Back-analysis and finite element modeling of jacking forces in weathered rocks

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HIGHLIGHTS

- A novel approach is proposed for pipe-jacking through weathered geology.
- Nonlinear rock strength parameters were developed to assess pipe-jacking forces.
- Higher rock strength leads to arching, which reduces pipe-rock friction and jacking forces.
- 3D FEM confirms reliability of direct shear test results in assessing jacking forces.

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D E L Ong^{1,*}; C S Choo²

Abstract: Prediction of jacking forces has been well-established for pipe-jacking drives traversing soils. However, the accrual of jacking forces for drives negotiating weathered rock formations has not been well understood. Three pipe-jacking drives in Kuching City, Malaysia spanning weathered lithological units of sandstone, phyllite and shale were studied. In the absence of in-situ pressuremeter testing during the investigation stage, tunneling rock spoils were collected and characterized through direct shear testing. The “generalized tangential” technique was applied to the nonlinear direct shear test results to obtain linear Mohr-Coulomb parameters, c'_p and ϕ'_p . This allowed for back-analysis of frictional coefficient, μ_{avg} through the use of a well-established predictive jacking force model. The reliability of using c'_p , ϕ'_p and μ_{avg} was assessed through 3D finite element modeling of the studied pipe-jacking drives. Based on these parameters, the results obtained from the numerical analyses of the studied pipe-jacking drives show good agreement with the jacking forces measured in-situ. The outcome of this research demonstrates that the derived strength parameters from direct shear testing of tunneling rock spoils has the potential to be used as reliable input parameters in finite element modeling to predict pipe-jacking forces in highly weathered geological formations.

Keywords: Pipe-jacking; Friction; Direct shear; Rocks; Finite element modeling

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1. INTRODUCTION

The understanding of pipe-jacking forces is critical in project planning and cost estimation of underground infrastructure projects. Such was the case in the installation of 7.7 km trunk sewerage pipelines within the central business district of Kuching City, Malaysia. The microtunneling works traversed the young pre-upper carboniferous Tuang formation, characterized by tightly folded, highly fractured lithological units of shale, phyllite, and sandstone (Tan, 1993).

As a result of the highly fractured geology, the extracted rock cores were scarcely intact. The majority of recovered cores from the studied pipe-jacking drives had Rock Quality Designation (RQD) values of zero. Intact rock core lengths are necessary for common rock strength tests, such as uniaxial compression strength (UCS), and point load. The lack of intact rock cores created challenges in assessing the in-situ strength parameters of the highly fractured weathered rock lithological units. The situation was made worse as in-situ pressuremeter tests were not locally available and were not budgeted during the investigation stage of the project. As rock strength parameters were essential in the prediction of jacking forces, the lack of any reliable rock strength values impeded further efforts in assessing jacking loads accrued during pipe-jacking works.

The work described in this paper was motivated by the need to characterize rock mass strength of the various highly fractured lithological units found within the Tuang Formation in Kuching City. Hence, a novel approach by reconstituting tunneling rock spoils obtained during the construction stage of the project was proposed to characterize the rock strengths for subsequent use in the back-analysis and prediction of jacking forces. The use of tunneling rock spoils for the purpose of back-analyzing jacking forces was explored (Choo and Ong, 2012, 2014, 2015), where rock spoils from tunneling works were reconstituted and scalped for direct shear testing. Based on field measured jacking forces, the direct shear test results of

c' and ϕ' were applied to a well-established empirical jacking force predictive model (Pellet-Beaucour and Kastner, 2002) to reliably back-analyze soil-pipe frictional coefficients.

In this paper, finite element analysis was used as a tool to assess the reliability of applying the strength parameters obtained from direct shear testing to predict jacking forces and compare against corresponding field measured values.

1.1. Direct shear testing

Iscimen (2004) developed an apparatus capable of measuring soil-pipe interface shear stresses for curved surfaces, such as that on the outer peripheral of jacked pipes. The apparatus was outfitted with interchangeable inner walls to allow for testing at different pipe curvatures. Various sands were tested against pipes made from different materials. The soil-pipe interface strength was found to increase with surface roughness, up to an upper limit “critical roughness”, beyond which the influence of surface roughness on shear strength value was negligible. This value corresponded with the internal shear strength of the soil. This observation was similar to that made by Kishida and Uesugi (1987). The study found that particle shape greatly affected the soil-pipe interface shear strength, with angular particles exhibiting stronger interlocking than sub-rounded particles, and this is an important observation for the work presented in this paper as both argillaceous and arenaceous materials are tested.

Staheli (2006) conducted interface direct shear testing on various commonly available pipe materials, using sands. Interface friction values were developed from the tests and applied to field case studies for comparison. From analyses of the measured jacking forces for unlubricated sections of the case studies, a frictional pipe-jacking force model was developed to estimate normal stresses acting along the pipeline. The pipe-jacking force model was based on Terzaghi’s arching theory. However, usage of the model is restricted to unlubricated pipe-

jacking drives. The work showed that the direct shear test is capable of obtaining useful strength parameters for the assessment of jacking forces in unlubricated pipe-jacking drives.

