Evaluation of pipe-jacking forces based on direct shear testing of reconstituted tunneling rock spoils

C S Choo[1]; D E L Ong[2]

Abstract: The installation of underground trunk sewer lines in the Tuang Formation of Kuching City, Malaysia utilized trenchless technology in the form of the pipe-jacking method. The evaluation of pipe-jacking forces mainly involves empirical models developed for soils, with rather limited considerations for drives through weathered rock. Therefore, a novel approach is proposed to evaluate strength parameters by reconstituting and subsequently shearing scalped tunneling rock spoils in the direct shear apparatus. The direct shear results are then applied to a well-established pipe-jacking force model, which considers arching theory. The outcomes indicate that the back-analyzed frictional coefficients, μ_{avg} are not only reliable but also related to their surrounding geologies due to soil-structure interaction. This study also highlights the significance of lubrication and effect of rock arching in assessing jacking forces. The successful characterization of reconstituted tunneling rock spoils in this paper has shown potential use in assessing jacking forces during micro-tunneling works.

Keywords: Pipe-jacking; Trenchless technology; Friction; Shear strength; Direct shear; Rocks; Arching effect; Power law; Non-linear

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[1]Associate Lecturer, Research Centre for Sustainable Technologies, Faculty of Engineering, Science & Computing, Swinburne University of Technology Sarawak Campus, 93350 Kuching, Sarawak, Malaysia. E-mail: cschoo@swinburne.edu.my

[2]Director (Acting), Research Centre for Sustainable Technologies, Faculty of Engineering, Science & Computing, Swinburne University of Technology Sarawak Campus, 93350 Kuching, Sarawak, Malaysia. E-mail: elong@swinburne.edu.my

INTRODUCTION

Pipe-jacking may have become the preferred delivery method over conventional open trenching for installation of buried infrastructure, largely due to the mitigated disturbances to ground surface, as well as the reduced disruptions to road traffic. Such trenchless technology methods involving micro-tunneling can be implemented in the form of pipe-jacking. Pipe-jacking works in the central business district of Kuching City, Malaysia were necessary as trunk sewer lines were proposed to be installed in depths up to 25 m from the existing ground level. The pipe-jacking works traversed the pre-upper carboniferous Tuang Formation, characterized by highly fractured and tightly folded phyllite, with highly fractured lithological units of shale, metagreywacke, and sandstone (Tan, 1993). The relatively young and weathered geological formation created challenges during extraction of rock cores in soil investigation (SI) works, particularly from the argillaceous units of shale and phyllite. The SI works revealed that majority of the recovered cores of shale and phyllite had a Rock Quality Designation (RQD) value of zero. RQD is defined as the total length of recovered cores longer than 100 mm expressed as a percentage of the total rock core length (Deere, 1989). The low RQD values implied a lack of suitable core lengths for uniaxial compression strength (UCS) and point load testing. This made it difficult to assess the in-situ weathered rock strength parameters as the preferred in-situ pressuremeter tests were not readily available at that point in time. Furthermore, local expertise in the industry in performing the in-situ pressuremeter tests was very limited. Unfortunately, pressuremeter tests were not originally budgeted for in the investigation stage of the project.

This study was motivated by the possibility of using tunneling rock spoils for the purpose of back-analyzing pipe-jacking forces during the construction of trunk sewer lines in Kuching City, Malaysia (Choo and Ong, 2012). Rock spoils from tunneling works were collected from four different pipe-jacking drives. Direct shear testing of reconstituted tunneling rock spoils was used as an alternative method of obtaining rock strength parameters so that pipe-jacking forces could be reliably understood and back-analyzed. Results from direct shear tests were applied to a well-established empirical pipe-jacking force equation. The calculated pipe-jacking forces with respect to drive lengths were compared to corresponding measured values. It has been found that the back-analyzed values of μ agreed reasonably well with the suggested range of μ = 0.1–0.3 considering lubrication (Stein et al., 1989) and must be explained with reference to their surrounding geologies due to soil-structure interaction.

DIRECT SHEAR TESTING

The direct shear test has been used to determine shear strength properties of various soil-structure interfaces. Such applications of direct shear testing range from fundamental studies on the effects of dilatancy (Simoni and Houlsby, 2006), particle size (Yu et al., 2006) and shear box platen fixities (Jewell, 1986) to field applications including skin friction generated between various construction materials (Potyondy, 1961), geosynthetic-reinforced structures (Anubhav and Basudhar, 2013; Arulrajah et al., 2013), large-scale hydropower projects, and excavations of lunar regoliths (Iai and Luna, 2011). Direct shear tests can be conducted under conditions of constant normal load (CNL), constant normal stiffness (CNS), or constant volume (CV) (Pellet and Keshavarz, 2014). The direct shear tests in this current study were conducted under CNL conditions to reflect the in-situ stress conditions along the drives.

The direct shear test has also been used for studies on pipe-jacking forces. The effect of surface roughness for some common jacking pipe materials was studied by displacing the top half of a shear box along the outer periphery of the studied pipes (Staheli, 2006). The shear box was filled with sand and respective frictional coefficients for the respective pipe materials were produced. The frictional coefficients were subsequently applied to back-analyses of measured jacking forces from actual pipe-jacking drives. A frictional jacking force model was developed from the back-analyses; however this force model was developed for unlubricated drives traversing clays and sands only.

A separate study on the effect of lubrication using a modified shear box, involves the dragging of a concrete block over a large sample of soil (Shou et al., 2010). Lubricant was placed between the concrete block and the soil. Measurements were made of the critical drag force required to move the concrete block, with variations made to the lubricant type and applied normal force on the concrete block. An unlubricated condition was used as a benchmark. The results showed that the lubricant mix of plasticizers with polymer fluid reduced the pipe-soil interface friction by 75%, due to the discrete layer of plasticizer present between the concrete block and soil. The findings were subsequently applied to a case study of a 2.85 m diameter pipe of length 400 m jacked through gravel at a depth of 9.65 m below the ground surface. There was a large discrepancy between the back-analyzed jacking forces against those resulting from the modified direct shear test. This discrepancy was attributed to the overestimation of the pipe-soil contact area.

Therefore, in view of limited studies on pipe-jacking in relatively young, weathered sedimentary and metamorphic rocks, this paper makes a novel attempt to effectively quantify jacking forces via a systematic approach of utilizing reconstituted rock spoils tested in a direct shear apparatus.

EMPIRICAL METHOD TO DETERMINE JACKING FORCES

Some commonly used equations for predicting forces in pipe-jacking works through soil have been derived statistically (Chapman and Ichioka, 1999), empirically (Osumi, 2000), or experimentally (Staheli, 2006). Very limited considerations are made for drives in rock. In empirically derived jacking force models, two approaches are used for assessing frictional jacking forces.

The first considers the soil stresses acting on the outer periphery of the pipeline. This approach is typically dependent on the strength properties of the geology surrounding the pipe. However, the usage of such equations is often restricted to drives traversing sands or clays, with limited experience for tunneling through weathered rocks. The evaluation of jacking forces through weathered rocks can be made further complicated due to the surrounding highly fractured rock mass or, in contrast, the presence of an arching effect.

Pellet-Beaucour and Kastner (2002) developed a jacking force equation, which incorporated the geomechanical phenomenon of soil arching. Terzaghi (1936) observed this phenomenon through the introduction of a trap door beneath a sand mass. Opening the trap door disturbed the geostatic stresses, thus inducing a relaxation of soil stresses above the trap door. Such phenomenon is also evident in rock tunnels. Hence, the model developed by Pellet-Beaucour and Kastner (2002) was chosen for use in this paper due to its inclusion of the soil arching effect. This jacking force model is shown in Eq. (1), which is expressed as the vertical soil stress at the pipe crown, σ_{EV} shown in Eq. (2).

