Impact of highly weathered geology on pipe-jacking forces

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For the Kuching Wastewater Management System Phase 1 project in Kuching, Malaysia, 7.7 km of trunk sewer lines were constructed in the highly fractured, highly weathered Tuang Formation using a pipe-jacking method. The pipelines were founded at depths of up to 35 m below Kuching City, where the majority of the pipe-jacking activities would traverse the Tuang Formation. Jacking forces in highly fractured geology are not well understood as most jacking force models were derived for drives traversing soils. Therefore, a novel method was developed, whereby equivalent rock strength characteristics were interpreted from direct shear testing on reconstituted tunnelling rock spoils. Tangential peak strength parameters, c'_{tp} and ϕ'_{tp} , were developed from the nonlinear behaviour of the reconstituted spoils and applied to a well-established jacking model to assess arching and development of jacking forces from four documented drives. The back-analysed parameters μ_{avg} and σ_{EV} were used to demonstrate that geology had significantly affected the development of jacking forces. The back-analysis of the studied drives was successfully validated through finite-element modelling. This research shows that the developed parameters c'_{tp} and ϕ'_{tp} and the back-analysed parameters μ_{avg} and σ_{EV} can be reliably used to predict jacking forces in highly fractured, highly weathered geology.

Notation

A	dimensionless constant in the power function
A_p	peak power law parameter
AR	arching ratio
B	dimensionless exponential in the power function
B_p	peak power law parameter
b	influencing soil width above the pipe
C	soil cohesion
c'	effective cohesion of the modelled geology
c'_{tp}	tangential peak cohesion of the tunnelling rock spoils
D_e	outer pipe diameter
E'	Young's modulus
E_{inter}	modulus of the rock–pipe interface
E_M	stiffness of the rock mass
E_t	tangential modulus
F	total frictional jacking force
h	soil cover from the ground level to the pipe crown
$JF_{Fy,cal}$	jacking force calculated from ΣF_y
JF_{meas}	measured jacking force
$JF_{\tau,cal}$	jacking force calculated from τ_1
K	lateral earth pressure coefficient
K_0	at-rest lateral earth pressure coefficient
K_2	thrust coefficient of soil acting on the pipe
L	pipe span
Q	rock mass quality
RMR	rock mass rating
RQD	rock quality designation
γ	soil unit weight
δ	friction angle of the soil–pipe interface
μ	coefficient of soil–pipe friction
μ_{avg}	average back-analysed rock–pipe frictional coefficient

ν	Poisson's ratio
ΣF_y	reaction force resulting from finite-element analysis
σ'	effective normal stress
σ_{EV}	back-analysed effective vertical stress at the tunnel crown
σ'_n	effective normal stress acting on the rock–pipe interface
σ'_v	in situ effective overburden pressures
ϕ	soil internal friction
ϕ'	effective friction angle of the modelled geology
ϕ_i	friction angle of the rock–pipe interface
ϕ'_i	effective friction angle of the rock–pipe interface
ϕ'_{tp}	tangential peak friction angle of the tunnelling rock spoils
τ_1	shear stress at the rock–pipe interface resulting from finite-element analysis

Introduction

Rapid urbanisation has seen a rise in the need for buried infrastructure for the transportation of water and waste water. This is particularly so in developing countries, such as Malaysia. The construction of buried pipelines by using conventional open-trenching methods in densely populated urban centres is increasingly subject to restrictions (Stein, 2005a). Some of the socioeconomic and political–ecological adversities associated with open trenching include traffic diversions, construction noise and dust, damage to adjacent properties and complaints by residents and property owners. Hence, trenchless technologies can be a viable solution for the installation of buried pipelines in densely populated city centres. One such trenchless technology is microtunnelling by pipe-jacking.

As part of the Kuching Wastewater Management System Phase 1 project, the use of trenchless technology in the form of microtunnelling by pipe-jacking was essential in the construction

of 7.7 km of trunk sewerage pipelines beneath the central business district of Kuching City, Malaysia. The 1.2 and 1.5 m dia. reinforced concrete sewerage pipelines were constructed at depths of up to 25 m below the busy streets at the existing ground level, placing the pipelines well within the upper carboniferous Tuang Formation. The Tuang Formation is characterised by highly fractured, tightly folded lithological units of phyllite, shale, metagreywacke and sandstone (Tan, 1993).

Due to the highly fractured geology, the extraction of intact cores for rock strength testing was a challenge. The majority of recovered cores had a rock quality designation (*RQD*) of zero, where *RQD* is defined by the summation of pieces of sound core longer than 100 mm expressed as the percentage of the length of the core run (Deere and Deere, 1988: p. 92). Figure 1 shows phyllite cores recovered from the Tuang Formation, exhibiting very low *RQD* despite high core recovery. Similar observations of rock cores extracted from the Tuang Formation exhibiting low *RQD* were made by Ong and Choo (2011).

It is typically necessary to have intact rock core lengths or prismatic specimens in order to proceed with rock strength tests, such as the uniaxial compression strength test, point load test, Brazilian tensile test, triaxial test or shear wave velocity test (Bhasin *et al.*, 1995; Gurocak *et al.*, 2012; Ramamurthy *et al.*, 1993; Saroglou *et al.*, 2004; Shea and Kronenberg, 1993; Tiwari and Rao, 2007). However, the highly fractured Tuang Formation meant that it was difficult to attain sound rock core lengths, which subsequently created challenges in measuring the rock strength parameters. The availability of such rock strength parameters would have facilitated the assessment of jacking forces. Additionally, expertise for in situ pressuremeter testing of the rock mass was not locally available. In the absence of intact rock cores of suitable lengths, it was necessary to develop an alternative method of obtaining rock strength parameters.

