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Shear strength model for sediment-infilled rock discontinuities and field applications

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ABSTRACT

Discontinuities have a great influence on the reduction of rock mass shear strength depending on the geometry, roughness and the nature of infill sediments. These discontinuities are generally filled with different types of sediments. When the infill sediments are saturated, they make a considerable impact on the overall strength of the rock mass. Most infilled discontinuities in the field are over-consolidated due to various factors such as climatic and weathering conditions, as well as hydrothermal alteration. The increase in the thickness of joint infill subdues the otherwise prominent role of joint roughness. This paper presents some of the existing mathematical models for predicting the shear strength of sediment-infilled rock joints and modification to the shear strength model for overconsolidated infill joints considering energy balance principles to better predict the clean joint strength. A simplified approach for using this model in practice is presented through a hypothetical example of slope stability, and use of the model to predict safety factor of a slope under different overconsolidation ratios of infill is discussed. An analysis to stabilise a potentially unstable rock slope using pre-tensioned fully grouted bolts is also presented
Keywords: clays, shear strength, consolidation, rock joints

1 INTRODUCTION

The discontinuities present in a rock mass are usually filled with soft material such as clay and silt. These infill materials are in general under overconsolidated state rather than normally consolidated due to stress relaxation or unloading, which is common in nature. For example, periodic erosion and landslides induce stress relief of underlying joint planes making the infilled clayey sediments overconsolidated. A typical example of this condition can be observed in Kangaroo Valley, NSW, Australia, where much of the groundwater transported infill is kaolinite rich clayey sediments (Indraratna et al. 1999).

There are a considerable amount of studies in the literature on clean and infilled joints conducted under constant normal load (CNL) (Barton 1974, Phien-wej et al. 1990, de Toledo and de Freitas 1993) and constant normal stiffness (CNS) conditions (Ohnishi and Dharmaratne 1990, Indraratna et al. 1999, 2005, 2008). However, only limited studies have been carried out on the effect of overconsolidation (Barton 1974, de Toledo and de Freitas 1993, Indraratna et al. 2008).

The presence of infill material inside a discontinuity will drastically reduce its shear strength and can be considered as the most pronounced effect on the shear strength of an infilled joint. It has been common practice to assume that the shear strength of an infilled joint is that of the infill material alone. This assumption can often lead to underestimation or overestimation of the joint shear strength. From the previous studies, it can be noted that the shear strength of an infilled joint will vary from its clean joint strength to infill strength with an increase in infill thickness. After a certain thickness is reached, shearing only takes place through the infill material as there is no rock-to-rock contact; hence the shear strength is only governed by infill alone (Indraratna et al. 2005). This paper describes a semi-empirical model proposed to predict the peak shear strength of an infilled joint considering the overconsolidation ratio of the infill. The original model proposed by Indraratna et al. 2008 was modified to include the energy balance principles. A slope stability analysis was carried out on a hypothetical infilled rock slope to investigate the applicability of the shear strength model presented.

2 THEORETICAL BACKGROUND

Among all the parameters of constituent materials, the infill thickness can be considered the most important parameter controlling the shear strength of an infilled joint. Indraratna et al. (2005) proposed a conceptual model to predict the normalised peak shear strength of an infilled joint. This model predicts the strength contributions from the rock interfaces and the soil infill by two different algebraic functions for varying infill thickness to asperity height ratio (Figure 1). Function *A* predicts the contribution of the strength from the rock interfaces with the thickness (*t*) to asperity height (*a*) ratio. It starts from the clean joint strength (i.e. *t/a* = 0) and reaches zero when the infill thickness reaches its critical value $(t/a)_{cr}$. Function *B* predicts the contribution from the soil infill. When *t/a* = 0, there is no infill present in the joint hence the strength contribution from soil equals to zero. It will reach its maximum value when the infill thickness reaches a critical value indicating shearing only through infill.

$$\frac{\tau_p}{\sigma_n} = A + B = \tan(\phi_b + i_0) \times (1-k)^{\alpha} + \tan(\phi_{fill}) \times \left\{\frac{2}{1+1/k}\right\}^{\beta} \tag{1}$$

where, τ_p is the peak shear stress of the joint; σ_n is the normal stress; ϕ_b is the basic friction angle of the joint; i_0 is the initial asperity angle; ϕ_{fill} is the friction angle of the normally consolidated/remoulded infill material, *k* is the ratio of (t/a) to $(t/a)_{cr}$; α and β are empirical constants defining the geometric loci of the function *A* and *B*.

