Numerical Prediction of Plastic Behavior of Highly Fractured and Weathered Phyllite Subjected to Pressuremeter Testing

Une prédiction numérique du comportement plastique de la géologie très fracturée et altérée soumise au test du pressomètre

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ABSTRACT: Pressuremeter test (PMT) is an in-situ test which serves well as an alternative to characterize highly fractured rock masses. However, complications arise as most of the pressuremeter probes are designed for tests in soils whose strength and stiffness are much lower than rocks. As a result, the pressuremeter probe used cannot apply sufficient pressure to cause significant plastic deformation, which leads to the lack of data points on the plastic behavior of the highly fractured rocks. Coupled with the limited understanding in the stress-strain response of highly fractured rocks subjected to the test, these challenges provide the research motivation to develop a reliable characterization method to generate useful parameters that can accurately describe the physical behavior of highly fractured rocks. This paper presents a numerical approach to predict the plastic deformation of pressuremeter tests in highly fractured rocks through the use of numerical modelling. The results generated from the developed method show good potential to characterize the unload-reload modulus, E_{ur}, effective cohesion, c' and friction angle, φ' of the highly fractured rocks that cannot be directly obtained from the measured PMT results from an established case study.

RÉSUMÉ : Le test du pressomètre (PMT) est un test in situ qui sert d'alternative à la caractérisation des masses rocheuses très fracturées. Cependant, des complications surviennent, car la plupart des sondes pressoir sont conçues pour des essais dans des sols dont la résistance et la rigidité sont beaucoup plus faibles que les roches. Il en résulte que la sonde de pression utilisée ne peut pas appliquer une pression suffisante pour provoquer une déformation plastique importante, ce qui conduit à l'absence de points de données sur le comportement plastique des roches fortement fracturées. En plus de la compréhension limitée de la réponse stress-déformation des roches fortement fracturées soumises au test, ces défis fournissent la motivation de la recherche pour développer une méthode de caractérisation fiable pour générer des paramètres utiles qui peuvent décrire avec précision le comportement physique des roches fortement fracturées. Cet article présente une approche numérique pour prédire la déformation plastique des essais de pressiomètres dans les roches fortement fracturées à l 'aide de la modélisation numérique. Les résultats obtenus à partir de la méthode développée montrent un bon potentiel pour caractériser le module de déchargement-rechargement, E_{ur}, cohésion effective, c' et angle de frottement, φ' des roches fortement fracturées qui ne peuvent pas être directement obtenues à partir des résultats PMT mesurés d'un cas établi étude.

KEYWORDS: Pressuremeter, Hardening Soil model, Fractured Rocks, Numerical Model

1 INTRODUCTION

The geology in Kuching, Sarawak consists of various metamorphic and sedimentary rocks which are highly fractured and weathered (Tan, 1993). Due to this highly fractured nature, characterizing the physical properties of the rock became a major challenge since there were very limited rock core lengths that were suitable for laboratory tests. This was reflected by the Rock Quality Designation (RQD) values, which were mostly zero. Therefore, pressuremeter test (PMT) was recommended as an alternative approach to characterize the highly fractured rocks since the test is performed directly in-situ, eliminating the need for rock core extractions.

Historically, PMTs are more commonly performed in soils. Assuming that ideal PMT results are obtained, several parameters can be interpreted directly from the test, such as pressuremeter modulus, E_p (Briaud, 1992), limit pressure, p_L and undrained shear strength, c_u (Marsland and Randolph, 1977), and friction angle, φ' for purely frictional soils (Hughes et al., 1977). It is also common and highly encouraged to perform unload-reload cycle in a PMT (Federal Highway Administration, 2002; ASTM, 2000; BS5930, 1999) in order to derive the unload-reload modulus, E_{ur}, which is commonly useful for settlement prediction (Kahle, 1983; Konstantinidis et al., 1986). However, there was no detailed specification on the procedure to perform the unload-reload cycle in most standards for PMT (BS5930, 1999; ISO, 2012). Federal Highway Administration (2002) recommends that the unload-reload cycle should be performed when the tested geomaterial has experienced significant plastic deformation. However, the unload-reload cycles should be performed with caution. ASTM D4719 (2000) stressed that the performance of unload-reload cycles requires the ability of an experienced operator to precisely control the rate of decrease in pressure, which would allow for accurate measurements in the decrease of volume.