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

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

where F = total frictional jacking force; L = pipe span; D_e = outer pipe diameter; γ = soil unit weight; h = soil cover from the ground level to the pipe crown; K = lateral earth pressure coefficient; K_2 = thrust coefficient of soil acting on the pipe, with a suggested value of 0.3 (French Society for Trenchless Technology, 2006); C = soil cohesion; and ϕ = soil internal friction angle. C and ϕ are derived from the in-situ rock mass above the pipe, where arching would develop. b is the influencing soil width above the pipe and is expressed in Eq. (3) as

$$b = D_e\left(1 + 2\tan\left(\frac{\pi}{4} - \frac{\varphi}{2}\right)\right) \qquad (3)$$

The formulation of σ_{EV} is based on the classic limit equilibrium approach developed by Terzaghi (1943), which was dependent on the Mohr-Coulomb (MC) strength parameters of the yielding soil.

During pipe-jacking works, tunnel boring creates changes to the geostatic stresses, resulting in soil relaxation around the tunnel and pipe. This phenomenon redistributes soil stresses around the bored tunnel, allowing the tunnel to self-stand through the phenomenon of arching. The result is a reduction of vertical stresses experienced at the pipe crown, σ_{EV}.

μ is the coefficient of pipe-soil friction, given in Eq. (4) as

$$\mu = \tan\delta \qquad (4)$$

where δ = pipe-soil friction angle.

For Eq. (1), values of μ between 0.1 – 0.3 were recommended for lubricated drives or ‘fluid friction’ (Stein et al., 1989). These values for lubricated drives were previously used for the prediction of jacking forces during tunneling in sands and clays (French Society for Trenchless Technology, 2006; Pellet-Beaucour and Kastner, 2002), as well as limestone (Barla et al., 2006). Pellet-Beaucour and Kastner (2002) reported μ values below 0.1, which also indicated adequate lubrication. The recommended frictional coefficients for lubricated drives assume ideal lubricating conditions, whereby a distinct lubricating layer is formed between the jacked pipeline and the surrounding geology.

The use of these jacking force models is still dependent on the surrounding rock strength characteristics and pipe-rock interface properties, which are difficult to assess in this Kuching City study due to the friability of the extracted rock cores from the Tuang Formation. A novel approach of acquiring the representative rock strength properties from reconstituted tunneling rock spoils for the assessment of jacking forces is thus studied hereinafter.

PROPOSED METHOD OF TESTING RECONSTITUTING ROCK SPOILS

Rock cores obtained from the SI works at the launching and retrieving shaft locations along the Tuang Formation in Kuching City revealed very low RQD values. The average RQD value was 17% for rock cores obtained from the studied drives. In total, 30.7 m of core lengths were extracted, of which 13.4 m produced RQD values of zero. A majority of these extracted cores were highly fractured and not fully intact.

Similar observations were made by Ong and Choo (2011) for a separate project sited also in the Tuang Formation. In this case study, 206 m of rock cores were extracted, of which 151 m had RQD values of zero. This provided challenges in conducting UCS tests and consequently difficulties in obtaining useful, consistent strength values.

In other microtunneling projects, it may have been possible to survey the rock faces exposed in more detail during shaft construction (Barla et al., 2006). In this study, unfortunately exposed rock faces were quickly shotcreted as stand-up time was critical to prevent water ingress, which would have resulted in ground surface deformations. Therefore, joint directions of the rocks could not be further studied.

Furthermore, tests for determining strength anisotropy (e.g. UCS, Brazilian tensile test, triaxial test, point load test, shear wave velocity) are usually conducted on intact cylindrical cores (Bhasin et al., 1995; Desai and Salami, 1987; Gurocak et al., 2012; Saroglou et al., 2004; Shea Jr and Kronenberg, 1993; Yilmaz, 2009) or prismatic specimens (Ramamurthy et al., 1993; Singh et al., 2002; Tiwari and Rao, 2007). As a result of the challenges in obtaining natural intact rock cores of suitable lengths, and also due to the friability of the in-situ rock, it was not practically possible to ascertain the anisotropy of the rock mass.

Thus, a novel method to test and assess the reconstituted micro-tunneling rock spoils was initially developed for a typical pipe-jacking drive in Kuching, Malaysia (Choo and Ong, 2012). Literature review shows that the reconstitution of rock fragments has been successfully conducted on Australian black coal in order to homogenize the high variability of its mechanical properties (Jasinge et al., 2009). Cement was used to stabilize the coal samples. UCS and PLT tests were conducted on the reconstituted stabilized coal. The testing results indicated reasonable homogeneity of the strength of the reconstituted coal. This particular study successfully showed good potential in obtaining useful strength parameters and correlations for reconstituted rock samples. This fundamental understanding was applied hereinafter to allow for direct shear testing of tunneling rock spoils to obtain the relevant shear strength properties.

In the current study, as the in-situ rock mass is naturally friable due to deep weathering in a hot and humid climate (Malaysia is located on the Equator) (Tan, 1993), the concept of reconstituting the excavated spoils into a shear box is to 're-create' the situation of intensely fractured, irregular and poorly sorted rocks with many arbitrary joints or fine cracks found on the surfaces of the in-situ samples. This could be described as a highly weathered 'soft rock' which perhaps behaves more closely to soil, and hence the possible use of 'soil' equations as shown in Eqs. (1) and (2). If proven reliable, research into the properties of reconstituted tunneling rock spoils could provide a platform for consistent prediction of jacking forces accrued during pipe-jacking works in highly fractured geology.

Characteristics of test samples

Petrographic analyses of thin sections were conducted on rock cores obtained from pipe-jacking shaft locations. Fine-grained sandstone (metagreywacke) (see Fig. S1(a)) was composed mainly of angular to subangular quartz grains. X-ray diffraction (XRD) tests conducted on the samples of sandstone further validated the presence of quartz. Grains of shale were mainly composed of clay minerals and silt-sized quartz grains, with tiny flakes of mica (see Fig. S1(b)). Metamorphic phyllite was mainly composed of fine-grained quartz with flaky sericite and mica (see Fig. S1(c)). The grains in phyllite were finer than those found in shale. XRD tests did not reveal the presence of bentonite slurry, if any. (Note that petrographic images can be found in Fig. S1 in the Supplemental Data.)

Test samples of tunneling rock spoils were obtained from desanding machines or decantation chambers at four pipe-jacking sites in Kuching, Malaysia. Samples were collected from the decantation chambers, which removed coarse particles from the tunneling rock spoils transported from underground up to the ground surface by a slurry system. The decantation chambers used standard screens mounted on shaker decks to segregate coarser spoils from the transport slurry thus allowing for reuse of the slurry fluid for continued transportation of tunneling rock spoils.

The particle size distribution tests of the tunneling rock spoils were conducted according to testing standard ASTM D422-63 (2002). The particle size distribution curves for the tunneling rock spoils before and after scalping are presented in Fig. 1**.** Scalping was conducted to fulfil direct shear test requirements. Based on sieve analysis, both sets of spoils (before and after scalping) comprised of sand-sized grains and can be classified as poorly graded tunneling rock spoils according to both the Unified Soil Classification System (ASTM, 2000) and the Australian Soil Classification System (Standards Australia, 1993). Table 1 shows the results of sieve analyses performed on the scalped spoils.

EXPERIMENTAL PROCEDURE

Strain-controlled drained direct shear tests were conducted on the tunneling rock spoils in accordance to ASTM D 3080 (2003) and AS 1289.6.2.2 (1998), through the use of the GeoComp ShearTrac II apparatus. The fully automated direct shear system allows for full automation of testing and extraction of test results, ensuring reliability of the tests. Control of the testing parameters (consolidation, normal pressure, shear strain rate, and limits) was achieved through pre-set limits of the automation functions, allowing for close scrutiny of shear stresses and vertical displacements with horizontal deformation. The direct shear tests were conducted under constant normal load (CNL) conditions to reflect the in-situ stress conditions along the drives.