The need to obtain useful rock mass strength parameters of the various lithological units found in the highly fractured Tuang Formation was the motivation for the current study. Such a finding

Figure 1. Cores of phyllite recovered from the highly fractured Tuang Formation, showing very low *RQD*

would lead to the possibility of assessing jacking forces in highly fractured geology. The majority of empirical jacking force models have been developed for pipe-jacking drives traversing soils (Bennett, 1998; Chapman and Ichioka, 1999; Olson *et al.*, 2016; Osumi, 2000; Staheli, 2006), with very limited considerations for drives through rock. Thus, an alternative approach comprising direct shear testing of reconstituted tunnelling rock spoils was developed for obtaining useful strength parameters. This novel method of subjecting tunnelling rock spoils to a shear box test was previously used by Choo and Ong (2012, 2014, 2015) and Ong and Choo (2016). The tunnelling rock spoils demonstrated non-linearity in their strength behaviour. By using a generalised tangential approach, the results from direct shear testing were applied to the well-established Pellet-Beaucour and Kastner (2002) jacking force model and interpreted based on the back-analysis of the rock–pipe frictional coefficient, μ_{avg} , and the vertical stress at the tunnel crown, σ_{EV} . The reliability of these back-analysed parameters was validated through subsequent numerical simulations of the studied pipe-jacking drives. This current study demonstrates the application of this back-analysis method on four case studies and how the outcomes of the back-analysis are related back to the geology surrounding the studied pipe-jacking drives.

Previous studies on jacking forces

The direct shear test has been used for various studies on soil–structure interfacial friction, ranging from fundamental soil mechanics (Jewell, 1986; Simoni and Houlsby, 2006) to its application on geotechnical engineering problems (Arulrajah *et al.*, 2013; Iai and Luna, 2011; Kulhawy and Mayne, 1990). The suitability of using a direct shear test for investigating jacking forces was demonstrated by Staheli (2006), who conducted direct shear testing of sand–pipe interfaces, based on a customised shear box developed by Iscimen (2004). Interfacial friction values were developed from these tests and used for the assessment of measured jacking forces from several pipe-jacking sites traversing sand. This led to the development of a jacking force model for unlubricated pipe-jacking drives in sands.

A separate study by Shou *et al.* (2010) comprised a concrete block being dragged over a soil mass. A layer of lubrication was introduced between the concrete block and sample soil, thus allowing the study to assess the effects of lubrication on pipe–soil interfacial friction. This effectively simulated the friction accrued in the overcut annulus of a lubricated drive. The interfacial frictional coefficients developed from these direct shear tests were used to study site measurements of jacking forces from drive sections in Taiwan. The test results were also applied to numerical simulations of the studied drive sections. The study found that the developed frictional coefficients overestimated the measured jacking forces. This discrepancy was due to full pipe–soil contact being assumed in the direct shear tests. In reality, the outer surface of the pipe would only be partially solicited in the accrual of jacking forces.

These earlier studies provided the motivation for Choo and Ong (2014, 2015) to develop a novel method of assessing jacking

forces by subjecting reconstituted tunnelling rock spoils to direct shear tests. This would allow for the development of equivalent rock strength parameters in the absence of intact rock core lengths. The results from direct shear tests were interpreted using a simple non-linear power law together with the generalised tangential method. These resulting tangential peak Mohr–Coulomb (MC) strength parameters c'_{lp} and ϕ'_{lp} were applied to the well-established jacking force model developed by Pellet-Beaucour and Kastner (2002).

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

where F is the total frictional jacking force; L is the pipe span; D_e is the outer pipe diameter; γ is the soil unit weight; K_2 is the thrust coefficient of soil acting on the pipe; and μ is the coefficient of soil–pipe friction, given in Equation 2

$$2. \quad \mu = \tan \delta$$

where δ is the friction angle of the soil–pipe interface. Stein (2005b) suggested values of μ for various states of friction between varying pipe materials and soils in pipe-jacking drives. The μ values of 0.1–0.3 were indicative of lubricated drives. σ_{EV} is the vertical soil stress at the pipe crown, which is given as

$$3. \quad \sigma_{EV} = \frac{b(\gamma 2C/b)}{2K \tan \phi} \{1 - \exp[-2K(h/b) \tan \phi]\}$$

$$4. \quad b = D_e \{1 + 2 \tan[(\pi/4)(\phi/2)]\}$$

where C is the soil cohesion; ϕ is the soil internal friction angle; K is the lateral earth pressure coefficient; h is the soil cover from the ground level to the pipe crown; and b is the influencing soil width above the pipe, as shown in Equation 4. The formulation of σ_{EV} is based on Terzaghi's estimation of vertical stresses acting at a tunnel crown, based on his classic trapdoor experiment (Terzaghi, 1936, 1943). The creation of an excavation resulted in the localised redistribution of stresses around the tunnel, resulting in a decrease in vertical stresses at the tunnel roof. This has a significant effect on accrued pipe-jacking forces, as the frictional jacking forces are a direct consequence of the normal soil stresses acting on the outer surface of the pipeline. Choo and Ong (2015) identified that the highly fractured Tuang Formation behaved as 'soft rock', in that it exhibited soil-like behaviour and demonstrated arching when subjected to tunnelling. Thus, the incorporation of the arching phenomenon into the Pellet-Beaucour and Kastner (2002) jacking force model made it suitable for the assessment of jacking forces in the soft rock of the highly fractured Tuang Formation. This was achieved by comparing the back-analysed parameters μ_{avg} and σ_{EV} against construction activities and the traversed geology.

