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Assessment of Non-linear Rock Strength Parameters for the Estimation of Pipe-jacking Forces. Part 2. Numerical modeling

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Abstract

In the earlier companion paper [1], direct shear tests were performed on reconstituted tunneling rock spoils for the development of equivalent tangential cohesion, $c'_{t,p}$ and friction angle, $\phi'_{t,p}$. These equivalent rock strength parameters were used in assessing jacking forces, which were measured from three pipe-jacking drives negotiating the highly weathered lithologies from the Tuang Formation, Malaysia. The measured jacking forces were back-analyzed, leading to the development of frictional coefficient, μ_{avg} and the vertical stress at the tunnel crown, σ_{EV} . In this paper, the reliability of using the parameters, $c'_{t,p}$, $\phi'_{t,p}$, μ_{avg} and σ_{EV} was assessed through the use of three-dimensional finite element modeling of the studied pipe-jacking drives. The peak tangential strength parameters, $c'_{t,p}$ and $\phi'_{t,p}$ were applied to the modeled rock mass and the strength parameters of the pipe-rock interface were developed from μ_{avg} . Arching ratios were developed from σ_{EV} , which were subsequently used for post-analysis assessment of the results from numerical modeling. There was sound agreement between the resulting pipe-rock interface shear stresses, τ_1 and reaction loads, ΣF_y from the numerical analysis against the field measured jacking forces, thus demonstrating the reliability of the use of developed equivalent rock strength parameters, $c'_{t,p}$, $\phi'_{t,p}$, μ_{avg} and σ_{EV} in the assessment of pipe-jacking forces through highly weathered lithologies.

Keywords: Pipe-jacking; Friction; Finite element; Arching ratio

1. INTRODUCTION

In the earlier companion paper [1], tunneling rock spoils were used for the backanalysis of jacking forces. This method has been used successfully for pipe-jacking drives traversing lithological units of sandstone, shale and phyllite origins [2]. From the earlier companion paper, direct shear tests were performed on reconstituted tunneling rock spoils of shale and interbedded lithological units of metagraywacke – siltstone and greywacke – phyllite origins. It was essential for the direct shear test results to be relevant in the understanding of jacking forces for pipe-jacking drives through the highly weathered and fractured Tuang Formation in Kuching City, Malaysia. This necessitated the development of tangential strength parameters, $c'_{t,p}$ and $\phi'_{t,p}$ based on arching theory. Typically, arching theory is more applicable to soil. However, in highly fractured lithologies it is possible for the highly weathered rock to demonstrate soil-like behavior, akin to a highly weathered ‘soft rock’ [2]. In the current study, the consideration of the traversed rock masses as ‘soft rock’ will be explained, hereinafter.

From the backanalysis of the jacking forces, the developed average frictional coefficient, μ_{avg} were used to indicate the nature of pipe-rock friction, i.e. lubricated or unlubricated. The stresses acting normal to the outer periphery of the pipe crown, σ_{EV} was also developed from the tangential strength parameters. Values of σ_{EV} were used to indicate the degree of arching, as arching effect was found to be rather significant in the assessment of pipe-jacking forces.

The work described in the current paper was motivated by the need to evaluate the reliability of the peak tangential strength parameters, $c'_{t,p}$ and $\phi'_{t,p}$ (developed from direct shear testing) and μ_{avg} and σ_{EV} (developed from backanalysis of field measured jacking forces), particularly for the assessment of pipe-jacking forces in highly weathered and fractured lithologies. Hence, the reliability of using these parameters was assessed through

three-dimensional (3D) finite element modeling of the pipe-jacking drives studied in this paper. The peak tangential strength properties, $c'_{t,p}$ and $\phi'_{t,p}$ were applied to the modeled rock mass, while the values of backanalyzed μ_{avg} were applied to the strength models of the pipe-rock interface elements. Prescribed displacements were used to simulate the forward advancement of the rigid ‘wish-in-place’ concrete pipeline. This modeling technique was validated through the use of the lower bound theorem for assessing the incurred pipe-rock interface shear stresses, τ_1 and the reaction loads, ΣF_y . For the consideration of arching in the results from the numerical analysis of the pipe-jacking drives, arching ratios were developed from the backanalyzed σ_{EV} , and subsequently applied to jacking forces computed from τ_1 and ΣF_y as post numerical analysis.