In Sarawak, PMT is still considered to be a relatively new technology, hence the experience of performing the test is still limited in this region. Furthermore, the PMTs in this study were conducted in highly fractured rocks. Since PMTs are mostly intended for use in soils, the allowable working pressure that the pressuremeter probe could apply was insufficient to deform the highly fractured rocks and achieve plasticity. This led to non-ideal PMT results which lacked data on plastic deformation, thus restricting the ability to interpret c_u. Furthermore, the insufficient plastic deformation prohibits the performance of the unload-reload cycle. This created challenges in deriving E_{ur} of the tested rock mass from the PMT results, which, according to Haberfield and Johnston (1986) and Gannon et al. (2004) is the intact modulus of the rock.

Therefore, this paper aims to present an alternative approach which would allow for derivation of the unload-reload modulus, E_{ur} of highly fractured rocks from non-ideal PMT results via numerical modelling. The numerical models of PMT were constructed using the commercial finite-element package, PLAXIS 3D, and the stress-strain behavior of the highly fractured rocks were modelled using the Hardening Soil Model (HSM). Effective MC strength parameters in the form of cohesion, c' and friction angle, φ' were also able to be derived as they are used to define the strength of the HSM. This method still relies on the usage of common interpretations of PMT results as specified in ASTM D4719 (2000). Therefore, a general understanding on the typical PMT result is still required, and is provided hereinafter.

2 PMT RESULTS INTERPRETATION

Menard pressuremeter was used to perform all PMTs presented in this study. It is essentially a pre-bored pressuremeter which requires a pre-excavated test borehole to provide access for the positioning of the probe at the required test depth. An ideal Menard PMT result is presented in Figure 1.

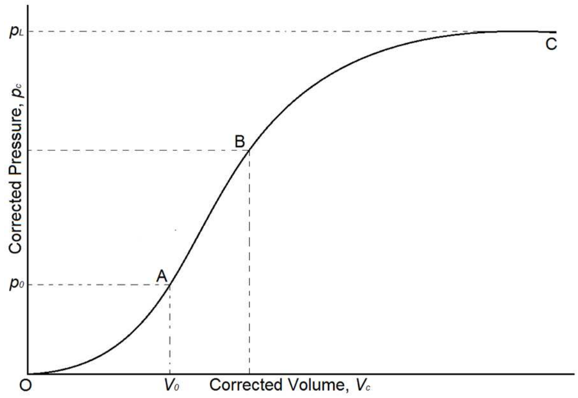

Figure 1 Typical stress-strain curve obtained from a PMT.

The stress-strain behavior shown in the PMT curve can be divided into 3 stages; Initiation Stage (OA), Pseudo-elastic Stage (AB) and Plastic Stage (BC). Initiation Stage represents the effect of stress-relief which is experienced by the tested geology due to the excavation of the test borehole. As the pressuremeter probe applies radial pressure to the tested geology, the horizontal pressure of the tested geology was restored to the in-situ horizontal stress condition, p_0, which was shown in point A. It is only from this point that the applied pressure would induce true stress-strain behavior of the tested geology, as represented in the Pseudo-elastic Stage and the Plastic Stage. As a result, these two stages were used by various authors to derive geological strength and stiffness parameters of the tested geologies (Briaud et al., 1983; Clarke, 1994; Palmer, 1972; Vesic, 1972).