For the validation of this method of back-analysis, Ong and Choo (2016) performed three-dimensional (3D) finite-element analysis of the studied pipe-jacking drives. The tangential peak MC strength parameters c'_{lp} and ϕ'_{lp} were applied to the model rock mass. The back-analysed rock–pipe frictional coefficient μ_{avg} was used as an input parameter for modelling the strength of the rock–pipe interface elements. The pipelines were modelled as 'wish-in-place' plate elements and propelled using prescribed displacements at the tail end of the pipe. A post-simulation arching ratio was applied to the resulting rock–pipe interface shear stresses τ_1 with the arching ratio AR given as

$$5. \quad AR(\%) = \frac{\sigma_{EV} + \gamma D_e/2}{\gamma(h + D_e/2)} \times 100\%$$

Through the use of this arching ratio, the interface shear stresses τ_1 and the reaction forces ΣF_y were found to be comparable against field measurements of the jacking forces. The successful simulation of the studied drives demonstrated the reliability of the method of back-analysis, through the use of direct shear testing on reconstituted tunnelling rock spoils. It also demonstrated that jacking forces accrued through pipe-jacking in highly fractured geology are significantly affected by the arching phenomenon due to the surrounding geology. In the current study, this method of assessing jacking forces is shown for four pipe-jacking drives traversing various lithological units in the Tuang Formation. The four studied drives were subsequently simulated using 3D finite-element modelling to verify the reliability of the direct shear test results – that is, c'_{lp} and ϕ'_{lp} – and the back-analysed parameters μ_{avg} and σ_{EV} .

Development of tangential rock strength parameters

Rock cores were retrieved during soil investigative works at the locations of launching and reception shafts along the alignment of the 7.7 km trunk sewer line. As previously stated, the RQD of the recovered rock cores was very low, owing to the highly fractured Tuang Formation. Of the 30.7 m of core lengths extracted, 13.4 m produced RQD values of zero, with the remaining core lengths being highly fractured. Hence, a novel method was developed, comprising direct shear testing of reconstituted tunnelling rock spoils. Reconstitution of rock fragments has been performed successfully by Jasinge *et al.* (2009) on Australian black coal, to allow for subsequent testing of the reconstituted material. In the current study, the reconstitution of the microtunnelling spoils would allow for the re-creation of the intensely fractured Tuang Formation, which is also characterised by irregularly sorted rocks with arbitrary joints (Tan, 1993). This geological situation is akin to the earlier description of soft rock.

Samples of tunnelling rock spoils were obtained from desanding machines located at the pipe-jacking sites for four different drives in the Kuching Wastewater Management System Phase 1 project. The sampled spoils were of various lithological origins typically characteristic of the Tuang Formation, namely metagreywacke,

sandstone, shale and phyllite. Petrographic images from thin sections of incomplete rock cores are shown in Figure 2. Prior to direct shear testing, the samples were prepared and scalped in accordance to ASTM D 3080-03 (ASTM, 2003) and AS 1289.6.2.2 (SA, 1998), whereby the largest particle size in the sample did not exceed 1/10 of the thickness of the test sample (Head, 1994). The particle size distributions of the samples before and after scalping are shown in Figure 3. The scalped samples were reconstituted through compaction in three layers within the shear box, achieving relative

densities of between 65 and 95%. These samples were subsequently subjected to direct shear testing at varying effective normal stresses, σ' . The shear box used had a diameter of 60 mm and a height of 30 mm. Table 1 shows the properties of the tested tunnelling rock spoils and the applied σ' for the direct shear tests.

Figure 4 shows the peak shear strength behaviour of the scalped reconstituted tunnelling rock spoils, interpreted from direct shear testing results. To allow for the subsequent assessment of the arching phenomenon, only peak results are shown as arching is instigated at low displacements. Tests 1, 2 and 4 on metagreywacke interbedded with siltstone, sandstone and phyllite, respectively, showed non-linearity in the strength behaviour of the reconstituted tunnelling rock spoils. This was particularly apparent for tests 1 and 2, where tests were performed at low levels of σ' – that is, below 100 kPa. For test 3 on shale, the strength behaviour was linear. Therefore, the simple elastic-perfectly plastic MC model was sufficient for characterising the strength behaviour of reconstituted shale spoils. However, for tests 1, 2 and 4, the use of regressive MC strength criteria was not suitable; as such, a strength model would lead to the overestimation of shear strength at low or high levels of normal stresses and underestimation at intermediate normal stress levels. Therefore, a simple power-type function was used, given as

$$6. \quad \tau = A(\sigma')^B$$

where A is a dimensionless constant that governs the magnitude of the power function and B is a dimensionless constant that governs the curvature of the power function. This power-type function has been successfully used in various studies for geotechnical applications (Anyaeibunam, 2015; Charles and Watts, 1980; De Mello, 1977; Lade, 2010; Soon and Drescher, 2007). The peak power law parameters A_p and B_p for the tested reconstituted tunnelling rock spoils are shown in Table 1.