Shou et al. (2010) conducted a simple large scale frictional experiment to test the frictional properties of various lubricants. A discrete lubricating layer was introduced between a concrete block and a soil mass. This would simulate the soil-pipe interaction in pipe-jacking at the pipe invert. It was found that the combination of plasticizer with polymer was most effective in reducing jacking forces due to frictional resistance. The findings from the large scale frictional test were applied to two sections (one linear and one curved) of a pipe-jacking case study in Taiwan. The analyses of jacking forces using frictional coefficients from the experiment resulted in overestimation of the jacking forces when compared against site measurements. This discrepancy was attributed to overestimation of the soil-pipe contact area. Shou et al. (2010) showed that the direct shear test can be used to assess jacking forces for lubricated drives.

Choo and Ong (2015) evaluated pipe-jacking forces accrued through drives in highly fractured and weathered rock. Equivalent strength parameters based on nonlinear power law functions were obtained from direct shear testing of reconstituted tunneling rock spoils. These strength parameters were subsequently applied to a well-established jacking force model by Pellet-Beaucour and Kastner (2002). Interpreted values of frictional coefficient, μ_{avg} for various weathered lithological units were analyzed against jacking forces, lubrication patterns and stoppages for consistency. The authors found that the use of reconstituted tunneling rock spoils was reliable in the analyses of jacking loads for drives traversing weathered rock formations. Hence, the use of finite element modeling in the current paper is a natural extension of the established work by Choo and Ong (2015).

The various studies presented above that involve interface friction and soil-structure interaction have clearly indicated that direct shear testing can be used to study jacking forces.

The applicability of direct shear testing allows for the characterization of rock strength during the construction stage, and subsequent back-analyses of measured jacking forces. This is significant in light of the unavailability of in-situ testing methods (such as the pressuremeter test) during the early investigative stage of the project. The suitability of using direct shear test results from reconstituted tunneling rock spoils for the back-analyses of jacking forces can be validated through 3D finite element analysis.

1.2. Jacking force models

Jacking force models are typically dependent on the geology traversed during pipe-jacking (Chapman and Ichioka, 1999; Osumi, 2000; Staheli, 2006). The majority of these models are well-developed for predicting pipe-jacking forces when traversing soils. Such models usually rely on the shear strengths of the traversed geomaterials, which are typically characterized by the Mohr-Coulomb model. Frictional coefficients for different combinations of soil-pipe materials are often developed empirically. There are very limited provisions made for strength characterization and values of frictional coefficients for pipe-jacking involving rock-structure interaction. This study proposes a novel method for obtaining equivalent rock strength properties for use in the analysis of jacking forces using a well-established jacking force equation developed by Pellet-Beaucour and Kastner (2002).

Pellet-Beaucour and Kastner (2002) developed a predictive jacking force equation based on nine microtunneling operations in France. The model postulated that normal stresses acting on the outer periphery of the pipe resulted from ground overburden pressure. As a pipeline is buried at depth, the phenomenon of soil arching becomes relevant in understanding the contact stresses acting on the pipe. The phenomenon of soil arching was observed by Terzaghi (1936) by the downward deflection of a trap door underlying a sand mass. The opening of the trap door is analogous to the excavation of a tunnel. This disturbance to the geostatic conditions induces a redistribution of soil stresses, principally

oriented to the tunnel perimeter. The soil stresses acting normal to the trap door (or tunnel in this case) are relaxed, thus reducing the contact stresses on the outer surface of any installed structure, i.e. jacked pipes in a microtunneling operation. The model developed by Pellet-Beaucour and Kastner (2002) was preferred for use in this paper due to its consideration of arching effect. This predictive jacking force model is shown in Eq. (1),

$$F = \mu L D_e \frac{\pi}{2} \left[\left(\sigma_{EV} + \frac{\gamma D_e}{2} \right) + K_2 \left(\sigma_{EV} + \frac{\gamma D_e}{2} \right) \right] \quad (1)$$

in which F is the total frictional jacking force; L is the pipe span; D_e is the outer pipe diameter; σ_{EV} is the vertical soil stress at the pipe crown; γ is the soil unit weight; and K_2 is the thrust coefficient of soil acting on the pipe, with a suggested value of 0.3 (French Society for Trenchless Technology, 2006). μ is the coefficient of soil-pipe friction, given in Eq. (2) as

$$\mu = \tan \delta \quad (2)$$

where δ is the soil-pipe friction angle. Previous studies have empirically assumed δ to be half the internal soil friction angle, ϕ (Sofianos et al., 2004; Staheli, 2006). More realistically, Stein et al. (1989) recommended values of μ for various states of friction encountered during pipe-jacking. In lubricated drives, μ was recommended to be between 0.1 – 0.3. Various studies have used these values of μ as a basis for identifying lubricated drives through soil and rock (Barla et al., 2006; French Society for Trenchless Technology, 2006; Pellet-Beaucour and Kastner, 2002).