2. NUMERICAL MODELING OF TUNNELS

Several studies have used continuum methods in the modelling of tunnels negotiating rock masses [3-11]. Basarir et. al [3] and Genis et. al [4] utilized continuum-based finite element analysis for the simulation of tunnels traversing fractured rock masses from Turkey for the purpose of designing tunnel support systems. These rock masses included moderately weathered limestone, highly weathered diabase (with average RQD of 28%), and moderately weathered sandstone (with average RQD of 34%) [3], as well as heavily broken phyllite and moderately weathered granodiorite [4]. Pusch and Börgesson [5] proposed functions for the computation of rock mass moduli and stiffnesses of discontinuities through the use of finite element methods. Zhu et. al [6] simulated tunnel excavation schemes (single- and multi-benching) for various rock conditions (including soft to hard rocks as well as fault materials) in Taiwan in order to study tunnel stability through the use finite element method. The continuum-based models were able to simulate the development of plastic zones in the rock masses around the tunnel, thus allowing the study of the distribution of rock movements around the tunnel. These past studies show that continuum-based finite element methods can

be used for the modelling of tunnels traversing rock masses of assorted lithologies with varying weathering conditions.

However, the numerical simulation of pipe-jacking drives is less common, particularly when modeling the advance of the pipeline through a rock mass. Li et. al [7] used continuum-based finite element analysis software to study the interfacial contact area between rocks (with behavior simulated using the Mohr-Coulomb failure criterion) and pipes during pipe-jacking. The continuum models comprised of contact conditions along the modelled pipeline. This model was used to simulate rock-pipe interfacial stresses from a case study of a pipe-jacking drive traversing lithological units of sandstone. Barla et. al [8] performed 2D analyses of pipe-jacking drives through fractured limestone, using continuum and discontinuum modeling techniques. The initial continuum modeling method was performed using FLAC [9]. Through the reduction of tensile strength of the rock mass, the modeled rock mass failed in a localized zone at the tunnel crown, with lower tensile strength resulting in larger failure zones. In order to study the influence of discontinuities in the rock mass on pipe-jacking forces, a discontinuum analysis was performed using UDEC [10]. The Geological Strength Index (GSI) of the modeled rock was varied. The numerical simulation found that the discontinuities in the rock mass resulted in high jacking forces, which led to a stoppage in pipe-jacking works. The work by Li et. al [7] and Barla et. al [8] demonstrated the possibility of modeling pipe-jacking drives in fractured lithologies.

Shou et. al [11] modeled linear and curved pipe-jacking alignments in 3D space to assess the effect of lubrication in the overcut annulus. The simulated pipe-jacking drives were jacked through the model soil based on repeated sequence of deactivating soil elements at the tunnel face, followed by a forward displacement of the modeled pipeline. The effect of lubrication was simulated by varying the pipe-soil frictional coefficient, f . It was found that jacking forces were not only a consequence of lubrication, but that the contact area between

the outer surface of the pipeline and the surrounding soil mass was a significant factor in the understanding of jacking forces. In a subsequent study, Yen and Shou [12] utilized the displacement control approach to simulate the jacking of pipes. Other studies have also showed that the forward advancement of the pipeline could be simulated through the use of prescribed displacements on the pipeline elements [17-19].

The aforementioned studies on numerical modeling emphasize the use of continuum-based finite element analysis in the simulation of pipe-jacking drives through soils and fractured rocks. The use of prescribed displacements is also necessary in the modeling of a rigid concrete pipeline, traversing a model rock mass. The findings from the earlier studies will form the basis for the numerical analysis of pipe-jacking forces conducted, hereinafter.

3. NUMERICAL ANALYSIS OF PIPE-JACKING FORCES

3D finite element analysis was performed on the pipe-jacking drives in this paper through the use of PLAXIS 3D, a commercially available finite element software package for geotechnical engineering purposes. The results from the direct shear testing of reconstituted tunneling rock spoils and the subsequent backanalysis of jacking forces are shown in **Table 1**, along with properties related to the traversed geology.