Scalping of the spoils was carried out in relation to the size of the shear box, where the maximum particle size in the tested samples did not exceed 1/10th of the thickness of the test specimen (Head, 1992). Successfully scalped samples were compacted in three layers within the shear box by using a tamping plate to ensure even distribution of the compaction effort. Test samples achieved relative densities ranging from 65% to 99% (see Table 1). Initial tests were conducted at effective normal stresses representative of the in-situ overburden pressures of the pipe-jacking works. Additional tests were performed beyond the confining pressures in order to generate more data points to establish failure criteria. The said effective normal stresses experienced by each sample type are also presented in Table 1. Specimens were saturated and consolidated under the applied effective normal stresses until the completion of primary consolidation, which was typically achieved within 5 minutes. ASTM (2003) recommends for clean dense sands to be sheared at a rate computed from Eq. (6).

$$d_r = d_f / t_f \tag{6}$$

where d_f = estimated horizontal displacement rate at failure (5 mm); t_f = total estimate elapsed time to failure (600 sec for clean dense sands); and d_r = displacement rate (mm/sec). Based on the recommended values for d_f and t_f, the shearing rate, d_r was 0.0083 mm/sec. A lower shearing rate of 0.0017 mm/sec was adopted for dissipation of excess pore pressures, if any. The samples were all tested to a maximum applied horizontal deformation of 15 mm, which was sufficient to achieve residual state for the samples tested.

DIRECT SHEAR TEST RESULTS

The variations of shear stress and vertical displacements against horizontal displacement from direct shear testing on scalped tunneling rock spoils are shown in Fig. 2(a). Test 1 showed distinct peak shear stress values for sandstone, before decreasing to a lower residual shear stress value. For Tests 2, 3 & 4 on rock spoils of an argillaceous nature, the degree of post-peak strain-softening was not as noticeable as tests conducted on rock spoils of an arenaceous nature, i.e. sandstone. This was especially apparent on spoils of sedimentary shale. Relatively large displacements were necessary to reach residual shear stresses, which is typical of argillaceous materials. This was attributed to the larger grain sizes found in the argillaceous spoils (Cerato and Lutenegger, 2006).

For vertical deformation (see Fig. 2(b)), positive values indicated compression of the test specimen, while negative values indicated dilation. All tested samples showed initial contractive behavior during initial increase of shear strain, indicating particulate interlocking. In general, dilative behavior was observed upon reaching peak shear stresses. In Tests 2, 3 & 4, subsequent contraction was observed at relatively larger horizontal displacements in excess of 8 mm, with larger applied effective normal stresses resulting in larger contractions. Particle breakage appears to have occurred at this stage. This has been attributed to the flaky and angular particles of shale spoils and phyllite spoils.

Non-linear strength envelopes for arenaceous (i.e. sandstone) and argillaceous (i.e. phyllite and shale) rock spoils respectively are presented in Fig. 2(c). MC strength criteria were initially developed based on regressive lines through the data points obtained from direct shear testing. This was utilized to illustrate the shear strengths of the tested tunneling rock spoils. However, the suitability of linear lines of best fit was dependent on the range of stresses at which the specimens are tested. If not selected accurately, this may result in over-estimation of shear strengths at extremely low or high stresses, while under-estimating shear strengths at intermediate stress levels. Therefore, power law functions were considered. Power law functions for geomaterials have been studied for various geotechnical applications (Anyaegbunam, 2015; Lade, 2010; Soon and Drescher, 2007). De Mello (1977) introduced a simplified power-type function stated as

$$\tau = A \cdot \left(\sigma'\right)^{B} \quad (7)$$

where *A* and *B* are constants. This function was adopted by Charles and Watts (1980) and De Mello (1977) in characterizing the non-linear shear strength behavior of rockfills. Table 2 lists the values of *A* and *B* based on test data from the tunneling rock spoils, using Eq. (7). Values of *B* were typically between 0.6 and 1.0, which conform to power law exponent values reported by Anyaegbunam (2015), i.e. $0 < B < 1$. For Test 2, a linear line of best fit was sufficient to represent the data points. For convenience, the linear envelope was directly characterized with a suitable MC criterion.

Despite the non-linearity of shear strength for the tested tunneling rock spoils, the jacking force predictive model (Eqs. (1) & (2)) is dependent on the linear MC failure criteria, and requires the use of values for c' & ϕ'. Yang and Yin (2004) introduced a "generalized tangential" technique to approximate the non-linear power law failure criterion. Similar approaches have been used in other applications (Collins et al., 1988; Drescher and Christopoulos, 1988; Soon and Drescher, 2007). The non-linearity is simplified as linear MC failure criterion tangential to the non-linear power law functions. Tangents to the respective power-type curves were applied at the effective confining pressures pertaining to the respective pipe-jacking depths (Fig. 2). Values for these tangential MC parameters for both peak and residual phases are presented in Table 3. (Note that the direct shear test results and interpreted strength profiles for Tests 2, 3, and 4 can be found in Fig. S2, Fig. S3, and Fig. S4 respectively in the Supplemental Data).

The tested specimens exhibited reduction in shear strength from peak to residual phases. This was illustrated in the power function parameters by a decrease in *A* values, and a corresponding increase in values of *B* (implying reduced curvature of the non-linear power function). From the interpreted tangential MC parameters, drops in c' and ϕ' were also observed. Generally, values of residual friction angle were lower than values corresponding with peak frictional angle. Tested spoils of sandstone (Test 1) showed significant loss of cohesion. Post-peak behavior of the sample suggests the formation of the shear zone, marked by strain softening and dilation of the specimen. Li and Aydin (2010) attributed this to the rotation and rolling of large particles, or larger quartz crystals (see Fig. S1(a)) as in the case of Test 1.

In sands and gravels, friction angle ϕ' values decreased with increasing values of coefficient of uniformity, C_u for frictional MC specimens (Wang et al., 2013). As stated earlier, B indicates the curvature of the power function strength envelope. For peak shear strengths, an increase in C_u from Test 1 to Tests 3 & 4 saw a decrease in the curvature of the power function strength envelopes, as indicated by an increase in B. Hence, as C_u increased, the variation of ϕ' with effective normal stresses was less apparent.

Spoils of shale (Tests 2 & 4) exhibited relatively low values of apparent cohesion. Peak phases were not as apparent as those observed in Test 1, shown by the relatively minimal dilation. Similar observations were made for phyllite spoils (Test 3). Phyllite spoils exhibited the highest values for apparent cohesion, with only a slight decrease in cohesion from peak to residual phases. In contrast to the blocky quartz crystals found in sandstone (Test 1), angular and plate-like mica present in phyllite imply that formation of the shear zone was more likely to be achieved through particle breakage than through rotation and rolling of the particles (Lade et al., 1996).

APPLICATION OF DIRECT SHEAR TEST RESULTS TO THE BACK-ANALYSIS OF μ_{avg}

Field measurements of pipe-jacking activities comprising of jacking forces, jacking speeds, and lubrication use are shown in Fig. 3 for Drive A where tunneling rock spoils were collected for direct shear testing. The interpreted geology, cumulative days elapsed, and cumulative lubricant injected have also been included for the respective drives. (Note that the field measurements of pipe-jacking activities for Drives B, C, and D can be found in Fig. S5, Fig. S6, and Fig. S7 respectively in the Supplemental Data).

The jacking forces for the four pipe-jacking drives consisting of different geological settings were studied. These jacking forces, with respect to the jacking length, were characterized as gradients (kN/m), representing the average jacking forces for each drive, respectively.

The strength parameters obtained from direct shear testing of scalped tunneling rock spoils were used for back-analyses of the average jacking forces through the use of the jacking force equations (Eqs. (1) & (2)). For calculation of vertical normal stress acting on the pipeline due to arching, σ_{EV} (Eq. (2)), peak strength parameters c'_p & ϕ'_p were used in place of C & ϕ, respectively. Arching is caused by initial vertical slippage of the overburden soil or rock mass over an excavated void during pipe-jacking. This usually occurs at low displacements of the soil or rock mass into the bored tunnel. Hence, peak values were used for estimating σ_{EV}. In some cases, the calculated σ_{EV} resulted in negative values, which implied tensile soil stresses acting on the pipe. Terzaghi (1943) stated that beyond certain tunnel depths, the vertical stresses at a tunnel roof were equal to zero. From Eq. (2), σ_{EV} is equal to zero provided

$$b \leq \frac{2C}{\gamma} \qquad (8)$$

Further explanation on the measured jacking forces shall be described in detail hereinafter.