2.1 Pressuremeter Modulus (E_p)

One of the most common parameters interpreted directly from a pressuremeter test is the pressuremeter modulus, E_p. The derivation of E_p in accordance to ASTM D4719 (2000) is shown in Eq. 1.

$$E_p = 2(1+v)(V_0+V_m)\frac{\Delta P}{\Delta V} \quad (1)$$

where v is Poisson's ratio, V_0 is the uninflated probe volume, V_m is the median corrected volume reading at the Pseudo-elastic stage, and ΔP and ΔV are the changes in pressure and volume between point AB (Figure 1) respectively. However, it should be noted that the value of E_p does not equate to, and is often lower than, the Young's modulus, E' due to the disturbance caused by the pre-excavation of test borehole. Menard and Rousseau (1962) proposed a correction factor, K which allows a direct correlation between E_p and E' (Eq. 2)

$$E' = KE_p \quad (2)$$

Briaud (1992) proposed several values of K, ranging from 1 to 3 for various types of soils and rocks. In the case of PMTs in highly fractured rocks, the value of K is 3, which leads to the expression

$$E' = 3E_p \quad (3)$$

3 NUMERICAL MODEL OF PMT

Numerical models of PMT were generated using 10-node tetrahedral elements in the finite element modelling package PLAXIS 3D. The hyperbolic constitutive soil model, the Hardening Soil Model (HSM) was selected to represent the non-linear stress-strain behavior of the highly fractured phyllite.

3.1 The Hardening Soil Model

The selection of HSM was motivated by its ability to incorporate stress-dependent stiffness (Schanz et al., 1999) which allows for modelling of non-linear stress-strain behavior that is usually shown in typical PMT results. Furthermore, the HSM defines its strength in terms of cohesion, c' and friction angle, φ', which are the effective Mohr-Coulomb (MC) strength parameters (Plaxis, 2013). This would potentially fit the purpose of using PMT to characterize highly fractured geology for assessments of various geotechnical works which still use MC strength parameters.

Another feature of the HSM is the utilization of three stiffness moduli in the definition of its stiffness behavior. The three stiffness moduli are E_{50}^{ref}, E_{oed}^{ref}, and E_{ur}^{ref}, which refers to the secant modulus at 50% shear strength, the oedometer modulus, and the unload-reload modulus, respectively. By default, PLAXIS 3D sets a ratio of E_{50}^{ref}: E_{oed}^{ref}:E_{ur}^{ref} to be 1:1:3. Further details regarding the HSM can be found in the material manual of PLAXIS 3D (Plaxis, 2013).

3.2 Model Generation

Numerical models of PMT were constructed by generating geological profiles based on the borehole logs of the test borehole. The horizontal boundary of the model was set to 1m in x- and y- directions, while the vertical boundary was set according to the deepest reading in the borehole logs. Figure 2 shows the model geometry which represents one of the PMT conducted.

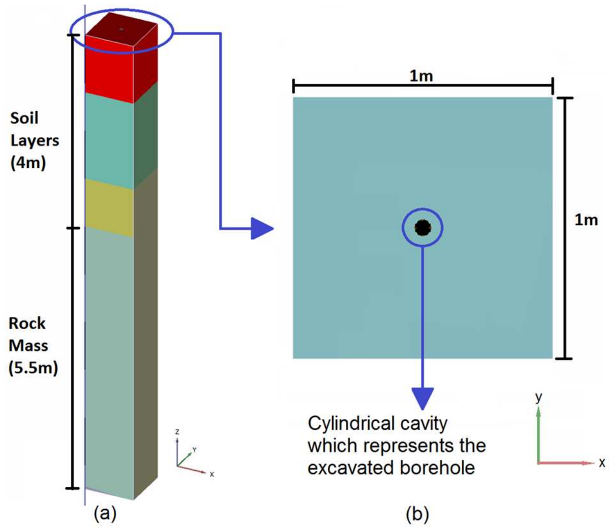

Figure 2 Numerical model geometry of PMT

The soil layers shown in Figure 2(a) were used to simulate overburden pressure. Hence, a simpler Mohr-Coulomb (MC) constitutive model was assigned to describe their properties. A cylindrical volume was defined at the horizontal center of the model down to the test depth according to the physical PMT in order to represent the test borehole. Afterwards, a portion of the cylindrical surface at the bottom of the modelled test borehole was applied with a radial pressure. The height of the cylindrical surface assigned with the radial pressure was identical to the height of the physical pressuremeter probe. This was done in order to simulate the pushing action of the inflated pressuremeter probe against the surrounding cavity walls.