3.1. Finite Element Mesh

The finite element models used in this study measured 30 m wide in the x -direction (transverse to pipeline), 20 m long in the y -direction (longitudinal to pipeline), and 30 m deep in the z -direction. **Fig. 1** shows the typical finite element mesh used for the numerical analysis hereinafter. 10-node tetrahedral elements were used to simulate the rock and soil masses, while 6-node plate elements were used to model the concrete pipelines. For the modeling of interactions between the driven pipelines and the surrounding rock mass, 12-node interface elements were placed along the outer periphery of the driven pipelines. A set

of general fixities were automatically generated and imposed along the boundaries of the model. The boundary conditions are as follows:

- Rollers were applied to vertical boundary faces normal to the x - and y -directions, restricting horizontal movements in the x - and y -directions respectively, while allowing for vertical movements;
- Full fixities were applied to the bottom boundary, restricting movements in all directions;
- The ground surface (top boundary) was free to move in all directions.

3.2. Rock and Soil Mass Properties

Rock masses for the studied drives were modeled using the elastic-perfectly plastic Mohr-Coulomb (MC) failure criterion, together with the peak tangential strength parameters, $c'_{t,p}$ and $\phi'_{t,p}$ developed in the earlier comparison paper [1]. The MC failure criterion was used to simulate the constitutive behavior of the rock mass. These MC strength parameters are used in the Pellet-Beaucour and Kastner [16] jacking force model, which is given as

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

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

The Pellet-Beaucour and Kastner jacking force model was developed for drives traversing sands and clays [16]. It considers soil arching phenomenon; therefore, it is dependent on MC soil strength parameters. This would be applicable to the ‘*soft rock*’ encountered in the current study, justifying the need to develop strength parameters $c'_{t,p}$ and $\phi'_{t,p}$ from the direct shear tests performed earlier [1]. This would allow for an assessment of the use of the tangential strength parameters in numerical modeling of the surrounding rock mass, as well

as in the backanalysis of jacking forces, which was performed earlier in Ong and Choo [1]. Peak strength properties were used for both backanalysis and numerical modeling due to the consideration of arching in the Pellet-Beaucour and Kastner [16] jacking force model. During the reduction in soil stresses due to tunneling, arching is initiated by a vertical slippage of the overburden rock or soil mass. The initial vertical slip of rock or soil into the excavated tunnel occurs at low displacements. Hence, the use of peak strength parameters, $c'_{t,p}$ and $\phi'_{t,p}$ for the rock mass was justified. The Pellet-Beaucour and Kastner [16] model assumed uniform isotropic conditions for the backanalysis of jacking forces, i.e. $K_0 = 1$. Therefore, in order to further facilitate parity between the backanalysis of jacking forces and the numerical analysis hereinafter, isotropic conditions were assumed for the overburden soil and rock mass surrounding the tunnel [2].

The stiffnesses of the modelled rock masses were developed from the observations of RQD values being predominantly zero for typically highly weathered and fractured rock masses, such as the Tuang Formation. This allowed for the estimation of rock mass stiffness using the following equations by Bieniawski [17].

$$E_M = 10^{(RMR-10)/40} \quad (3)$$

$$RMR = 9 \ln Q + 44 \quad (4)$$

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \quad (5)$$

where E_M is the stiffness of the rock mass; RMR is the rock mass rating [18]; and Q is the rock mass quality. Q is a function of various rock joint properties including RQD [19], J_n which is the joint set number; J_r which is the joint roughness number; J_a which is the joint alteration number; J_w which is the joint water reduction number; and SRF which is the stress reduction factor [19].

For estimating Q , the values for relevant parameters were difficult to determine due to the absence of any exposed rock outcrops, or rock faces during shaft construction. These exposed rock faces were quickly shotcreted in order to mitigate water ingress, which would have otherwise caused ground surface deformation [2]. This prevented joint directions from being thoroughly studied. With this limited information, assumptions had to be made in estimating the Q rating, and subsequently the stiffness of the rock mass, E_M . These assumptions are shown in **Table 2**. The resulting rock modulus value of 7.1 GPa was applied to the numerical modeling of the highly fractured lithologies with homogeneously and consistently very poor quality as described in the Q system [17]. The overburden rock mass and soil mass properties are shown in **Tables 3, 4 and 5** for Drives A, B and C respectively.