Indraratna et al. (2008) extended this model to include the effect of infill overconsolidation ratio. From a series of comprehensive laboratory studies it has been noted that the overconsolidation ratio (*OCR*) of the infill only affect the strength contribution from the infill material. Therefore the function *B* is adjusted to incorporate the effect of *OCR* to better describe the strength contribution from the infill material. Modified model is presented in Figure 2 and can be analytically presented as:

$$\left(\frac{\tau_p}{\alpha_n}\right)_{oc,n} = A_n + B_n = \tan(\phi_b + i_0) \times \left(1 - k_{oc,n}\right)^{\alpha_n} + \tan\left(\phi_{fill}\right) \times OCR^{a} \times \left\{\frac{2}{1+1/k_{oc,n}}\right\}^{\beta_n} \tag{2}$$

where, $k_{oc,n}$ is the ratio of $(t/a)_{oc,n}$ to $(t/a)_{cr,n}$; $(t/a)_{oc,n}$ is the given value of *t/a* for an infilled joint with *OCR* of *n*; $(t/a)_{cr,n}$ is the critical *t/a* ratio of an infilled joint with *OCR* of *n*; α_n and β_n are empirical constants defining the geometric loci of the function A_n and B_n. a is an empirical constant used to describe the *OCR* effect in terms of the normally consolidated infill.

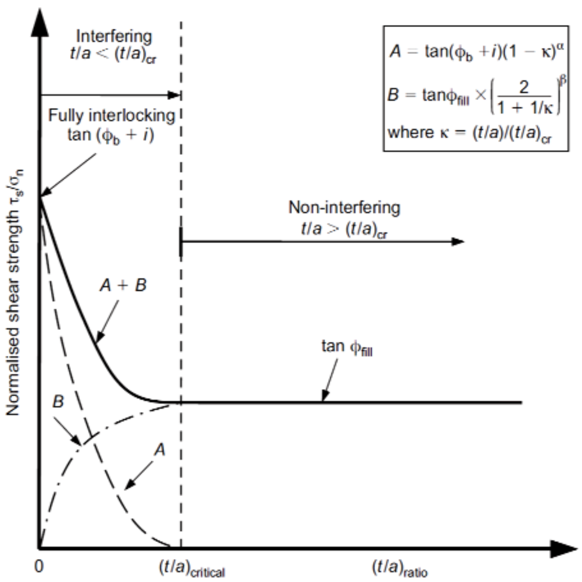

Figure 1. Conceptual normalised peak shear strength model for soil infilled joints (modified from Indraratna et al. 2005)

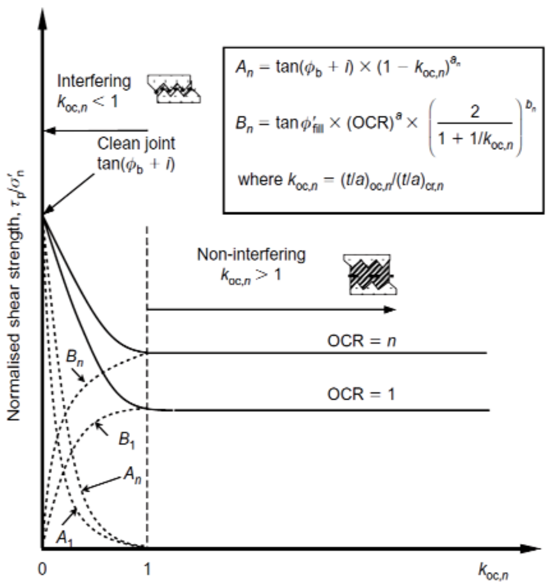

Figure 2. Shear strength model for overconsolidated infilled idealised joints (modified from Indraratna et al. 2008)