Once the model geometry was generated, three construction stages were applied to the model in order to simulate the sequence of a PMT. The three construction stages include:

Stage 1: Generation of geostatic stresses
Stage 2: Excavation of test borehole
Stage 3: Application of radial pressure.

Stage 1 activates the geostatic stresses and default boundary conditions within the modelled geology which represented a greenfield state before PMT was performed. In Stage 2, the cylindrical volume at the center of the model was deactivated to represent the excavation of the test borehole. In Stage 3, the assigned radial pressure was activated to simulate the pushing action from the inflation of the probe. The magnitude of the radial pressure was assigned to be significantly higher than that of the physical PMT in order to instigate significant plastic deformation in the modelled highly fractured phyllite.

From the subsequent construction stages, numerical PMT results were generated and benchmarked against the measured PMT results. However, the numerical model of PMT did not consider the effect of clearance between the pressuremeter probe and the test borehole cavity. As a result, the Initiation Stage (see Figure 1) was barely visible in the numerical PMT results. This had some implications on the interpretation of the measured PMT results, which will be discussed hereinafter.

3.3 *Modification of the Measured PMT Results*

In order to replicate the measured PMT results in the numerical model, the subsequent Pseudo-elastic Stage and the Plastic Stage of the numerical PMT results have to be fitted against that of the measured PMT results. This was done by performing a slight modification in the measured PMT results to exclude the Initiation Stage of the measured PMT results.

Briaud (1992) proposed a method to exclude the Initiation Stage in the measured PMT results by taking the reference point of deformation at the state of in-situ stresses, p_0. Under this condition, the circumferential strain, ε_θ at the cavity wall is zero. By changing the deformation axis to ε_θ, the measured PMT results can be represented to start at p_0 from the origin, with the Initiation Stage being shown at ε_θ < 0%. The circumferential strain ε_θ, was presented as percentage and calculated using Eq. 4.

$$\varepsilon_\theta = \Delta R / R_0 \tag{4}$$

where ΔR is the change in cavity radius and R_0 is the cavity radius at p_0. The cavity radius was able to be derived directly from the volumetric reading as the inflation of the pressuremeter probe was kept from expanding vertically. As a result, the height of the inflated cylindrical cavity was constant throughout the test and the cavity radius was able to be calculated directly from the volumetric reading. The results from this modification of the measured PMT results are illustrated in Figure 3. The modified PMT results were then used as a benchmark to obtain suitable HSM parameters for the modelling of the highly fractured phyllite in the simulated PMT.

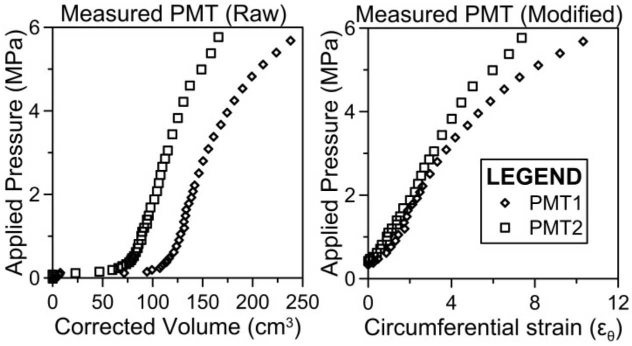

Figure 3 Comparison of the raw vs. modified PMT results.