The vertical soil stress at the pipe crown, σ_{EV} is determined by the soil cohesion, C ; the soil internal friction angle, ϕ ; the lateral earth pressure coefficient, K ; h is the soil cover from the ground level to the pipe crown; and the influencing soil width above the pipe, b ;

$$\sigma_{EV} = \frac{b \left(\gamma - \frac{2C}{b} \right)}{2K \tan \phi} \left(1 - e^{-2K \frac{h}{b} \tan \phi} \right) \quad (3)$$

$$b = D_e \left(1 + 2 \tan \left(\frac{\pi}{4} - \frac{\phi}{2} \right) \right) \quad (4)$$

σ_{EV} is derived from Terzaghi's limit equilibrium approach for estimating the vertical stresses acting on the roof of a tunnel (Terzaghi, 1943). The excavation of a tunnel was synonymous with the deflection of a trapdoor beneath a sand mass. The creation of a tunnel results in localized stress redistribution around the tunnel, which could result in a decrease in vertical stresses at the tunnel roof. This reduces the normal stresses acting on the pipe crown, resulting in reduced frictional jacking forces. The understanding of σ_{EV} is therefore significant in anticipating the magnitude of jacking forces. The formulation of σ_{EV} shows a dependence on Mohr-Coulomb (MC) strength parameters, c' and ϕ' . These MC parameters are difficult to evaluate as the rock cores extracted from the Tuang Formation were highly fractured.

1.3. Arching ratio

From Terzaghi's trapdoor experiment, several studies have been conducted to quantify the effects of arching. McNulty (1965) assessed the degree of soil arching induced by a yielding trapdoor by examining values of arching ratio. McNulty (1965) defined arching ratio as the ratio of the load on the yielded door to the load on the undeflected door. This arching ratio was expressed as

$$\rho = \frac{P_B}{P_S} \quad (5)$$

where ρ = arching ratio; P_B = average pressure acting on the trapdoor; and P_S = surcharge pressure acting on the surface of a soil mass. An arching ratio less than 1.0 implied the presence of active arching, while an arching ratio greater than 1.0 was indicative of passive arching. Based on McNulty's study, similar expressions of arching ratio were formulated for reinforced embankments to estimate effect of arching in the embankment fill (Borges and

Marques, 2011; Deb, 2010; Han and Gabr, 2002; Ye et al., 2012). Abbott (1967) theorized a similar formulation for arching ratio applied to buried structures, given as

$$A_{\sigma} = \frac{p_t}{p_o} = \frac{p_o + p_a}{p_o} \quad (6)$$

where A_{σ} is the arching ratio; p_t is the total average axial stress carried by structure; p_o is the static overpressure; and p_a is the average axial stress sustained by structure due to arching. p_o refers to additional static loading. Lee et al. (2006) offered a similar expression of arching ratio in terms of the change in vertical stresses due to tunneling, given as

$$AR(\%) = \frac{\Delta\sigma_v}{\sigma_{vo}} \times 100 \quad (7)$$

where AR is the arching ratio; $\Delta\sigma_v$ is the change in vertical stress during tunneling; and σ_{vo} is the total overburden pressure. These equations will be subsequently used to verify the presence of arching in the various pipe-jacking drives presented in this study.

2. STRENGTH CHARACTERIZATION OF RECONSTITUTED TUNNELING ROCK SPOILS

During soil investigation works, rock cores were extracted from boreholes located at the pipe-jacking shafts along the fractured and highly weathered Tuang Formation in Kuching City. The extracted rock cores revealed very low RQD values. Ong and Choo (2011) reported similar observations during rock coring for a separate project sited also in the Tuang Formation. Thus, the use of reconstituted tunneling rock spoils for the development of strength parameters was examined. Such work has been conducted by Choo and Ong (2015).

Samples of tunneling rock spoils were sourced from pipe-jacking sites in Kuching City, which traversed sandstone, phyllite, and shale. Tunneling rock spoils were collected from decantation chambers, which screened slurry for coarser crushed spoils. The tunneling rock spoils were products of the excavation at the face of the microtunneling boring machine

(MTBM), and subsequent crushing in the crushing cone of the MTBM. Preparation of specimens for direct shear testing required the removal (or scalping) of grains with sizes exceeding a tenth of the thickness of the test specimen (Head, 1992).

The scalped tunneling rock spoils were subjected to strain-controlled drained direct shear tests in accordance with ASTM D 3080 (2003) and AS 1289.6.2.2 (1998). The fully automated GeoComp ShearTrac II direct shear system was used to conduct the consolidation and shearing of scalped tunneling rock spoils. Full automation of the test ensured reliability of the tests, and a high resolution of monitored parameters, including shear stresses, shear deformations, and vertical deformations.

Choo and Ong (2015) highlighted the nonlinearity of the shear strength of reconstituted tunneling rock spoils. Hence, a simple power law function was adopted to capture the nonlinearity of the test data (De Mello, 1977)

$$\tau = A \cdot (\sigma')^B \quad (8)$$

where A and B are constants. However, the jacking force model (see Eq. (1)) and the estimation of vertical stresses due to arching (see Eq. (3)) still required the use of MC parameters. Thus, the “generalized tangential” method was used to develop MC parameters from the nonlinear data obtained from direct shear tests (Yang and Yin, 2004). The direct shear test results and interpretation of tangential MC parameters are shown in Fig. 1.