Despite the use of power-type functions to acknowledge the non-linearity in the stress dependency of the shear strengths, MC parameters were still necessary for the back-analysis of jacking forces, as shown in Equations 3 and 4. The ‘generalised tangential’ approach, as introduced by Yang and Yin (2004), was used by applying tangential lines to the non-linear power law failure criterion for peak results at the respective in situ effective overburden pressures, σ'_v . The gradient of the tangential line was the tangential peak friction angle, $\phi'_{t,p}$. The intercept of the tangential line was the tangential peak cohesion, $c'_{t,p}$. These tangential peak parameters could then be used in the Pellet-Beaucour and Kastner (2002) jacking force model, as shown in Equations 7 and 8, respectively

$$7. \quad \phi = \phi'_{t,p} = \tan^{-1} \left(A_p B_p \sigma_v'^{B_p-1} \right)$$

$$8. \quad C = c'_{t,p} = A_p \sigma_v'^{B_p} (1 - B_p)$$

Figure 2. Petrographic images from thin sections of (a) metagreywacke, (b) shale and (c) phyllite

Figure 3. Particle size distribution of test samples: (a) before scalping; (b) after scalping to create reconstituted direct shear test samples

Table 1. Properties of reconstituted tunnelling rock spoils with applied effective normal stresses σ' for direct shear tests and interpreted power law and tangential parameters for peak results

Test Drive	1 A	2 B	3 C	4 D
Geology	Metagreywacke interbedded with siltstone	Sandstone	Shale	Phyllite
Relative densities: %	68–88	65–95	77–86	—
Applied σ' : kPa	25, 50, 75, 100, 380, 455, 530, 750	25, 50, 75, 100, 255, 600	50, 100, 250, 400	100, 250, 380, 400
A_p	3.17	4.68	—	1.92
B_p	0.79	0.76	—	0.87
In situ effective overburden σ'_v : kPa	204	112	108	162
$c'_{t,p}$: kPa	44.6	40.7	0	45.1
$\phi'_{t,p}$: °	39.9	49.8	41.3	46.2

The developed tangential peak MC parameters are shown in Table 1. These values would allow for the subsequent back-analysis of jacking forces measured from the studied pipe-jacking drives in the Tuang Formation.

Back-analysis of measured jacking forces: case studies

The assessment of jacking forces using tangential peak MC parameters in the jacking force model was for gauging the degree

of arching in the rock surrounding the tunnel. This was possible by using Equations 3 and 4 to produce values of the vertical stresses acting on the tunnel crown σ_{EV} for the respective pipe-jacking drives. Subsequently, this led to the development of values of the interfacial rock–pipe frictional coefficients μ_{avg} for the respective drives.

By comparing the developed μ_{avg} against μ values recommended by Stein (2005b), the state of lubrication in the overcut can be

Figure 4. Interpretation of peak strength behaviour of reconstituted tunnelling rock spoils using power law function and generalised tangential approach at in situ effective overburden pressures

assessed. The following case studies will demonstrate how the tangential peak MC parameters c'_{lp} and ϕ'_{lp} lead to the development of σ_{EV} and μ_{avg} and how these parameters are used to assess the measured jacking forces in relation to the traversed geology.

Figure 5 shows the changes in injected lubricant, jacking speeds, construction progress and jacking forces for the studied drives. Table 2 summarises the measurements made during pipe-jacking activities. Measurements of lubrication usage, jacking speed and jacking forces, JF_{meas} , were made at 1 m intervals along the pipe span. The results from back-analysis of the measured jacking forces are also presented and will be used to discuss the observations made during pipe-jacking as well as the influence of geology on the accrued jacking forces.

Drive A

Drive A was constructed using a Herrenknecht AVN 1200TC microtunnel boring machine (MTBM). This 39 m span traversed metagreywacke interbedded with siltstone. No lubricant was injected into the overcut annulus between the outer surface of the concrete pipe and the surrounding geology. The jacking speed was fairly consistent, averaging at 17 mm/min. This was evident

by the steady progress of the drive. The measured jacking forces JF_{meas} was 20.7 kN/m. From the tangential peak parameters of $c'_{lp} = 44.6$ kPa and $\phi'_{lp} = 39.9^\circ$, the back-analysed σ_{EV} was -19.9 kPa. The negative value of σ_{EV} implied that tensile vertical soil stresses were acting on the pipe crown. However, such a stress state is not possible due to the presence of the overcut annulus. Terzaghi (1943) stated that in such cases, the vertical stresses at the tunnel roof were equal to zero. This also indicated that significant arching had occurred in the surrounding geology. This would lead to reduced frictional jacking forces. However, the back-analysed μ_{avg} was 0.491, which exceeded the upper limit value of $\mu = 0.3$ suggested for lubricated drives (Stein, 2005b). This was expected since the drive was not lubricated. The assessment of jacking forces for drive A demonstrated that the method of back-analysis was reasonable for unlubricated drives.