3 MODIFIED PEAK SHEAR STRENGTH MODEL

The model proposed by Indraratna et al. (2008), to predict the peak shear strength for infilled joints, was modified using the energy balance principles proposed by Siedel and Haberfield (1995). The function A_n of the original model is predicted using the Patton (1996) model for clean joints in the range of non-breakage of asperities. As a result, the model does not correctly predict the shear strength of the clean joint. Therefore, function A_n was modified using the energy balance principles where initial asperity angle (i_0) was replaced by the dilation angle at peak shear stress of the clean joint $(i_{\tau p})_{clean}$ (Oliveira et al. 2009) The shear strength of the clean joint (τ_{clean}) is then proposed as:

$$\tau_{clean} = \sigma_n \left\{ \frac{tan(\phi_b)+tan(i_0)}{1-tan(\phi_b)*tan(i_{\tau p})_{clean}} \right\} \tag{3}$$

Function *A* is modified as:

$$P = \left\{ \frac{tan(\phi_b)+tan(i_0)}{1-tan(\phi_b)*tan(i_{\tau p})_{clean}} \right\} \left(1-k_{oc,n}\right)^{\alpha'} \tag{4}$$

The Function *B* remains the same as proposed by the original model predicting the strength contribution from the infill:

$$Q = tan(\phi_{fill}) * OCR^a * \left(\frac{2}{1+1/k_{oc,n}}\right)^{\beta'} \tag{5}$$

$$\frac{\tau_p}{\sigma_n} = P + Q \tag{6}$$

When (*t/a*) = 0, the function *Q* vanishes and the model reverts to peak shear stress of a clean joint. When (*t/a*) reaches its critical limit $(t/a)_{cr}$, function *P* vanishes and hence the peak shear stress of the joint is only given by the function *Q* (infill alone). The model can be proposed mainly for two regions as proposed by Indraratna et al. (2008), for interference zone where $t/a < (t/a)_{cr}$ and for non-interference zone where $t/a > (t/a)_{cr}$. For the interference zone:

$$\frac{\tau_p}{\sigma_n} = \left\{ \frac{tan(\phi_b)+tan(i_0)}{1-tan(\phi_b)*tan(i_{\tau p})_{clean}} \right\} \left(1-k_{oc,n}\right)^{\alpha'} + tan(\phi_{fill}) * OCR^a * \left(\frac{2}{1+1/k_{oc,n}}\right)^{\beta'} \tag{7}$$

For the non-interference zone:

$$\frac{\tau_p}{\sigma_n} = tan(\phi_{fill}) * OCR^a * \left(\frac{2}{1+1/k_{oc,n}}\right)^{\beta'} \tag{8}$$

The empirical parameters of the overconsolidated peak shear strength model for a silty clay infill are presented in Table 1. Model properties used for the calculation are ϕ_b = 37°, ϕ_{fill} = 23°, i_0 = 18° and a = 0.24.

Table 1: Empirical constants and critical t/a ratios for different OCRs

OCR Value	$(t/a)_{cr}$	α'	β'
1	1.9	1.9	1.9
2	1.7	1.7	2.0
4	1.5	1.4	2.4
8	1.3	1	3.6

The model described earlier neglects the strength contribution from the cohesion of the infill material (Oliveira et al. 2009). Indraratna et al. (2005) suggested that for cohesive infill materials the term c'_{fill}/σ_n should be included in the model. Therefore, the proposed model can be rewritten as:

$$\frac{\tau_p - c_{fill}}{\sigma_n} = P + Q \tag{9}$$

$$\frac{\tau_p - c_{fill}}{\sigma_n} = \left\{ \frac{tan(\phi_b)+tan(i_0)}{1-tan(\phi_b)*tan(i_{\tau p})_{clean}} \right\} \left(1-k_{oc,n}\right)^{\alpha'} + tan(\phi_{fill}) * OCR^a * \left(\frac{2}{1+\frac{1}{k_{oc,n}}}\right)^{\beta'} \tag{10}$$

4 SLOPE STABILITY ANALYSYS USING THE NORMALISED PEAK SHEAR STRENGTH MODEL

The use of the proposed normalised shear strength model in a practical situation is illustrated using a simplified hypothetical slope stabilisation problem as presented in Figure 3. Indraratna and Haque (2000) presented a similar example and application of their model to a potential unstable wedge failure at Kangaroo Valley in NSW, Australia for a clean joint. In the current hypothetical slope stabilisation problem, the rock wedge has a slope angle of λ and it contains a soil-infilled joint at a dip angle of θ.