3. BACK-ANALYSES OF PIPE-JACKING FORCES

Three underground pipe-jacking drives were studied and analyzed by Choo and Ong (2015). The drives traversed through lithological units of sandstone, phyllite, and shale at various depths. By applying the “generalized tangential” technique on earlier direct shear testing of tunneling rock spoils, MC parameters were derived for the back-analysis of frictional coefficient, μ_{avg} in the studied drives using Eqs. (1) & (3). A comparison was conducted of the back-analyzed μ_{avg} against recommended values for lubricated drives, i.e. μ

= 0.1 – 0.3 (Stein et al., 1989). The parameters used for back-analysis of the studied drives are presented in Table 1, with the measured jacking forces and resulting back-analyzed μ_{avg} shown in Figs. 2, 3 & 4 for Drives A, B & C respectively.

The jacking of Drive A encountered lithological units of sandstone, during which the average measured jacking force, JF_{meas} was 14.4 kN/m (see Table 1). The MC parameters applied to the surrounding rock mass (Eq. (3)) were $C = c'_p = 50.8$ kPa and $\phi = \phi'_p = 47.8^\circ$. These values were derived from the tangent to the nonlinear function, applied at the effective overburden pressure indicative of the depth of the pipeline. Subsequently, the calculated σ_{EV} (Eq. (3)) was -20.8 kN/m^2 , indicating a strong occurrence of arching. For the purpose of the back-analysis, negative values of σ_{EV} were adjusted to zero, since negative values would imply tensile stresses acting on the pipe crown, which is impossible due to the large overburden pressure. The amount of lubrication injected in the overcut was measured to be 47 L/m. As discussed in Choo and Ong (2015), comparison of the amount of injected lubricant against the theoretical overcut volume (87 L/m) suggests that this drive was not well-lubricated, thus justifying the back-analyzed μ_{avg} of 0.315. This value was at the upper limit of the suggested frictional coefficient for lubricated drives. Despite the reduced volume of injected lubricant, significant arching had reduced the normal soil stresses on the pipe, mitigating further accrual of jacking forces.

For Drive B, the propulsion of the jacked pipeline through phyllite formations resulted in average measured jacking forces of 4.8 kN/m (see Table 1). From the direct shear tests on reconstituted phyllite spoils (i.e. $C = 57.8$ kPa and $\phi = 44.3^\circ$), the calculated σ_{EV} was -22.3 kN/m^2 (Choo and Ong, 2015). Similar to Drive A, the calculated σ_{EV} implied a high degree of arching over the pipe-jacking drive. Subsequently, the back-analyzed μ_{avg} was 0.068, suggesting the drive through phyllite was well-lubricated. This was verified by the high volumes of injected lubricant utilized for this drive, at 181 L/m compared to a theoretical

overcut volume of 113 L/m. The saturation of the overcut region with lubricant ensured a discrete lubricating layer, allowing for the reduction of rock-pipe interface shear stresses, and the suspension of the jacked pipeline within the bored tunnel. Suspension of the jacked pipeline reduced the rock-pipe contact area, thus further reducing jacking forces.

For Drive C in shale, the average measured jacking forces were 81.1 kN/m (see Table 1), which were the highest jacking forces observed among the studied drives. The estimation of vertical stresses on the pipe crown resulted in a σ_{EV} of 11.7 kN/m² (Choo and Ong, 2015). In comparison with Drives A & B, this implied a reduced degree of arching acting on the outer peripheral of the jacked pipeline. Measurements of the injected lubricant at 729 L/m far exceeded the theoretical overcut volume of 113 L/m. However, unlike Drive B, the excessive injection of lubricant indicated a significant loss of lubricant into the surrounding rock mass. This loss of lubricant suggested that the water tightness of the overcut region had been compromised, thus mitigating any lubricating and buoyancy effects, and increasing the jacking forces. This was verified by the back-analyzed μ_{avg} of 0.713. Coupled with the reduced arching, the back-analyzed μ_{avg} was significantly higher than those for Drives A & B.

The back-analyses of jacking forces based on the strength parameters obtained from the direct shear tests on reconstituted tunneling rock spoils have been conducted. The direct shear test results have been used to assess the measured jacking forces, in relation to arching phenomenon and lubrication use. As shown in Figs. 2, 3, and 4, the reliable outcomes of the back-analyses of jacking forces have successfully demonstrated the value in performing direct shear tests using tunneling rock spoils in the absence of in-situ rock strengths. The application of the direct shear test results is conducted hereinafter, through the use of 3D finite element analysis.

4. NUMERICAL ANALYSES OF PIPE-JACKING FORCES

3D finite element analysis was conducted to study the accrual of rock-pipe interface shear stresses through the simulation of typical spans along the studied drives. The analysis was performed using PLAXIS 3D, a commercially available finite element software package developed specifically for 3D analyses (Plaxis bv, 2013).

4.1. Finite element mesh

The finite element model used to simulate the observed pipe-jacking drives was 30 m wide (x -direction transverse to pipeline), 20 m long (y -direction longitudinal to pipeline) and 30 m deep (z -direction), as shown in Fig. 5. The rock and soil masses were modeled using 10-node tetrahedral elements, with the pipeline modeled using 6-node plate elements. 12-node interface elements were used to model rock-pipe interaction. Standard fixity conditions were applied to the boundaries. Full fixity was applied to the bottom face to restrain movements in all directions, with rollers applied to the vertical faces to restrain movements in the horizontal plane.