Drive B

Drive B was also constructed using the Herrenknecht AVN 1200TC MTBM. The drive spanned 140 m through sandstone. Lubricant was injected at a rate of 47 l/m into the theoretical overcut of 87 l/m, suggesting that a discrete layer of lubricant had not formed around the outer periphery of the pipeline. This

Figure 5. Variation in lubricant use, jacking speeds, construction progress and measured jacking forces for the studied drives

Table 2. Field measurements of pipe-jacking activities and outcomes from back-analysis of measured jacking forces JF_{meas}

Test	1	2	3	4
Drive	A	B	C	D
Geology	Metagreywacke–siltstone	Sandstone	Shale	Phyllite
Drive span: m	40	140	140	120
D_e : m	1.78	1.43	1.43	1.78
Volume of lubricant injected: l/m ^a	0	47	682	181
Theoretical overcut volume: l/m ^a	87	87	87	113
Jacking speed: mm/min ^a	17	16	10	44
JF_{meas} : kN/m ^a	20.7	14.4	29.0	4.8
c'_{tp} : kPa	44.6	40.7	0	45.1
ϕ'_{tp} : °	39.9	49.8	41.3	46.2
σ_{EV} : kPa	−19.9 ^b	−14.5 ^b	28.9	−14.4 ^b
μ_{avg} ^a	0.491	0.365	0.235	0.080

^a Quantities averaged from field measurements made intervals of 1 m along the driven span

^b Negative values of σ_{EV} were adjusted to be zero (Terzaghi, 1943)

suggested that the lubrication was not fully effective. Jacking speeds were fairly steady, averaging at 16 mm/min.

The jacking forces were measured at $JF_{meas} = 14.4$ kN/m. From direct shear tests on spoils of sandstone, the tangential peak parameters were $c'_{tp} = 40.7$ kPa and $\phi'_{tp} = 49.8^\circ$. This led to the back-analysed σ_{EV} of -14.5 kPa, suggesting that there was a significant degree of arching occurring in the sandstone around the driven pipeline. The resulting μ_{avg} was 0.365, which was slightly above the upper limit of 0.3 for lubricated drives. Despite the significant arching, the lubrication had not been fully

mobilised, thus resulting in the back-analysed μ_{avg} of 0.365, which was larger than 0.3, but smaller than the unlubricated drive A, for which the back-analysed μ_{avg} was 0.491.

Drive C

Drive C traversed the geology of shale origins and spanned 140 m. The drive was constructed using an Iseki Unclemole Super TCS 1200 MTBM. The theoretical overcut for drive C was 87 l/m. However, the amount of lubricant injected into the overcut was 682 l/m, which was 684% in excess of the theoretical overcut. This suggested that a significant amount of injected

lubricant was lost into the surrounding geology. Jacking speeds were as high as 45 mm/min at the beginning of the drive, before slowing down to as low as 1.4 mm/min near the end of the drive. The jacking speeds and lubricant usage are a response by the construction team towards the accrued jacking forces. Thus, the fluctuations in jacking speed were indicative of the erratic and excessive lubrication patterns.

The measured jacking forces JF_{meas} was 29.0 kN/m. Direct shear testing on the spoils of shale revealed that the tested material behaved as an MC material; thus, the strength parameters were $c'_{\text{lp}} = 0$ and $\phi'_{\text{lp}} = 41.3^\circ$. The resulting vertical stress at the pipe crown was $\sigma_{\text{EV}} = 28.9$ kPa. This was significantly larger than the σ_{EV} values corresponding to drives A and B previously, thus indicating a reduced degree of arching. The compromise in the arching phenomenon was due to the absence of cohesion in the strength of the tested shale spoils. This resulted in a back-analysed μ_{avg} of 0.235, which was within the range suggested for lubricated drives – that is, $0.1 < \mu < 0.3$ (Stein, 2005b). Thus, the drive can be regarded as being well lubricated, which corroborates with the intense lubricating effort. However, this came at a huge expense since the injected lubricant was at 684% in excess of the overcut volume.

Drive D

Drive D used the Iseki Unclemole Super TCS 1500 MTBM for traversing 120 m of phyllite. Lubrication was initiated at 45 m of the drive progress. Across the entire drive, an average of 181 l/m of lubricant was injected, which was slightly in excess of the theoretical overcut volume of 113 l/m. Jacking progressed steadily at an average speed of 44 mm/min. This was significantly faster than pipe-jacking in the earlier drives, suggesting that the lubrication had been effective. This was validated by the measured jacking forces of $JF_{\text{meas}} = 4.8$ kN/m, as the low jacking forces allowed the construction team to jack at higher speeds. From direct shear testing of phyllite spoils, the tangential peak MC parameters were $c'_{\text{lp}} = 45.1$ KPa and $\phi'_{\text{lp}} = 46.2^\circ$. This led to a back-analysed σ_{EV} of -14.4 kN/m², suggesting that there was significant arching occurring in the phyllite around the tunnel. This led to a back-analysed frictional coefficient of $\mu_{\text{avg}} = 0.080$, implying that drive D was well lubricated. This confirms the earlier observation that the lubrication was effective.