CASE STUDIES

The case studies described hereinafter will illustrate the use of direct shear test results from shearing of reconstituted tunneling rock spoils, back-analyzed frictional coefficients, and effect of construction activities. The measured jacking forces are shown, superimposed with calculated average jacking force profiles based on theoretical upper μ = 0.3 and lower μ = 0.1 bounds due to lubrication (Stein et al., 1989), and corresponding back-analyzed μ_{avg}

based on Eqs. (1) & (2) using measured jacking forces from Fig. 3. The respective values of μ_{avg} have been compared against recommended frictional coefficient values of between 0.1 and 0.3 for lubricated drives, or 'fluid friction' (Stein et al., 1989). Table 4 shows the measured pipe-jacking activities from the studied drives, including face support pressure, pipe weight, TBM weight, jacking speed, and lubricant usage. The results from the back-analyses of jacking forces based on results from direct shear tests are also presented.

Drive A

Fig. 3 shows jacking forces, jacking speeds and lubrication for Drive A which traversed sandstone. The 1.43 m outer diameter, concrete pipeline (consisting of pre-cast, 3 m length concrete pipes) spanned 140 m at a depth of 12.5m. The average volume of injected lubrication was 47 L/m (see Fig. 3), into a theoretical overcut annulus of 87 L/m (see Table 4). This overcut region allowed for the injection of lubrication between the pipeline and the surrounding geology. Extracted sandstone cores from the receiving shaft of Drive A had majority RQD values of zero. The measured face support pressure was stable at 104 kN/m^2. Hence, the measured jacking forces could be analyzed in terms of frictional resistance, segregated from the face pressures. The jacking forces were well-represented with an average line of best fit (R^2 = 0.93) at 14.4 kN/m (see Fig. 3). From direct shear testing of sandstone (Test 1), the MC parameters for use in Eq. (2) were $C = c'_p$ = 50.8 kPa and $\phi = \phi'_p$ = 47.8° (see Fig. 2(c)). As stated previously, these values from the linear MC criterion were obtained by utilizing the "generalized tangential" technique on the non-linear power law function. The tangent to the non-linear function was applied at the effective overburden pressure with reference to the tunnel depth. This resulted in σ_{EV} = -20.8 kN/m^2, indicating a significant degree of arching over the pipe crown. However, negative values of σ_{EV} implied tensile stresses acting normal to the outer pipe peripheral. For the back-analysis of μ_{avg} (see Eq. (1)), σ_{EV} was adjusted to be equal to zero (Terzaghi, 1943). Fig. 4 shows the jacking forces incurred during jacking through sandstone (Test 1), with the back-analyzed frictional coefficient, μ_{avg} = 0.31. Jacking force profiles corresponding with the recommended upper μ = 0.3 and lower μ = 0.1 limits of μ values for lubricated drives are also shown in Fig. 4. The back-analyzed μ_{avg} of 0.31 matched with the upper limit of μ = 0.3, as recommended by Stein et al. (1989) for lubricated drives. This indicates that Drive A in sandstone was lubricated moderately, confirmed by the comparison between injected volumes of lubrication and theoretical overcut (see Table 4).

Drive B

Fig. S5 shows the jacking activities for Drive B. Similar to Drive A, the jacked pipeline was of 1.43 m outer diameter. It also spanned 140 m at a depth of 12.5m. However, Drive B negotiated through shale (Test 2). The volume of injected lubricant averaged 682 L/m (see Fig. S5) and was significantly in excess of the theoretical overcut annulus of 87 L/m (see Table 4). As summarized in Table 4, the extracted shale cores from the borehole done at both receiving and jacking shafts had RQD values ranging from 0 to 80%, with a mean RQD of 26.0%. From Test 2, the equivalent tangential peak MC parameters were c'_p = 0 and ϕ'_p = 41.4° (see Fig. S2). Using these values with Eq. (2), the calculated σ_{EV} was 34.1 kN/m^2, indicating a reduced arching effect as compared to Drive A. The measured face support pressure was constant throughout the drive at 68 kN/m^2; with the average jacking force measured at 29.0 kN/m (R^2 = 0.90; see Fig. S8 of the Supplemental Data). The back-analyzed frictional coefficient, μ_{avg} was 0.20. Fig. S8 illustrates that the back-analyzed μ_{avg} of 0.20 was within the margin recommended by Stein et al. (1989) for lubricated drives, suggesting that Drive B through shale was well-lubricated.

Drive C

Fig. S6 shows jacking activities for Drive C, which spanned 120 m through phyllite (Test 3). Tan (1993) reported phyllite from the Tuang Formation as being bedded, tightly folded and highly sheared. From Table 4, it is observed that the extracted cores were characterized by average RQD values of 17.5%, with 7.6 m of the total 15.1 m in extracted cores exhibiting RQD values of zero. The average volume of lubricant injected was 181 L/m (see Fig. S6), slightly in excess of the theoretical overcut of 113 L/m (see Table 4). From Test 3, the tangential peak MC parameters were c'_p = 57.8 kPa and ϕ'_p = 44.3° at σ' = 222 kPa (see Fig. S3). Using these values with Eq. (2), the calculated σ_{EV} was -22.3 kN/m^2, demonstrating that significant arching was present during pipe-jacking works at Drive C. The measured face support pressure was stable at 47 kN/m^2. The average measured jacking force of 4.8 kN/m (R^2 = 0.78) was the lowest of the jacking forces observed in this study (see Fig. S9 of the Supplemental Data), with back-analyzed μ_{avg} of 0.07, indicating that Drive C in phyllite was very well-lubricated.

Drive D

Fig. S7 shows jacking forces for Drive D, a pipe-jacking drive spanning 228 m, which navigated an initial 135 m section of stiff clay (SPT N value of 31), followed by a latter section through shale (Test 4). Extracted shale cores from the borehole done at the receiving shaft, corresponded with RQD values ranging from 10% to 23% with an average of 14% as shown in Table 4. In the shale section of Drive D, an average of 729 L/m of lubricant was injected into a theoretical overcut of 113 L/m (see Table 4). Tunneling shale spoils were obtained from the latter section of the drive (136 m to 228 m) for direct shear testing (Test 4). The equivalent tangential peak MC parameters were c'_p = 29.0 kPa and ϕ'_p = 38.7° at σ' = 234 kPa (see Fig. S4). From Eq. (2), the computed σ_{EV} was 11.7 kN/m^2. The average face support pressure had minimal fluctuations, measured at 115 kN/m^2. However, the measured jacking forces were highly scattered (R^2 = 0.45) averaging at 81.1 kN/m (see Fig. S10 of the Supplemental Data). This is believed to be due to a 19-day extended stoppage in the pipe-jacking works, which occurred at the clay-shale interface (see Fig. S7). The subsequent back-analyzed μ_{avg} was 0.71, indicating that lubrication during pipe-jacking of Drive D was ineffective, despite having about 6.5 times more lubricant injected into the theoretical overcut. This shall be explained in the discussion section later.

Summary of drives

Using the "generalized tangential" technique, equivalent MC parameters were estimated from non-linear failure envelopes of direct shear tests on scalped tunneling rock spoils. These tangential MC parameters have been used for the back-analysis of average measured pipe-jacking forces through highly fractured rock formations of varying geology. Values of σ_{EV} were computed based on Eq. (2) to provide indication of the arching effect in the studied drives. The back-analyzed μ_{avg} values have been compared against μ values recommended by Stein et al. (1989) for lubricated drives. The μ_{avg} values showed that the pipe-jacking drives were lubricated, with the exception of Drive D which was affected by stoppages. These back-analyzed μ_{avg} values were verified by the volume of injected lubrication. The effects of arching, lubrication and stoppages shall be discussed hereinafter.

DISCUSSION

Effect of arching on jacking forces

In Drive A (sandstone), the calculated σ_{EV} value of -20.8 kN/m^2 was much lower (representing relatively more arching effect) than that determined for Drive B (shale) at 34.1 kN/m^2, resulting in approximately two times lower jacking forces for Drive A. Jacking speeds were slightly higher in Drive A than in Drive B. In Drive A, the average injected lubricant was 47 L/m or average 6,580 L for the entire length of 140 m, which was only a fraction of the theoretical overcut volume of 12,180 L for 140 m. This may have explained the back-analyzed μ_{avg} of 0.31, which is close to the 0.3 upper limit suggested for lubricated drives by Stein et al. (1989).