3.4 *Parametric Study on the HSM Parameters*

Before determining the appropriate HSM parameters to produce the best-fit numerical PMT results against the modified PMT results, it is important to understand the role of the various moduli and strength parameters of the HSM, as well as the relationships between the parameters. This understanding of the effects of the various HSM parameters will allow for validation of the benchmarked numerical PMT results.

Since the HSM incorporates stress-dependent stiffness in its formulation, the strength parameters can affect the stiffness of the model through the stress-dependency factor, *N*, which is expressed as

$$N = \left(\frac{c'\cos\varphi' - \sigma'_3 \sin\varphi'}{c'\cos\varphi' + p^{ref} \sin\varphi'} \right)^m \tag{5}$$

where σ'_3 is the minor principle stress, p^{ref} is the reference pressure (100kPa by default), and *m* is the stress dependency power (Plaxis, 2013). As observed from Eq. 5, the friction angle, φ' can significantly affect the numerical PMT results. Therefore, the parametric study in this paper will focus on the relationship of the stiffness and strength parameters against φ'. This was done by producing a numerical PMT result which fits the measured PMT values, albeit being incomplete. Based on this benchmarked numerical PMT result, the parametric study was done by first varying φ', and then c'. The HSM stiffness moduli (E_{50}^{ref}, E_{oed}^{ref}, and E_{ur}^{ref}) were subsequently changed to assess their effects on φ'. This was important to validate the value of φ' used in the benchmarked numerical PMT results.

4 RESULTS AND DISCUSSION

The results on the parametric study will be discussed hereinafter. Figure 4 shows the response of c' towards the change of φ', when compared against the benchmarked numerical PMT results. The modified PMT result was also included for reference.

From Figure 4, it was shown that unlike c', φ' affects the Pseudo-elastic Stage significantly. As a result, the change of φ' could not be compensated by c' alone, which indicates that the determination of φ' is independent of c'. The role of c' however, is shown to be more significant for the Plastic stage (Figure 5), where various values of c' produced varying orientation of the Plastic stage while the Pseudo-elastic Stage remained relatively unchanged. Therefore, the value of c' can be determined from the Plastic Stage. However, the value of φ' requires further assessment of the Pseudo-elastic Stage, which involves E_{50}^{ref}, E_{oed}^{ref}, and E_{ur}^{ref}.

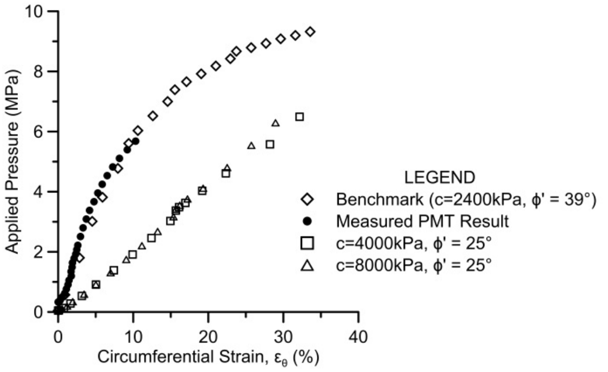

Figure 4 The effect of c' and φ' on the numerical PMT results.

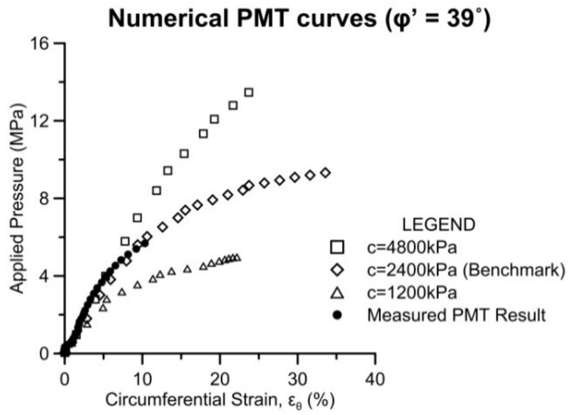

Figure 5 The role of c' in governing the Plastic Stage of the numerical PMT results.