4.2. Rock properties

The rock masses in the studied drives were assigned an elastic-perfectly plastic constitutive model, utilizing the MC failure criterion. It is recognized that other improved models are available for modeling of rock masses, such as the nonlinear Hoek-Brown failure criterion. However, the MC failure criterion was chosen to allow for parity with the stipulated MC parameters required for back-analyses of jacking forces (Pellet-Beaucour and Kastner, 2002) (Eqs. 1 & 3). Earlier interpretation of direct shear testing had acknowledged the nonlinearity of the shear strength for the tested tunneling rock spoils. Using the ‘generalized tangential’ technique (Yang and Yin, 2004), it was possible to express the nonlinear test results in terms of equivalent linear MC parameters, c'_p and ϕ'_p . These parameters were used

to model the rock mass, where $c' = c'_p$ and $\phi' = \phi'_p$. The stiffness of the modeled rock masses were based on the findings of Bieniawski (1989),

$$E_M = 10^{(\text{RMR}-10)/40} \quad (9)$$

$$\text{RMR} = 9 \ln Q + 44 \quad (10)$$

where E_M is the stiffness of the rock mass; RMR is the rock mass rating (Bieniawski, 1979); and Q is the rock mass quality, which is a function of various rock joint properties including RQD (Barton et al., 1974). Due to the highly weathered nature of the geology along the studied drives, an RQD value of zero was assumed. This resulted in a rock modulus value of 7.1 GPa (see Tables 2, 3 & 4).

In Eq. (3), Pellet-Beaucour and Kastner (2002) assumed constant isotropic conditions ($K = 1$ and $K_0 = 1$) for the soil above the tunnel. Hence, isotropic conditions were maintained for the numerical analyses conducted hereinafter to allow for a comparable simulation against the back-analyzed jacking forces.

4.3. Pipeline, and rock-pipe interface properties

The diameters of the modeled concrete pipelines are given in Table 1. The modeled pipelines were 20 m long in the y-direction, and penetrated the finite element model, where both ends of the modeled pipelines were flushed with the vertical model boundaries. Fixities at the front end of the pipelines were subsequently released to allow for longitudinal displacements of the pipeline, which was simulated by applying displacements at the tail ends of the pipelines. The concrete pipelines were regarded as rigid bodies by artificially increasing its elastic modulus. This was reasonable as the concrete pipeline has high stiffness relative to the surrounding rock-pipe interface (Tian et al., 2014; Wang, D et al., 2009; Zhang et al., 2013). The rigid pipeline would facilitate the segregation of the stresses due to pipe compression from the jacking forces.

For numerical modeling of pull-out tests, Yin et al. (1995) suggested an interface tension modulus, $E'' = 1$ kPa for sliding failure associated with the deformation of a soil-structure interface. The low value of E'' was assigned so as to simulate separation of the interface when subjected to tensile stresses. A similarly low value of tangential interface modulus, $E_t = 17.4$ kPa was used for a study of interface elements in pull-out tests which underwent large shear deformations (Wang, X and Wang, 2006). For the rock-pipe interface elements used in this study, a similarly low value of interface modulus, $E_{inter} = 1,000$ kPa was used.

The rock-pipe interface strength properties can be obtained from

$$\tau_i = \sigma'_n \tan \phi_i \quad (11)$$

$$\phi_i = \tan^{-1}(\mu_{avg}) \quad (12)$$

given that τ_i is the rock-pipe interface shear strength; σ'_n is the effective normal stress acting on the rock-pipe interface; ϕ_i is the interface friction angle; and μ_{avg} is the back-analyzed average frictional coefficient, obtained from earlier back-analyses of jacking forces. The material properties in the respective drives for soil, rock, rock-pipe interface and pipeline are shown in Tables 2, 3 and 4.

4.4. Modeling of pipe-jacking process

All drives were modeled in three stages:

Stage 1 - Application of initial stress conditions.

Stage 2 - Wish-in-place pipeline, and activation of rock-pipe interface elements.

Stage 3 - Application of displacements at tail-end of pipeline.

In Stage 1 the generation of initial greenfield stress conditions was performed through the assignment of soil material properties, groundwater levels, K_0 condition, and model boundary conditions. Plate elements and interface elements were present in this stage, but were deactivated.

In Stage 2 the plate elements were then activated. It was assumed that 20 m of the pipeline had already been installed into the block of elements, thus ignoring any installation effects. The soil elements within the pipeline were deactivated at this stage. Water conditions within the pipeline were also set to dry, removing the effects of hydrostatic pressure within the void pipeline. Subsequently rock-pipe interface elements were activated.

Techniques for modeling of tunnels include the deactivation of unsupported soil elements (Chen et al., 2011), or the volume loss control method through the prescription of surface contraction along the modeled pipeline (Potts and Zdravković, 2001). Such techniques are typically applicable to the excavation of NATM tunnels, or large diameter mechanized tunneling. When these modeling methods are applied to the simulation of pipe-jacking drives, the convergence issues are often encountered particularly due to the high deformations of the interface elements. Hence, it was proposed for the pipeline to be ‘wished-in-place’ in order to negate convergence issues. The effects of arching would be considered subsequently, when the finite element modeling results are being analyzed in detail.