The tightly folded phyllite is characteristically bedded, tightly folded and highly sheared (Tan, 1993). The tight folding could have created a stable bore, which facilitated the development of arching as well as reducing the presence of ‘thief zones’ through which lubricant could have been lost into the surrounding rock mass (Choo and Ong, 2015). This led to the development of a discrete lubricating layer in the overcut region, ensuring that the lubricant was effectively utilised.

Summary of drives

For the assessment of jacking forces in the case studies presented earlier, tangential peak MC strength parameters developed from direct shear testing on non-linear tunnelling rock spoils were

applied to the Pellet-Beaucour and Kastner (2002) jacking force model. The resulting back-analysed parameters σ_{EV} and μ_{avg} were developed for each drive and explained in relation to the pipe-jacking activities as well as the traversed geology. Drive A was an unlubricated drive to provide a benchmark of a ‘dry’ drive for the back-analysed μ_{avg} . The remaining drives were all lubricated; however, the effectiveness of the injected lubrication was affected by the geology.

Drives B, C and D showed that the traversed geology can affect the state of lubrication in the overcut, subsequently affecting the jacking speeds, construction progress and measured jacking forces, JF_{meas} . The geology also had a significant effect on the degree of arching, with drives in interbedded metagreywacke–sandstone, sandstone and phyllite showing significant arching, while drives in shale demonstrated reduced arching. This subsequently affected the normal soil stresses acting on the outer periphery of the driven pipelines and thus affected the back-analysed μ_{avg} values.

To verify the applicability of the direct shear test results in the assessment of jacking forces in highly fractured geology, 3D finite-element analysis is performed as follows.

Three-dimensional numerical analyses of pipe-jacking forces

The 3D numerical modelling of the studied pipe-jacking drives was simulated using the commercially available Plaxis 3D (Plaxis bv, 2013) in order to validate the applicability of direct shear test results to the assessment of jacking forces. The modelling technique described here was used by Ong and Choo (2016) to assess the jacking forces measured from highly fractured geology. Figure 6 shows a typical model used for simulating the studied pipe-jacking drives.

Figure 6. Connectivity plot of typical model used for 3D numerical analysis of studied drives

The boundaries of the finite-element model measured 30 m in the x direction, 20 m in the y direction and 30 m deep in the z direction. Standard fixity conditions were applied to the model boundaries, consisting of the full fixity at the bottom face of the model and rollers applied to the vertical faces of the model to restrict movements in horizontal directions. Details of the modelling technique are described below.

Rock properties

The linear elastic-perfectly plastic MC failure criterion was used to model the constitutive behaviour of the modelled rock masses. Despite the availability of higher-order soil and rock models, the MC failure criterion was chosen due to the development of tangential peak parameters from earlier direct testing. Hence, the use of the MC failure criterion would allow for equivalence between the parameters used for back-analysis of jacking forces (see Equations 3 and 4) and the parameters used in the simulation of the pipe-jacking drives – that is, $c' = c'_{lp}$ and $\phi' = \phi'_{lp}$.

The stiffness values of the modelled rock masses were estimated based on the following equations (Bieniawski, 1989)

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

$$10. \quad RMR = 9 \ln Q + 44$$

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 (Barton *et al.*, 1974). Q is a function of RQD , for which $RQD = 0$ was assumed. This produced a rock modulus value of $E_M = 7.1$ GPa. All the modelled rock mass parameters are shown in Table 3 for the respective studied drives. The soil generally comprised quaternary deposits, ranging from very soft clays to dense sand. However, the purpose of modelling the soil layers was to provide overburden pressure to the modelled rock mass. Hence, the unit weight of the soil layers was 18 kN/m^3 . This was appropriate as the pipelines were wish-in-place, and the effects of arching were considered post-simulation.

Pipeline properties, rock-pipe interface properties and prescribed displacements

The connectivity plot of the structural elements in the modelled pipeline is shown in Figure 7. The pipelines were modelled using plate elements and were aligned 20 m longitudinally in the y direction. The pipelines penetrated through the model; therefore, the ends of the modelled pipelines were flush with the vertical model boundaries on the x - z plane. The jacking of the modelled pipelines was performed by applying prescribed displacements at the tail end of the pipeline while simultaneously freeing the fixities at both ends of the modelled pipelines. Prescribed

Figure 7. Connectivity plot of modelled pipeline, rock-pipe interface elements and prescribed displacements

Table 3. Material parameters for modelling rock mass, rock-pipe interface and pipe for the studied drives

Drive		A	B	C	D
Geology		Metagreywacke-sandstone	Sandstone	Shale	Phyllite
Rock mass parameters	γ : kN/m^3	22	22	22	22
	c' : kPa	44.6	40.7	0	45.1
	ϕ' : $^\circ$	39.9	49.8	41.3	46.2
	E_M : kN/m^2	7.1×10^6	7.1×10^6	7.1×10^6	7.1×10^6
	ν	0.35	0.35	0.35	0.35
Rock-pipe interface parameters	c' : kPa	0	0	0	0
	ϕ' : $^\circ$	26.2	20.1	13.2	4.6
	E' : kN/m^2	1000	1000	1000	1000
	ν	0.35	0.35	0.35	0.35
	E' : kN/m^2	27×10^6	27×10^6	27×10^6	27×10^6
Pipe parameters	ν	0.15	0.15	0.15	0.15
	D_e : m	1.78	1.43	1.43	1.78

displacements were applied uniformly along the pipe periphery at the tail end of the pipe. The modelled pipeline was displaced 100 mm in the y direction. The effect of prescribed displacements on boundary fixities would be nullified, unless boundary fixities were deactivated. Therefore, it was necessary to free the fixities at both ends of the modelled pipeline to allow for the forward displacement of the pipe. The modelled pipelines were modelled as rigid elements, since the pipelines were much stiffer than the surrounding rock–pipe interface. Furthermore, this would facilitate the segregation of pipeline axial loads and pipeline compression from the frictional jacking forces and allow for parity in assessing the rock–pipe interface shear stresses against the back-analysed jacking forces and measured jacking forces, JF_{meas} . The properties of the modelled pipeline are presented in Table 3 for the respective studied drives.