The responses of E_{50}^{ref}, E_{oed}^{ref}, and E_{ur}^{ref}, towards the change of φ' are shown in Figure 6.

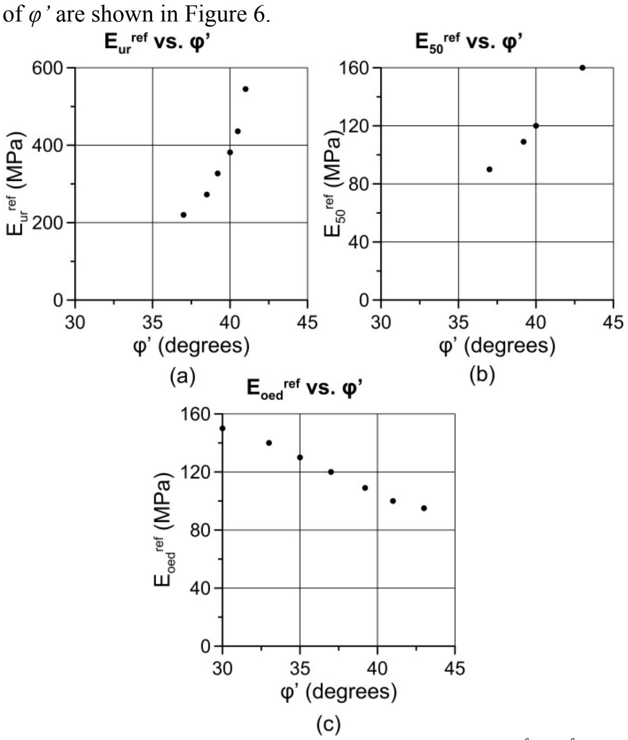

Figure 6 Parametric study results on the relationship of E_{50}^{ref}, E_{oed}^{ref}, and E_{ur}^{ref} against φ'.

Figure 6(a) shows that E_{ur}^{ref} has minimal effect on φ' as significant change in E_{ur}^{ref} only caused relatively smaller changes in φ'. This implies the minimal role of E_{ur}^{ref} in governing the orientation of the numerical PMT results. Therefore, the default configuration where $E_{ur}^{ref}=3E_{50}^{ref}$ is applicable as E_{ur}^{ref} has little influence on the numerical PMT results. This is analogous to Eq. 3 as suggested by Briaud (1992), which translates E_{50}^{ref} to E_{PMT} and E_{ur}^{ref} to the Young's modulus, E' of the highly fractured rock, in this case, the metamorphic phyllite. This is further supported by (Haberfield and Johnston, 1986) who stated that the unload-reload modulus of a rock mass is more closely related to the intact modulus of the rock.

Figures 6(b) and 6(c) show that, as compared to the change in E_{ur}^{ref}, a relatively smaller change in either E_{50}^{ref} and E_{oed}^{ref} led to a larger changes in φ', which implies that φ' is highly sensitive towards E_{50}^{ref} and E_{oed}^{ref}. This is because both E_{50}^{ref} and E_{oed}^{ref} governs the shear hardening yield surface and the yield cap surface, respectively (Plaxis, 2013). As a result, the determination of E_{50}^{ref} and E_{oed}^{ref} controls the stress level at which the modelled material starts to behave plastically, hence the high sensitivity of φ' in response to changes in E_{50}^{ref} and E_{oed}^{ref}.