The volume of lubricant injected for Drive B was 682 L/m, which was significantly in excess of the theoretical overcut volume. The large volume of lubricant was injected as a response to mitigate excessive increase in jacking forces. Back-analysis of the subsequent jacking forces resulted in μ_{avg} of 0.20, which corresponds with the margin of 0.1 to 0.3 recommended for well-lubricated pipe-jacking drives by Stein et al. (1989).

The effect of geology on jacking forces, and consequently on jacking speed and lubricant use was also apparent in Drive C (Test 3) through phyllite (see Fig. S6). The stress reduction at the pipe crown due to presence of arching (σ_{EV} = -22.3 kN/m2) could also be attributed to phyllite, which is characterized as being intensely folded with steep dips (Tan, 1993). Folds were also depicted in micrographs of phyllite (see Fig. S1(c)). These geological features created a structurally stable bore, allowing for re-distribution of soil stresses around the outer peripheral of the pipeline, i.e. arching.

The erratic structure of phyllite also reduced the lubrication injected, as the intense folding likely reduced the permeation of lubricant into the surrounding geology. The retention of lubrication in the overcut ensured that the discretization of a lubricating layer was maintained. This phenomenon has allowed Drive C to record the highest observed jacking speeds across the various drives studied. The reduction of stresses acting on the pipe outer surface together with the retention of lubricant resulted in relatively low jacking forces. This resulted in an average measured jacking force of only 4.8 kN/m, for which the back-analyzed μ_{avg} was 0.07, significantly lower than the recommended value of 0.1 by Stein et al. (1989).

For Drive D (shale), the calculated σ_{EV} value of 11.7 kN/m^2 indicated that arching effect was reduced, similar to Drive B. Similarly to Drive B, the volume of injected lubricant was well in excess of the theoretical overcut volume. It was likely that much of the injected lubrication was lost into the surrounding geology through fissures. However, Drive D encountered extended stoppage at the clay-shale interface, resulting in highly scattered jacking forces in the shale section (see Fig. 3(d)).

These observations indicate that geology had a significant effect on the jacking forces due to the stresses acting on the pipe by virtue of the arching effect. Subsequently, jacking forces affected the response of the construction process, i.e. lubricant usage and jacking speeds. This shows the coupling of arching and lubrication effects on jacking forces during pipe-jacking. Therefore, it is summarized that:

(i) Sandstone (Drive A): Significant arching, relatively low jacking forces; subsequently lower lubricant usage.

(ii) Phyllite (Drive C): Strongest arching due to folds and metamorphic nature, lowest jacking forces; subsequently moderate lubricant usage.

(iii) Shale (Drive B & D): Reduced arching, relatively high jacking forces; subsequently higher lubricant usage.

Effect of geology on lubrication

Drive A in sandstone had minimal lubrication, with only 47 L/m injected into the theoretical overcut of 87 L/m as summarized in Table 4. However, the back-analyzed μ_{avg} of 0.31 still corresponded well with the recommended upper limit of 0.3 as recommended by Stein et al. (1989) for lubricated drives. Drives B & C seem to be well-lubricated, as the back-analyzed values of μ_{avg} were within the limits of 0.1 and 0.3 as recommended by Stein et al. (1989) for lubricated pipe-jacking drives. For Drive B, the volume of injected lubricant was 682 L/m, largely in excess of the theoretical overcut of 87 L/m (see Table 4). For Drive C, the volume of injected lubricant amounted to 181 L/m, compared to the theoretical overcut of 113 L/m. In both Drives B & C, the injected lubrication was in excess of the theoretical overcut annulus, implying a continuous effort to saturate the overcut annulus. However, in Drive B, it seems that there was significant loss of lubrication in order to sustain well-lubricated conditions during pipe-jacking.

The excessive injection of lubricant observed in Drives B & D indicated a loss of lubrication during the pipe-jacking works. This could imply difficulties in maintaining a watertight overcut, most likely due to surrounding rock fissures or the inability of the lubrication to form a filter cake of low permeability (Pipe Jacking Association, 1995).

A permeable overcut would mitigate the establishment of a discrete lubricating layer between the pipe and the surrounding geology. Fig. 5(a) shows a schematic illustration of the postulated lubrication scenario for Drives B & D. The lack of lubrication retained in the overcut could also cause an increase in pipe-rock contact due to the loss of buoyant lubricating forces acting on the pipe, that otherwise would have supported the pipe in fluid suspension (French Society for Trenchless Technology, 2006; Pipe Jacking Association, 1995). Fig. 5(b) shows how buoyant forces can be achieved in a stable watertight bore, which was the postulated situation for Drives A & C.

Effect of stoppages on jacking forces

Stoppages also had significant effects on jacking forces during tunneling through rock. In Drive D, the initial transition from clay into rock corresponded with low jacking speeds of 3 mm/min (see Fig. S7), indicating difficulties in traversing through the change in geology. This difficulty was reflected by the suspension of tunneling activities lasting 19 days. Upon resumption of tunneling works, the average volume of injected lubricant increased from 380 L/m in clay to 729 L/m in rock, compared with a theoretical overcut volume of 113 L/m. For the shale section, the average measured jacking force was 81.1 kN/m, with a computed σ_{EV} of 11.7 kN/m^2. The back-analyzed μ_{avg} of 0.71 easily exceeded the suggested values for lubricated drives. It is also noted that the jacking forces in this shale section of Drive D fluctuated greatly. The line of best fit depicting the average measured jacking force was accurate to only R^2 = 0.45. Restarting of jacking works after extended stoppages produce large static frictional resistance (Chapman and Ichioka, 1999; Norris, 1992; Sofianos et al., 2004). The fluctuation of jacking forces seemed to have resulted from the restarting of works. Additionally, stoppages can significantly impact lubricant use, particularly when pipe-jacking through fissured geology. Any lubricant present in the overcut would be lost into fissures, which act as drains. Upon resumption of pipe-jacking, the void overcut would need to be refilled with lubricant. Re-injection of lubrication is usually ineffective during a restart due to squeezing of the soil and rock surrounding the pipeline. Ground squeezing reduces the overcut area and increases pipe-rock contact area. Full lubrication of the overcut is able to provide uplift and buoyancy to the pipeline, allowing for full suspension of the jacked pipes. This results in reduced contact between the pipe and the surrounding geology, particularly at the pipe invert.

CONCLUSIONS

Tunneling rock spoils collected from the decantation chambers of four different pipe-jacking sites in Kuching City, Malaysia were tested for the assessment of physical characteristics and geotechnical strength properties. The scalped test specimens were classified as sand-sized and poorly-graded. Direct shear tests were conducted on these scalped, reconstituted tunneling rock spoils in order to characterize them so that assessment of pipe-jacking forces could be better understood and reliably estimated.

Results from the direct shear tests were then applied to a well-established jacking force model and subsequently benchmarked against field measured jacking loads. The assessment of jacking forces was conducted by considering the vertical stresses at the pipe crown, σ_{EV} and the volumes of lubricant injected. The back-analyzed frictional coefficient values derived from the four pipe-jacking drives in the Tuang Formation of Kuching City have been found to be reliable and have been explained in relation to their surrounding geologies. The consistencies in findings and discussions made herein are important for the reconstituted rock spoils to be considered as friable, highly weathered 'soft rock', thus exhibiting characteristics that tend towards soil behavior. This has allowed for assessment of pipe-jacking forces using jacking force equations developed for pipe-jacking drives in soil. Water-tightness of the overcut region has been found to be important in maintaining a discrete layer of lubrication that can relieve frictional stresses along the pipe-rock interface. Comparison of the studied drives has shown that arching effects, jacking forces, amount and pattern of lubricant use as well as jacking speeds can be strongly related to the traversed geologies during pipe-jacking works. Stoppages were observed to be a significant factor that can lead to higher jacking forces upon resumption of a jacking drive. Although the assessment of jacking forces through rocks is limited in existing jacking force models, the current study shows that back-analyzed μ_{avg} can be used to evaluate pipe-jacking forces through weathered geology.