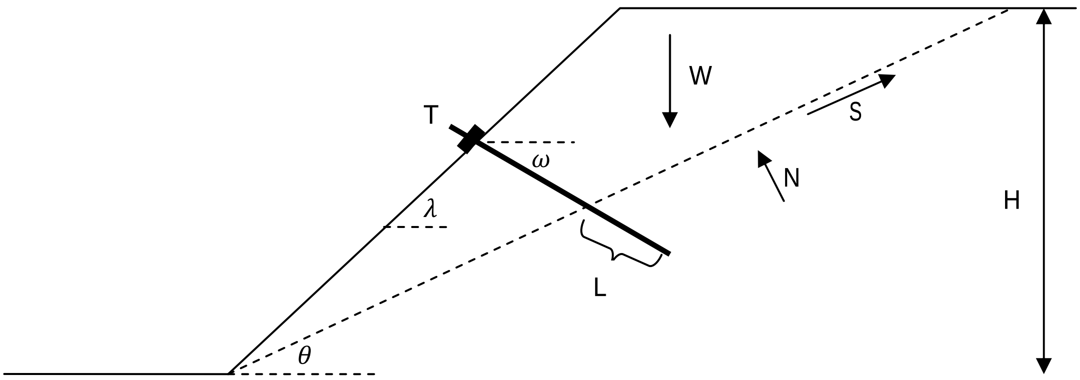

Figure 3. Slope supported by pre-tensioned grouted bolt (modified from Indraratna and Haque 2000)

4.1 Limit Equilibrium Analysis (Initial condition)

A limit equilibrium analysis before installing the rock bolts is presented in this section. The factor of safety of the slope can be calculated as the ratio between the Resisting Force *(RF)* and Disturbing Force *(DF)* where the resisting force would be the shear force *(S)* of the joint. Disturbing force can be calculated by resolving the forces:

$$DF = W\ sin\theta \tag{11}$$

$$FS = \frac{S}{W\ sin\theta} \tag{12}$$

The weight of the wedge *(W)* can be calculated as:

$$W = 0.5\,\gamma\,H^2(\cot\theta - \cot\lambda) \tag{13}$$

Using the proposed normalised shear stress model, the shear force can be calculated after determining all the model parameters and empirical constants. The shear force is given by:

$$S = N\left(\frac{\tau_p}{\sigma_n}\right) \tag{14}$$

where, *N* is the normal force applied to the joint, which is given by the component of the weight of the wedge perpendicular to the joint plane, γ is the unit weight of the intact rock and *H* is the height of the slope.

$$N = W\cos\theta \tag{15}$$

The idealised rock joint presented in this study is assumed as representative to the rockslide with a *t/a* = 0.9. Limit equilibrium analysis was carried out for the infilled joint with different overconsolidation ratios of the infill. As an illustrative example, the geometry of the rock wedge is defined by a height H = 30.5 m, λ = 80°, θ = 30° and γ = 27.5 kN/m^3. The factor of safety of the joint was calculated for different *OCR*'s by substituting the above values in Equations 11 to 15 and the results are tabulated in Table 2.

Table 2: Factor of safety of the slope for different OCRS

OCR	*FS*
1	1.048
2	1.103
4	1.200
8	1.349

4.2 Stabilisation using rock bolts

When the *OCR* of the infill is low, the factor of safety indicates possible instability of the jointed slope. Rock bolting can be used as a method to improve the overall stability of the joint. Pre-tensioned fully grouted bolts would work effectively on a rough rock joints, as the joint will dilate during the shearing. The bolts will generate an increased tensile force because of the dilation depending on the bolt-grout stiffness (Oliveira et al. 2009). If the bolts are drilled at an angle of ω with respect to the horizontal plane, the new normal load can be calculated as:

$$N' = W\cos\theta + \frac{n}{s_h}T\sin(\theta + \omega) \quad (16)$$

where, n is the number of bolts, s_h is the horizontal bolting spacing and T is the tension provided by the bolts. It is assumed that all the bolts contribute to an equal load and uniform stress variation is present along the bolt. The tension force from the grouted bolts is given by:

$$T = \frac{E_b A_b}{L_b}\delta_v + T_p \quad (17)$$

where, E_b is the modulus of elasticity of the bolt, T_p is the pretension of a single bolt, A_b is the area of the bolt, L_b is the effective grouted bolt length and δ_v is the dilation. If the stiffness of the grouted bolt annulus is neglected, the modulus of elasticity E_b and the area A_b are predominantly those of the steel. Considering the limit equilibrium analysis final factor of safety can be calculated as:

$$FS = \frac{S + \frac{n}{s_h}T\cos(\theta + \omega)}{W\sin\theta} \quad (18)$$

n number of rock bolts, which were each pre-tensioned at 20 KN, was drilled through the rock wedge at an angle of 30° (shown in Figure 3) to the horizontal plane with effective bolt length L_b = 1.0 m, s_h = 1.0 m and bolt diameter of 25 mm with the Young's Modulus of steel E = 200 GPa. The joint is assumed to dilate 5 mm normal to the joint plane and the minimum guidelines for a safety factor is assumed as 2.0. Number of rock bolts required to stabilise the slope were calculated using the proposed normalised peak shear strength model for different overconsolidation ratios and presented in Table 3.

Table 3: New safety factors for the stabilised the slope with different OCRs

OCR	*Number of bolts required (n)*
1	18
2	16
4	14
8	10

It can be noted that the factor of safety of the slope increases with increasing overconsolidation ratio of the infill, indicating higher overconsolidated infilled joints produce a larger resisting force. If the overconsolidation ratio is neglected in the stability analysis, a lower value of safety factor will be predicted. If we consider a normally consolidated and an overconsolidated infilled joint (*OCR* = 8), the number of bolts required to stabilise the slope can be reduced by almost 50%. Therefore, If the *OCR* effect is not considered in the analysis, the number of rock bolts required to stabilise an overconsolidated joint will be overestimated. In this example, if the same number of rock bolts of a normally consolidated joint is used for the analysis of *OCR* = 8, a safety factor of 2.438 is predicted.

In any practical application, if the guidelines are provided for a minimum factor of safety, the number of rock bolts required to stabilise the joint can calculated using the above method. If the overconsolidated ratio of the joint is properly evaluated and included in the calculation, a lesser number of rock bolts will be required for the stabilisation compared to the normally consolidated state. Such kind of analysis will lead to cost effective stabilisation using rock bolts.

5 CONCLUSION

The infilled joint model proposed to predict peak shear strength of an overconsolidated infill joint by Indraratna et al. 2008 was modified in this study by considering energy balance principles to better predict the clean joint strength. While the original model was only intended to capture cohesionless soils, a modification is also proposed here for cohesive soils extending its applicability to various conditions. Infill cohesion can be included in the model by subtracting the normalised peak shear stress (τ_p/σ_n) by a normalised cohesion (C_{fill}/σ_n). The proposed normalised shear strength model is used in a limit equilibrium analysis for a hypothetical example of jointed slope. The safety factor for the slope failure was calculated for the initial conditions (without rock bolts) under different overconsolidation ratios. The safety factors range between 1.048 - 1.349 for *OCR*'s varying from 1 to 8, indicating possible failure of the slope. To increase the stability of the jointed slope, pre-tensioned fully grouted bolts were introduced as they work effectively if the discontinuity plane dilates during the shear movement. The number of rock bolts required to stabilise the slope was calculated for a minimum safety factor of 2 under a joint dilation of 5 mm. It was assessed that the number of bolts required to stabilise the slope was reduced from 18 to 10 when an overconsolidated (*OCR* = 8) infilled joint is considered instead of a normally consolidated infilled joint. In a typical field application, given the joint dilation and the guidelines for minimum factor of safety, the number of rock bolts required to achieve the given minimum safety factor can be calculated if the model parameters are properly evaluated. It can be clearly understood that if the *OCR* of the infill is neglected, joint strength and the stability for a jointed slope will be under estimated,

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