By complying to the aforementioned considerations, the ratio between E_{50}^{ref}, E_{oed}^{ref}, and E_{ur}^{ref} was kept to 1:1:3 in order to conform to the findings of Haberfield and Johnston (1986) and Briaud (1992).and enable the HSM to translate the Young's modulus, E' and the pressuremeter modulus, E_{PMT} to E_{ur}^{ref} and E_{50}^{ref} respectively. The value of φ' was determined to be 39°, which was comparable to the results obtained from direct shear testing on tunnelling spoils of phyllite (Choo and Ong, 2015) and within the range of φ' that was specified by Meng et al. (2013). These HSM parameters produced numerical PMT results that agreed well with the modified, measured PMT results. The parameters are summarized in Table 1, and the generated numerical PMT results from the selected parameters are shown in Figure 7.

Figure 7 shows that the generated numerical PMT results were able to fit almost perfectly well with the measured, incomplete PMT results in the field. This reflects the accuracy and reliability of the derived HSM parameters in replicating the true stress-strain behavior of highly fractured phyllite from the physical PMT in the field. Furthermore, the accuracy for the extrapolation of the Plastic stage in the numerical PMT results depends on the availability of the plastic deformation in the measured PMT results as it would allow for more data points to be fitted by the numerical PMT results through the adjustment of c'.

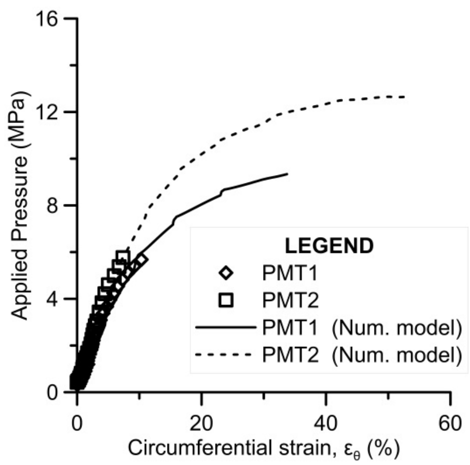

Figure 7 Comparison of numerical PMT results and measured PMT results in the field.

Table 1 Selected HSM parameters for the generation of numerical PMT results

PMT	1	2
E_{50}^{ref} (MPa)	109	120
E_{oed}^{ref} (MPa)	109	120
E_{ur}^{ref} (MPa)	327	360
E_{ur}^{ref} / E_{50}^{ref}	3	3
c' (kPa)	2400	3000
φ' (°)	39	39

5 CONCLUSION

The numerical approach presented in this paper has shown its potential in generating useful Hardening Soil Model (HSM) parameters to characterize the nonlinear stress-strain behavior of the highly fractured phyllite or as a matter of fact, it could be potentially use in other types of geology, but only after rigorous validation has taken place. The numerical models of PMT have successfully managed to extend or reasonably 'extrapolate' the Plastic Stage which was not fully developed in the measured PMT results due to the limitation in the allowable working pressure of the pressuremeter probe used for this particular field testing. In the study, the extended Plastic Stage in the numerical PMT results allows for better understanding of the plastic behavior of the highly fractured phyllite.

By assigning the E_{PMT} to E_{50}^{ref}, the default ratio of E_{50}^{ref}, E_{oed}^{ref}, and E_{ur}^{ref} in the HSM Plaxis (2013) was able to replicate the stress-strain behavior of the highly fractured rocks as presented in the measured PMT results. It was through this configuration as well that the unload-reload modulus, E_{ur} was able to be determined. It was observed in the parametric study that by using the default ratio of 1:1:3 for the three stiffness moduli of HSM and the value of E_{PMT} for E_{50}^{ref}, the friction angle, *φ'* can be determined by adjusting the *φ'* to fit the Pseudo-elastic Stage of the numerical PMT results against the modified PMT results.

Therefore, this approach has provided a possible alternative to derive the unload-reload modulus without having to conduct the unload-reload cycle. Additionally, the ability to extrapolate the plastic deformation in the numerical PMT results has also opened the opportunity to gain insights on the plastic behavior of weak rocks subjected to PMT, which the current pressuremeter designs are not able to provide. The numerical approach in this study is especially useful in cases where plastic deformation could not be significantly achieved via field testing for various reasons.

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