In Stage 3 horizontal (y-direction) displacements were applied at the tail end of the pipeline. Fixities at the front end of the pipeline were released by introducing displacements along the periphery of the plate elements at the front. This was done to facilitate movement of the pipe beyond the boundaries of the soil block.

4.5. Analysis of results from numerical simulation

The displacements of plate elements representing the pipeline are shown in Fig. 6. All pipeline elements in the three simulated drives were displaced uniformly by 100 mm in the y-direction through the prescription of displacements at the tail end of the pipe. The exaggeration of pipe stiffness allowed for the modeled pipeline to be displaced evenly, mitigating any effects of pipe compression on the rock-pipe interface shear stresses. The displacement of 100 mm was sufficient for the interface elements to achieve yield (Fig. 7).

Fig. 8 shows the distribution of interface shear stresses for Drive B, with similar observations made for Drives A & C. The distribution of interface shear stresses, τ_1 was uniform along the longitudinal cross-section of the modeled pipeline, with the exception of interface elements at the ends of the pipeline. This was due to end effects, resulting from the discontinuity of interface and soil elements at the pipe ends (Plaxis bv, 2013). Radial distributions of τ_1 at the cross-section of the pipeline taken at mid-span of the pipeline were also uniform for all simulated drives. Fig. 9 shows the radial distribution of τ_1 for Drive B, with similarly uniform distributions observed for Drives A & C. This demonstrates the consistency of the finite element modeling technique.

Aside from the pipe ends, the uniformity of interface shear stresses elsewhere indicated that all interface elements had reached plasticity. Plasticity is significant in the modeling of pipe-jacking works as most jacking force predictive models assume no change in frictional resistance with displacement when tunneling through homogeneous geological conditions. Plasticity of the interface stresses can be verified using the appropriate overburden pressure acting on the pipe crown, with the back-analyzed frictional coefficient, μ_{avg} (see Eqs. 11 & 12).

With the reliability of the finite element modeling technique developed herein, the issue of considering arching effects remains to be addressed. The pipe-jacking force model established by Pellet-Beaucour and Kastner (2002) relies on the effect of arching in order to determine the vertical stresses acting on the pipe crown, σ_{EV} (see Eq. (3)). Arching reduces the soil stresses acting normal to the peripheral of the pipe crown, thus reducing the frictional interface shear stresses. This reduction in rock-pipe interface shear stresses due to arching can be reflected by applying an appropriate arching ratio to the interface shear stresses, τ_1 computed from the earlier finite element modeling. For pipe-jacking works, the arching ratio can be expressed as the change in soil stresses due to tunneling in relation to the total

overburden pressure (see Eq. 7). By considering the pipe-jacking force model (see Eqs. (1) & (3)), the arching ratio, AR is re-expressed and proposed to be

$$AR(\%) = \frac{\sigma_{EV} + \frac{\gamma D_e}{2}}{\gamma \left(h + \frac{D_e}{2} \right)} \times 100 \quad (13)$$

The arching ratio given in Eq. (13) was applied to the post-processing of interface shear stresses obtained from the numerical modeling, with the calculated jacking forces, $JF_{\tau,cal}$ given as

$$JF_{\tau,cal} = AR \times \tau_1 \pi D_e \quad (14)$$

where τ_1 is the interface shear stress tangential to the direction of displacement (y-direction). The resulting calculated jacking forces for the studied drives are shown in Table 5, and were subsequently compared against the respective measured jacking forces, JF_{meas} . This shows reasonable agreement between the calculated jacking forces from numerical modeling against the jacking forces measured in-situ, i.e. error of between 15% and 25%. Further verification was conducted by examining the reaction forces on the prescribed displacements, given that

$$JF_{F_y,cal} = AR \times \frac{\sum F_y}{L} \quad (15)$$

where $JF_{F_y,cal}$ is the jacking force calculated from reaction forces; $\sum F_y$ is the reaction force on prescribed displacements; and L is the pipe length. The jacking forces, $JF_{F_y,cal}$ derived from the reaction force, $\sum F_y$ were also in reasonable agreement with the average measured jacking forces, JF_{meas} (see Table 5).

For each modeled drive, the stress fields were statically admissible since the reaction forces, $\sum F_y$ were in equilibrium with the interface shear stresses, τ_1 integrated along the outer surface area of the modeled pipeline (see Table 5). This state of equilibrium was achieved with the elastic-perfectly plastic MC failure criterion adopted for the interface elements, where the yield criterion was not exceeded. Hence, the solutions resulting from finite element

modeling are considered lower bound. During tunneling, the traversed geology was crushed into tunneling rock spoils, resulting in the significant loss of cementation in rock mass. Hence, the use of strength parameters obtained from testing on the tunneling rock spoils further reinforces that the results from finite element modeling were lower bound solutions. This resulted in the underestimation of measured jacking forces obtained from the finite element modeling. This underestimation of measured jacking forces could also have been attributed to the assumption of average uniform conditions along the studied drives, such as lubrication and rock-pipe contact across the modeled drive. However, this assumption was necessary in order to draw parity with the jacking force model (Eqs. 1 & 3), which considers jacking forces to be uniform along homogenous geological conditions. In reality, the distribution of lubrication and rock-pipe contact are expected to fluctuate, resulting in variations of jacking forces (see Figs. 2, 3 & 4) (Marshall and Milligan, 1996).