The rock–pipe interfaces were modelled using interface elements. The strength behaviour of the rock–pipe interface elements was modelled using a cohesionless MC failure criterion, whereby

$$11. \quad \tau_1 = \sigma'_n \tan \phi'_i$$

$$12. \quad \phi'_i = \tan^{-1}(\mu_{\text{avg}})$$

given that τ_1 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-analysed average frictional coefficient, obtained from the earlier back-analyses of jacking forces. Additionally, it was necessary to use an appropriately low value for the interface modulus, E_{inter} . This was to allow for the pipe–rock interfaces to deform, thus facilitating the simulation of large pipeline displacements in pipe-jacking. In a study on concrete–soil interfaces, Yin *et al.* (1995) suggested an interface tension modulus of 1 kPa to allow sliding at the interface. In a simulation of pull-out tests, Wang and Wang (2006) used a low interface modulus of $E_t = 17.4$ kPa to model large deformations. These values were initially assigned to the pipe–rock interface elements in the current study. However, these suggested values prevented the numerical convergence of the models. Therefore, the pipe–rock interface elements were prescribed an interface modulus, E_{inter} , of 1000 kPa.

Modelling of pipe-jacking process using prescribed displacements

The modelling of the pipe-jacking process was performed in three sequential stages

- stage 1: generation of initial stresses and standard boundary fixities
- stage 2: activation of wish-in-place model pipeline and rock–pipe interface elements, together with deactivation of rock elements within pipeline annulus

- stage 3: prescription of displacements at tail end of modelled pipeline, with deactivation of fixities at both ends of modelled pipeline.

Before any simulations were performed, it was necessary to initiate the model by generating greenfield conditions. Therefore, in stage 1, this was achieved through the prescription of soil material properties, groundwater conditions, K_0 conditions and standard fixities at the model boundaries. At this stage, elements necessary for subsequent modelling of the pipeline and rock–pipe interface were also present: plate and interface elements. However, these elements would remain deactivated in stage 1.

After the initiation of greenfield stresses, the plate elements and interface elements were activated, representing the wish-in-place of the model pipeline and rock–pipe interface, respectively. The modelling of the pipeline was wish-in-place in order to simulate ongoing pipe-jacking construction. Therefore, soil elements within the pipe annulus were deactivated. Water conditions were also changed to ensure that the region within the pipe annulus was dry.

In stage 3, for the propulsion of the modelled pipeline forward, displacements were applied at the tail end of the modelled pipeline. Displacements were applied in the y direction, in the direction of the longitudinal axis of the modelled pipeline. Simultaneously, boundary fixities were released at both ends of the modelled pipeline.

Post-simulation consideration of arching effect

The modelling of the pipeline as wish-in-place implied that the effect of arching was not considered during the numerical analysis of the drives. The horizontal displacement of the rigid modelled pipelines did generate useful results; however, the rigid pipelines implied that stresses acting on the modelled pipelines did not induce significant deformations in the pipe, thus preventing arching from taking place in the models.

In order to include the effect of arching on the results from the numerical models, an arching ratio proposed by Ong and Choo (2016) was used: Equation 5. This aforementioned arching ratio considers the changes in vertical stresses; therefore, arching in both transverse and longitudinal directions caused by the tunnelling was not considered. The arching ratio, A_R , was subsequently applied to the results of numerical modelling, namely, the interface shear stresses τ_1 and reaction forces ΣF_y , leading to Equations 13 and 14

$$13. \quad JF_{\tau, \text{cal}} = A_R \times \tau_1 \pi D_e$$

$$14. \quad JF_{F_y, \text{cal}} = A_R \times \sum F_y / L$$

where $JF_{\tau, \text{cal}}$ are the jacking forces calculated from τ_1 ; $JF_{F_y, \text{cal}}$ are the jacking forces calculated from ΣF_y ; and L is the drive span.

The results from numerical analysis with consideration of arching effect are discussed next.

Discussion of results from numerical analysis

Figure 8 shows the distribution of pipe-rock interface shear stresses, while Figure 9 shows the radial distribution of τ_1 resulting from numerical modelling of the pipe-jacking drives. Table 4 shows these results from the numerical analysis, together with the outcomes from post-simulation consideration for arching effect $JF_{\tau,cal}$ and $JF_{Fy,cal}$. Despite some irregularities at the ends of the modelled pipelines due to end effects, the results showed that the distribution of τ_1 was uniform all along and around the pipelines for all the studied drives.