ACKNOWLEDGMENT

The Authors would like to express their thanks for the generosity shown by Hock Seng Lee Bhd. and Jurutera Jasa (Sarawak) Sdn. Bhd. during this study. The authors are also thankful to Ms. Hsiao-Yun Leong, Mr. Mohammud Irfaan Peerun, and Mr. Yi-Zhou Tan for assisting in the work described in this paper.

SUPPLEMENTAL DATA

Figs. S1 to S10 are available online in the ASCE Library (www.ascelibrary.org).

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Table 1

Table 1. Physical properties of scalped tunneling rock spoils.

Test no. / Drive	Test 1 / Drive A	Test 2 / Drive B	Test 3 / Drive C	Test 4 / Drive D
Geology	Sandstone	Shale	Phyllite	Shale
D_{60} (mm)	0.53	0.51	0.73	0.73
D_{30} (mm)	0.25	0.35	0.29	0.27
D_{10} (mm)	0.11	0.21	0.10	0.13
D_{50} (mm)	0.45	0.45	0.55	0.61
Weighted average particle size, D_{av} (mm)	1.02	0.59	0.80	0.82
Coefficient of uniformity, C_u	3.53	2.32	5.57	5.20
Coefficient of curvature, C_c	0.77	1.16	1.12	0.67
Material classification	Poorly graded sand-sized spoils (SP)	Poorly graded sand-sized spoils (SP)	Poorly graded sand-sized spoils (SP)	Poorly graded sand-sized spoils (SP)
Applied effective normal stresses for direct shear tests (kPa)	25, 50, 75, 100 255, 600	50, 100, 250, 400	100, 250, 380, 400	150, 375, 395, 500
Densities of tested samples (kN/m^3)	15.9 – 16.9	13.7 – 14.8	–[a]	15.5 – 16.7

[a]Note: Measurements were not made for Test 3 specimens.

Table 2

Table 2. Shear strength of tunneling rock spoils and rockfill materials using power law functions, where $\tau = A \cdot (\sigma')^B$

Material type	Geology	*A*	*B*	Source
Tunneling rock spoils	Sandstone (Test 1 – peak)	4.68	0.76	This study
	Sandstone (Test 1 – residual)	2.24	0.84	
	Phyllite (Test 3 – peak)	1.92	0.87	
	Phyllite (Test 3 – residual)	1.07	0.96	
	Shale (Test 4 – peak)	3.86	0.79	
	Shale (Test 4 – residual)	3.37	0.78	
Rockfill	Sandstone	6.8	0.67	Charles and Watts (1980)
	Slate	5.3	0.75	
	Slate	3.0	0.77	
	Basalt	4.4	0.81	
	Basalt	1.54	0.821	De Mello (1977)
	Diorite	1.10	0.870	
	Conglomerate	1.27	0.846	
	Conglomerate	1.19	0.881	
	Conglomerate	1.59	0.808	

Table 3

Table 3. Parameters used in pipe-jacking force model for back-analyses of μ_{avg}

Drive (Test no.)		Drive A (Test 1)	Drive B (Test 2)	Drive C (Test 3)	Drive D (Test 4)
Geology		Sandstone	Shale	Phyllite	Shale
D_e (m)		1.43	1.43	1.78	1.78
γ (kN/m^3)		22	22	22	22
h (m)		12.5	12.5	18.5	19.5
Power function, $\tau = A \cdot (\sigma')^B$ applied to data points[a]	A_p	4.68	N/A since power function is not applicable.[b]	3.86	1.92
	B_p	0.76		0.79	0.87
	A_r	2.24		3.37	1.07
	B_r	0.84		0.78	0.96
MC parameters[a]	c'_p (kPa)	50.8	0	57.8	29.0
	ϕ'_p (°)	47.8	41.4	44.3	38.7
	c'_r (kPa)	24.2	0	50.3	8.3
	ϕ'_r (°)	39.6	37.6	39.2	39.4
Average measured jacking forces (kN/m)		14.4	29.0	4.8	81.1
Back-analyzed μ_{avg}, using Eq. (1)		0.31	0.20	0.07	0.71
R^2		0.93	0.90	0.78	0.45

[a]Note: Subscript p denotes peak values; subscript r denotes residual values

[b]Note: Data points can be conveniently represented by line of best fit, hence power law is not necessary here.

Table 4

Table 4. Comparison of pipe-jacking performance for various drives

Test no.	1	2	3	4
Drive	Drive A	Drive B	Drive C	Drive D
Geology	Sandstone	Shale	Phyllite	Shale
Length of nearby rock cores extracted (m)	4.5	5.1	15.1	6.0
Average RQD (%)	0	26.0	17.5	14.0
Length of rock cores with RQD = 0 (m)	4.5	0.65	7.6	0.7
Average volume of lubricant injected including losses (L/m)	47	682	181	729
Average theoretical overcut volume (L/m)	87	87	113	113
Effective overburden pressure (without arching) (kN/m^2)	150	150	222	234
Average jacking speed (mm/min)	16	10	44	34
Average measured face support pressure (kN/m^2)	104	68	47	115
Cutter face diameter (m)	1.47	1.47	1.82	1.82
TBM weight (tonnes)	15	15	20	20
Pipe weight (kN/m)	17.3	17.3	11.6	11.6
Calculated σ_{EV} (kN/m^2) (Eq. (2))	-20.8[a]	34.1	-22.3[a]	11.7
Average measured jacking forces (kN/m)	14.4	29.0	4.8	81.1
Back-analyzed μ_{avg}	0.31	0.20	0.07	0.71

[a]Note: Negative values of σ_{EV} (Eq. (2)) indicate possible presence of significant arching. For back-analysis of μ_{avg}, these negative values of σ_{EV} were adjusted to be equal to zero (Terzaghi, 1943).