Direct shear test results, c'_p & ϕ'_p and back-analyzed frictional coefficient, μ_{avg} have been successfully applied to three pipe-jacking drives. The application of these parameters has also been conducted using 3D finite element modeling technique to assess the reliability of these parameters for the assessment of pipe-jacking forces through the highly fractured and weathered geology of the Tuang Formation in Kuching City, Malaysia. The jacking forces derived from numerical modeling have shown reasonable agreement with the measured jacking forces, demonstrating the reliability in using the tangential strength parameters developed from direct shear tests on reconstituted tunneling rock spoils.

5. CONCLUSIONS

A novel approach was proposed for the assessment of pipe-jacking forces in highly weathered and fractured geology, involving the reconstitution of tunneling rock spoils for direct shear testing. The tunneling spoils were assessed for geotechnical strength properties using a “generalized tangential” technique to allow for the application of the nonlinear test

results to a well-established jacking force model (Pellet-Beaucour and Kastner, 2002), which requires the use of linear elastic-perfectly plastic MC strength parameters, i.e. c'_p and ϕ'_p . Back-analyses of the jacking forces using the tangential MC strength parameters have been successfully executed, resulting in the development of μ_{avg} for the three drives, i.e. $\mu_{avg} = 0.315$ (not well-lubricated) for Drive A in sandstone, $\mu_{avg} = 0.068$ (well-lubricated) for Drive B in phyllite, and $\mu_{avg} = 0.713$ (loss of lubrication resulting in high friction) for Drive C in shale.

Further validation of c'_p , ϕ'_p and μ_{avg} was conducted through 3D finite element modeling technique of the three pipe-jacking drives. Strength parameters c'_p and ϕ'_p were applied to the model rock mass, while the back-analyzed μ_{avg} was applied to the failure criteria of the rock-pipe interface elements. The modeled concrete pipelines were wished-in-place, with the effects of arching considered during post-processing of the results through the use of a proposed arching ratio. The proposed arching ratio was applied to the resulting rock-pipe interface shear stresses and reaction forces calculated from the finite element modeling results, which demonstrated reasonable agreement between the corresponding predicted and measured jacking forces.

These results have demonstrated the consistency and reliability of the results from reconstituted tunneling rock spoils tested in the direct shear apparatus. This allows for potential prediction of pipe-jacking forces through weathered geological formations by using the direct shear test on reconstituted tunneling rock spoils for future pipe-jacking works in the subsequent phases of the project.

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 566

Figure 1

Fig. 1. Results from direct shear testing of scalped tunneling rock spoils (Choo and Ong, 2014b)

Figure 2

Fig. 2. Drive A: Outcome of back-analyzed μ_{avg}

Figure 3

Fig. 3. Drive B: Outcome of back-analyzed μ_{avg}

Figure 4

Fig. 4. Drive C: Outcome of back-analyzed μ_{avg}

Figure 5

Fig. 5. Typical model used for numerical analysis of interface shear stresses (model for Drive B shown above)

Figure 6

Fig. 6. Distribution of pipeline displacements after application of prescribed displacements

Figure 7

Fig. 7. (a) Absence of plastic interface elements before displacement, as indicated by mesh outline; (b) all interface elements achieve plasticity after displacement of 100 mm, as indicated by blocks.

Figure 8

Fig. 8. Uniform distribution of interface shear stresses, τ_1 along modeled pipeline (results for Drive B shown above)

Figure 9

Fig. 9. Distribution of interface shear stresses, τ_1 at mid-span of modeled pipeline (results for Drive B shown above)

Table 1. Parameters used in pipe-jacking force model for back-analyses of μ_{avg} (Choo and Ong, 2014b)

Drive Geology		Drive A Sandstone	Drive B Phyllite	Drive C Shale
External pipe diameter, D_e (m)		1.43	1.78	1.78
Soil unit weight, γ_{soil} (kN/m ³)		18	18	18
Depth to rock level (m)		9.5	15.0	19.0
Rock unit weight, γ_{rock} (kN/m ³)		22	22	22
Height of rock cover (m)		3.0	3.5	0.5
Tangential MC parameters obtained from direct shear testing	c'_p (kN/m ²)	50.8	57.8	29.0
	ϕ'_p (°)	47.8	44.3	38.7
Average measured jacking forces, JF_{meas} (kN/m)		14.4	4.8	81.1
Vertical stress acting on pipe crown, σ_{EV} (kN/m ²) (see Eq. 3)		-20.8	-22.3	11.7
Back-analyzed μ_{avg} , using Eq. (1)		0.315	0.068	0.713
R^2		0.93	0.78	0.45

