. (1990) show some discrepancies of the predicted peak shear strength values for interfering zone, where $t/a < (t/a)_{cr}$. This is due to the lack of consideration of shear strength parameters in their models. All the models predict accurately, the peak shear strength data for non-interfering zone where $t/a > (t/a)_{cr}$.

Each model presented in this paper requires the determination of empirical constants for a specific infill and joint type before modeling shear behavior. Therefore a series of laboratory tests has to be performed for better prediction of the data. The scale effects were not considered in any model. The single value for t/a ratio assumed for the joint is also a simplification and might not fully represent natural infilled joints where the infill thickness varies considerably along its length. The models should be tested and validated further for natural infill joint profiles with various infill materials. Once the necessary coefficients have been evaluated in the laboratory, the shear-displacement model presented here offers a promising option in the stability analysis of jointed slopes and underground excavations.

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