Figure 1

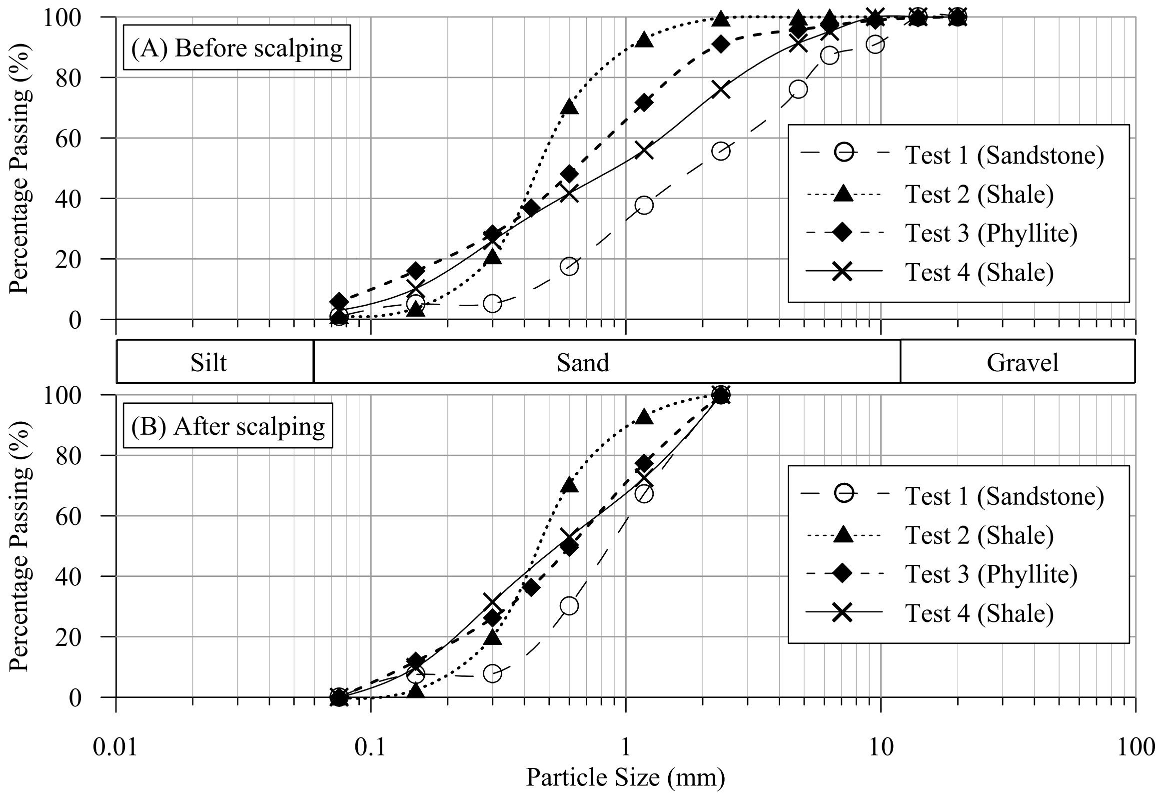
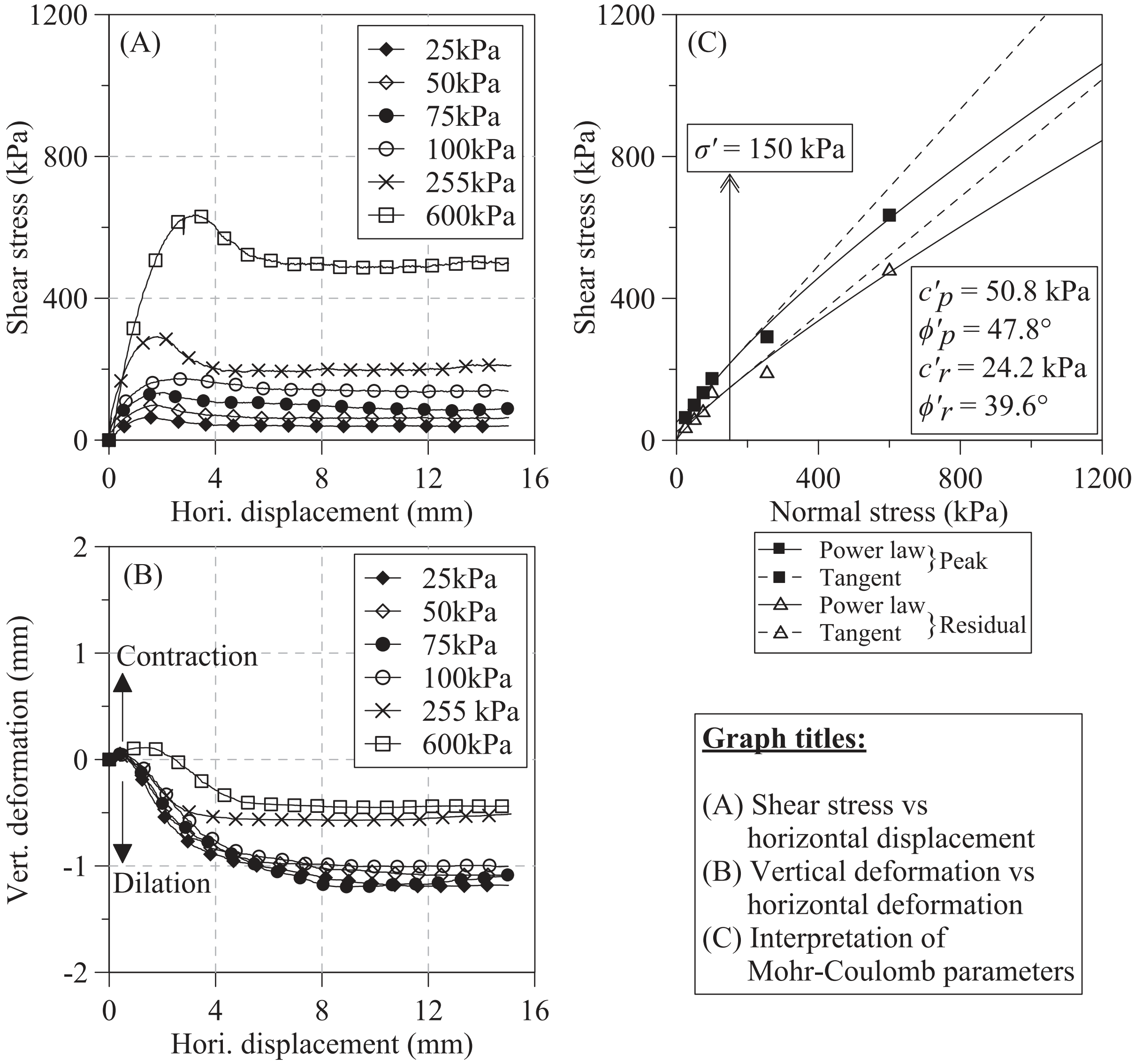
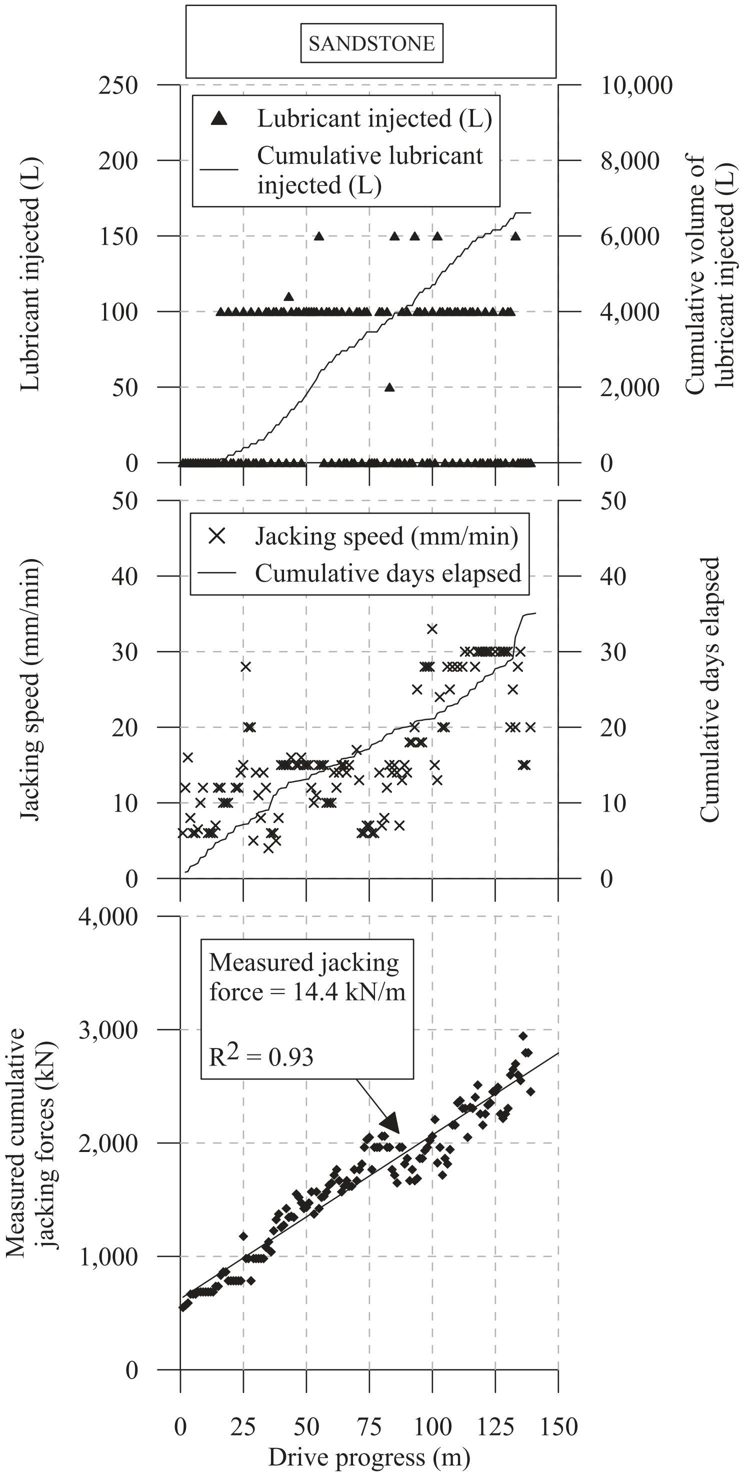