Table 2

Table 2. Material properties for finite element modeling of Drive A

Material	Stiff silty clay (SPT 10)	Very stiff silty clay (SPT 22)	Hard silty clay (SPT 50)	Sandstone	Rock-pipe interface	Pipe
Elements used	Soil	Soil	Soil	Soil	Interface	Plate
Material model	MC	MC	MC	MC	MC	Linear elastic
Depth (m)	0 to 5.8	5.8 to 8.2	8.2 to 9.5	9.5 to 20	-	-
Unit weight, γ (kN/m ³)	18	18	18	22	-	24
Cohesion, c' (kPa)	5	10	15	50.8 ^a	0	-
Friction angle, ϕ' (°)	28	28	30	47.8 ^a	20.05 ^b	-
Elastic modulus, E' (kN/m ²)	11,000	24,200	55,000	7.1×10^6	1,000	27×10^9 ^c
Poisson's ratio, ν	0.35	0.35	0.35	0.35	0.35	0.15

^aNote: MC strength values were obtained from peak parameters from earlier direct shear testing on tunneling rock spoils, i.e. c'_p and ϕ'_p .

^bNote: MC strength values for interface elements

^cNote: Elastic stiffness of modeled concrete pipeline was artificially increased to isolate effects of pipe compression from interface shear stresses.

Table 3. Material properties for finite element modeling of Drive B

Material	Firm silty clay (SPT 6)	Very stiff silty clay (SPT 18)	Hard silty clay (SPT 50)	Phyllite	Rock-pipe interface	Pipe
Elements used	Soil	Soil	Soil	Soil	Interface	Plate
Material model	MC	MC	MC	MC	MC	Linear elastic
Depth (m)	0 to 8.6	8.6 to 10.3	10.3 to 15	15 to 25	-	-
Unit weight, γ (kN/m ³)	18	18	18	22	-	24
Cohesion, c' (kPa)	5	10	15	57.8 ^a	0	-
Friction angle, ϕ' (°)	28	28	30	44.3 ^a	4.57 ^b	-
Elastic modulus, E' (kN/m ²)	13,200	19,800	55,000	7.1×10^6	1,000	27×10^9 ^c
Poisson's ratio, ν	0.35	0.35	0.35	0.35	0.35	0.15

^aNote: MC strength values were obtained from peak parameters from earlier direct shear testing on tunneling rock spoils, i.e. c'_p and ϕ'_p .

^bNote: MC strength values for interface elements

^cNote: Elastic stiffness of modeled concrete pipeline was artificially increased to isolate effects of pipe compression from interface shear stresses.

Table 4

Table 4. Material properties for finite element modeling of Drive C

Material	Loose sand (SPT 8)	Very soft clay (SPT 2)	Loose sand (SPT 8)	Dense sand (SPT 34)	Medium dense sand (SPT 19)	Very soft clay (SPT 2)	Shale	Rock-pipe interface	Pipe
Elements used	Soil	Soil	Soil	Soil	Soil	Soil	Soil	Interface	Plate
Material model	MC	MC	MC	MC	MC	MC	MC	MC	Linear elastic
Depth (m)	0 to 6.1	6.1 to 10.5	10.5 to 11.6	11.6 to 13.5	13.5 to 14.9	13.5 to 19	19 to 30	-	-
Unit weight, γ (kN/m ³)	18	18	18	18	18	18	22	-	24
Cohesion, c' (kPa)	5	5	5	15	10	5	29.0 ^a	0	-
Friction angle, ϕ' (°)	28	28	28	30	28	28	38.7 ^a	38.1 ^b	-
Elastic modulus, E' (kN/m ²)	13,910	2,200	13,910	59,130	33,040	2,200	7.1×10^6	1,000	27×10^9 ^c
Poisson's ratio, ν	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.15

^aNote: MC strength values were obtained from peak parameters from earlier direct shear testing on tunneling rock spoils, i.e. c'_p and ϕ'_p .

^bNote: MC strength values for interface elements

^cNote: Elastic stiffness of modeled concrete pipeline was artificially increased to isolate effects of pipe compression from interface shear stresses.

Table 5. Analysis of finite element modeling results

Drive Geology	A Sandstone	B Phyllite	C Shale
Prescribed displacement (mm)	100	100	100
Vertical stress acting on pipe crown, σ_{EV} (kN/m ²)	-20.8 ^a	-22.3 ^a	11.7
Arching ratio, AR (%)	6.2	5.3	8.6
Measured jacking forces, JF_{meas} (kN/m)	14.4	4.8	81.1
Interface shear stress, τ_1 (kN/m ²)	44	14	133
Jacking forces calculated from interface shear stresses, $JF_{\tau,cal}$ (kN/m)	12.4	4.2	63.8
Error, $\frac{JF_{\tau,cal} - JF_{meas}}{JF_{meas}} \times 100$ (%)	12.4	13.8	21.4
Integration of τ_1 along modeled pipeline (kN)	3953	1566	14875
Reaction force, ΣF_y (kN)	3727	1544	14630
Jacking forces calculated from reaction force, $JF_{Fy,cal}$ (kN/m)	11.6	4.1	62.7
Error, $\frac{JF_{Fy,cal} - JF_{meas}}{JF_{meas}} \times 100$ (%)	19.7	14.9	22.6

^aNote: Negative value of σ_{EV} was readjusted to zero.