This implied that the interface elements had yielded, allowing for all points along the rock-pipe interface to achieve the same shear

Figure 8. Longitudinal distribution of pipe-rock interface shear stresses, τ_1 , along modelled pipeline

Figure 9. Radial distribution of pipe-rock interface shear stresses τ_1 around modelled pipeline

stresses. It also implied that the prescribed displacements of 100 mm were sufficient for attainment of yield. Additionally, the achievement of plasticity of the interface elements can be easily verified by computing the interface shear strength by using Equations 11 and 12 with the respective in situ effective overburden, σ'_v , from Table 1. This is an important result as the Pellet-Beaucour and Kastner (2002) jacking force model shows that frictional jacking forces increase at a linear rate with the pipeline length – that is, the frictional stresses around the outer surface of the pipeline are uniform. Therefore, there is parity between the jacking force model and the numerical analysis.

The jacking forces calculated from τ_1 , $JF_{\tau,cal}$, were compared against the measured jacking forces, JF_{meas} , with the variation ranging between 15 and 25%. This showed reasonable agreement between the pipe-rock interface stresses and the measured jacking forces. To verify further the results from numerical modelling, jacking forces calculated from ΣF_y , $JF_{Fy,cal}$, were also compared against measured jacking forces. A similarly reasonable agreement was observed between the reaction forces, ΣF_y , and measured jacking forces, with variations ranging between 15 and 29%. The difference in variations was assessed, given as

$$15. \quad JF_{\tau,cal} - JF_{Fy,cal} / JF_{meas} \times 100$$

The differences between the jacking forces calculated using both τ_1 and ΣF_y were minimal, ranging between 0 and 5%. Since τ_1 was measured along the modelled pipeline and ΣF_y values were the total reaction forces acting on the prescribed displacement, the difference would indicate the influence of end effects. With a minimal difference, this suggested that end effects were negligible, thus validating the modelling technique. It also showed that τ_1 and ΣF_y were in static equilibrium; thus, the stress fields in all the modelled drives were statically admissible. With the yielding of the interface elements as described earlier, the numerical analysis together with the earlier back-analysis of jacking forces are considered lower-bound solutions. This was possibly due to the use of reconstituted tunnelling rock spoils, which were crushed during pipe-jacking, thus losing a significant portion of cementation. Furthermore, measured quantities such as jacking speed, lubricant use and jacking forces were averaged across the respective pipeline spans, when these quantities typically fluctuate, as shown in Figure 5. This further contributed towards the lower-bound solution, resulting in the underestimation of measured jacking forces during numerical modelling.

Conclusions

For the construction of trunk sewers in the Kuching Wastewater Management System Phase 1 project, microtunnelling by pipe-jacking was the preferred technique. The highly fractured Tuang Formation implied that the extraction of intact cores for rock strength characterisation was not feasible. A novel method developed by Choo and Ong (2015) was used to obtain equivalent rock strength parameters from reconstituted tunnelling rock spoils

Table 4. Outcomes from numerical modelling of studied pipe-jacking drives

Drive	A	B	C	D
Geology	Metagreywacke–siltstone	Sandstone	Shale	Phyllite
JF_{meas} : kN/m	20.7	14.4	29.0	4.8
Tunnel depth h : m	20	12.5	12.5	18.5
D_e : m	1.78	1.43	1.43	1.78
A_R : %	3.75	6.22	17.94	5.34
τ_1 : kN/m ²	104	44	27	14
$JF_{\tau, \text{cal}}$ ^a : kN/m	17.5	12.2	22.0	4.1
Variation using τ_1 : %	–15.4	–15.2	–24.2	–14.9
ΣF_y : kN	8759	3685	2294	1544
$JF_{F_y, \text{cal}}$ ^b : kN/m	16.4	11.5	20.6	4.1
Variation using ΣF_y : %	–20.8	–20.6	–29.0	–14.9
Difference in variations: ^c %	5.4	5.4	4.8	0

^a From Equation 13

^b From Equation 14

^c From Equation 15

by using direct shear testing. For the application of the test results to a well-established jacking force model, it was necessary to use the generalised tangential on the test results, resulting in the development of tangential peak parameters c'_{tp} and ϕ'_{tp} . By applying these strength parameters to four pipe-jacking drives in varying geology, back-analysed parameters μ_{avg} and σ_{EV} were developed and used to explain the accrual of jacking forces and the variation in construction activities in the studied drives. It was found that geology had significant effects on lubricant and arching around the tunnel, subsequently affecting the accrued jacking forces in each drive.

The applicability of c'_{tp} , ϕ'_{tp} , μ_{avg} and σ_{EV} to the back-analysis of jacking forces was validated through the use of 3D finite-element modelling of the studied drives. The models used rigid modelled pipelines and interface elements of very low stiffness to facilitate the longitudinal propulsion of the modelled pipelines. By using the earlier developed strength parameters and back-analysed parameters as input for the finite-element models, the resulting interface shear stresses, τ_1 , and reaction forces, ΣF_y , were compared against measured jacking forces. This was possible through the use of arching ratios derived from the tangential rock strength parameters to match the measured jacking forces. The results from numerical modelling showed reasonable agreement with the measured jacking forces.

Therefore, the use of direct shear testing on reconstituted tunnelling rock spoils can be a potentially practical option for assessing jacking forces in highly fractured, highly weathered soft rock material. This could lead to the development of a predictive jacking force model for drives in soft rock formations, such as Tuang Formation.

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