(A) Before scalping
(B) After scalping
Percentage Passing (%)
Particle Size (mm)
Silt
Sand
Gravel
Test 1 (Sandstone)
Test 2 (Shale)
Test 3 (Phyllite)
Test 4 (Shale)
0
20
40
60
80
100
0.01
0.1
1
10
100

Figure 2



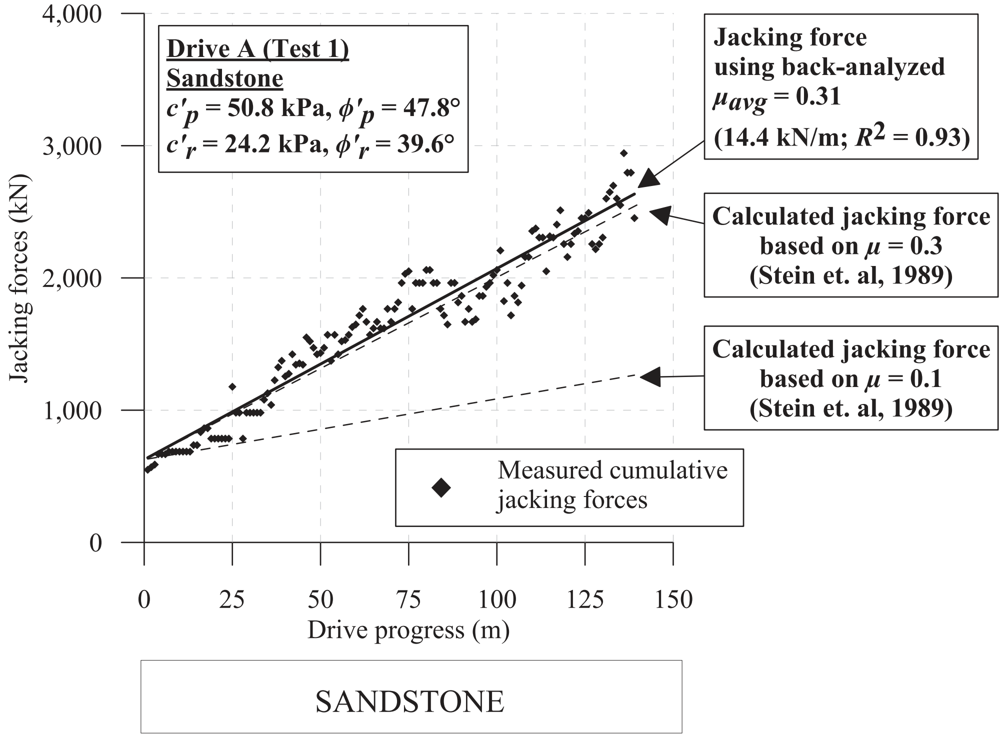
(A)
Shear stress (kPa)
Hori. displacement (mm)
25kPa
50kPa
75kPa
100kPa
255kPa
600kPa
(B)
Vert. deformation (mm)
Contraction
Dilation
255 kPa
(C)
Normal stress (kPa)
σ′ = 150 kPa
c′p = 50.8 kPa
ϕ′p = 47.8°
c′r = 24.2 kPa
ϕ′r = 39.6°
Power law
Tangent
Peak
Residual
Graph titles:
(A) Shear stress vs horizontal displacement
(B) Vertical deformation vs horizontal deformation
(C) Interpretation of Mohr-Coulomb parameters

Figure 3



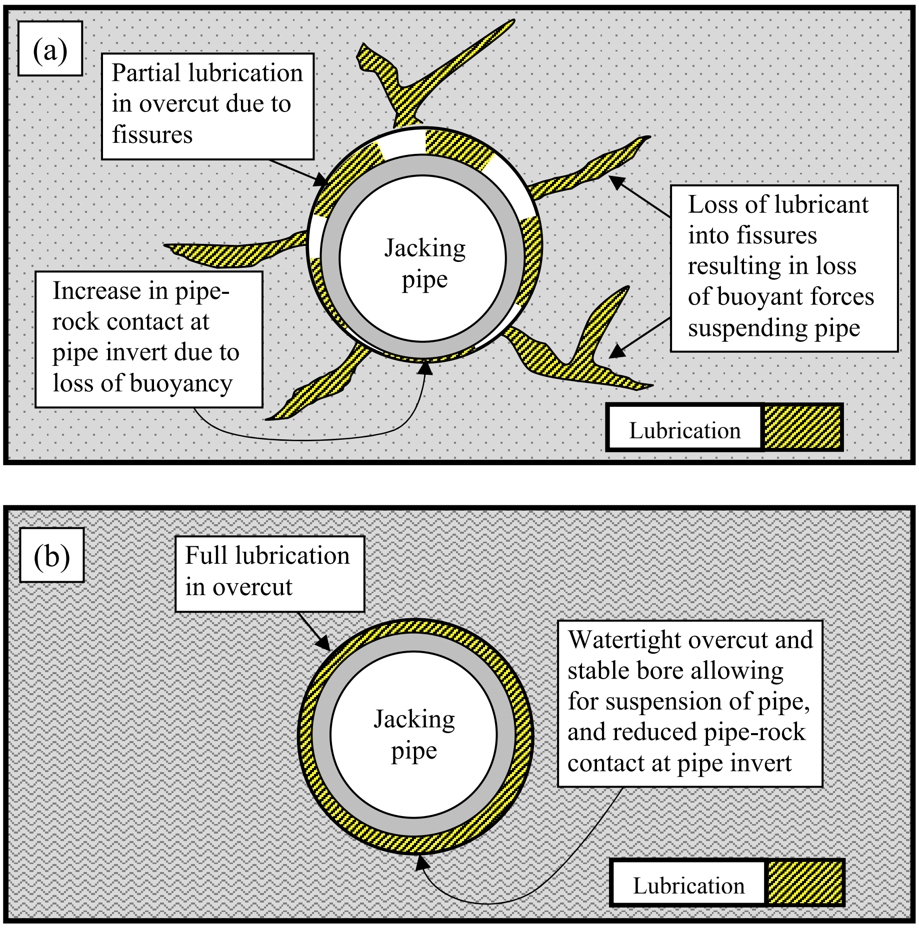
SANDSTONE
Lubricant injected (L)
Cumulative lubricant injected (L)
Cumulative volume of lubricant injected (L)
250
200
150
100
50
0
10,000
8,000
6,000
4,000
2,000
0
Jacking speed (mm/min)
Cumulative days elapsed
50
40
30
20
10
0
Measured cumulative jacking forces (kN)
Measured jacking force = 14.4 kN/m
R^2 = 0.93
4,000
3,000
2,000
1,000
0
0
25
50
75
100
125
150
Drive progress (m)

Figure 4



Drive A (Test 1)
Sandstone
c'_p = 50.8 kPa, ϕ'_p = 47.8°
c'_r = 24.2 kPa, ϕ'_r = 39.6°
Jacking force
using back-analyzed
μ_{avg} = 0.31
(14.4 kN/m; R^2 = 0.93)
Calculated jacking force
based on μ = 0.3
(Stein et. al, 1989)
Calculated jacking force
based on μ = 0.1
(Stein et. al, 1989)
Measured cumulative
jacking forces
Jacking forces (kN)
4,000
3,000
2,000
1,000
0
0
25
50
75
100
125
150
Drive progress (m)
SANDSTONE

Figure 5


(a)
Partial lubrication in overcut due to fissures
Loss of lubricant into fissures resulting in loss of buoyant forces suspending pipe
Jacking pipe
Increase in pipe-rock contact at pipe invert due to loss of buoyancy
Lubrication
(b)
Full lubrication in overcut
Watertight overcut and stable bore allowing for suspension of pipe, and reduced pipe-rock contact at pipe invert
Jacking pipe
Lubrication

LIST OF TABLES

LIST OF FIGURES