3.3. Pipeline and Rock-pipe Interface Properties

The concrete pipelines were modelled as 20 m long hollow cylinders, longitudinal in the y -direction. The geometry of the modeled pipeline was created by initially generating a circular polycurve of a diameter appropriate to the respective pipe-jacking drives. The diameters of the modeled pipelines are shown in **Tables 3, 4 and 5** for Drives A, B and C, respectively. The polycurve was positioned on a vertical boundary. The polycurve was subsequently extruded 20 m in the y -direction, creating a hollow cylinder which had both ends flushed with vertical model boundaries. No tunnel face was considered in this simulation since the earlier study [1] was focused on studying frictional jacking forces. This was reasonable since frictional jacking forces increase with drive span, and are usually the major component in jacking forces.

The rock-pipe interface was modelled using 12-node interface elements. The interface elements are pairs of nodes, thus ensuring compatibility with the 6-noded triangular faces of both soil and plate elements. The use of finite elements for simulating the behavior of highly weathered lithologies has been mentioned previously, and has been used successfully

when applied to tunnels in highly weathered rocks [3-7, 20]. To simulate the longitudinal displacement of the concrete pipeline in the bored tunnel, it was necessary to nullify the effects of the boundary fixities at the pipe ends. This was achieved through the use of user-defined prescribed displacements at the ends of the pipelines. The application of user-defined prescribed displacements at any point along the boundaries prevail the effects of any automatically generated boundary fixities [21]. Prescribed displacements allowed for the measurement of reaction forces, ΣF_y , the use of which will be explained, hereinafter.

The modeled concrete pipelines were regarded as rigid bodies to allow for the segregation of compressive axial forces in the modeled pipelines from the pipe-rock shear stresses generated at the rock-pipe interfaces. The treatment of the modeled pipelines as rigid bodies was justifiable since the concrete pipelines were of a higher stiffness in relation to the less rigid pipe-rock interface surrounding the modeled pipelines [25-27].

The pipe-rock interface was simulated using the Mohr-Coulomb model, which allowed for direct prescription of rock-pipe interface strength properties from the earlier back-analyses of frictional jacking forces [1]. The interfacial strength properties are given as

$$\tau_i = \sigma'_n \tan \phi_i \quad (6)$$

$$\phi_i = \tan^{-1}(\mu_{avg}) \quad (7)$$

where, τ_i is the rock-pipe interface shear strength; σ'_n is the effective normal stress acting on the rock-pipe interface; ϕ_i is the interface friction angle; and μ_{avg} is the back-analyzed average frictional coefficient, obtained from earlier back-analyses of jacking forces [1].

In addition to the treatment of concrete pipelines as rigid bodies and the use of user-defined prescribed displacements, it was necessary to assign suitable values to the stiffness of the pipe-rock interface elements, to facilitate the segregation of the pipe and rock elements, thus allowing for longitudinal displacement of the concrete pipeline. This was achieved by prescribing a low value to interface modulus, E_{inter} . The use of low interface moduli has been

applied to simulations of slippage between two materials. Yin et. al [25] and Wang and Wang [26] adopted low tangential interface modulus values to enable large interface deformations for studies on pull-out tests. For the current study on modeling the large deformation of pipe-rock interfaces, an interface modulus of E_{inter} of 1,000 kPa was used as it was still significantly small relative to the moduli of the concrete pipeline and surrounding rock mass. This method of modelling the rock-pipe interface was also used successfully by Ong and Choo [20].

3.4. Calculation Phases in Modeling of Pipe-jacking Process

For simulating the propulsion of the concrete pipeline through the rock mass, modeling of the pipe-jacking process was performed in three consecutive phases.

- Stage 1 – Application of initial stress conditions
- Stage 2 – Wish-in-place pipeline and activation of rock-pipe interface elements
- Stage 3 – Application of displacements at tail-end of pipeline

In Stage 1, material properties for the soil and rock masses, groundwater levels and K_0 condition were assigned to the model for the generation of initial stresses under greenfield conditions. Model boundary conditions were also activated. Plate elements, interface elements, prescribed displacements and prescribed loads were deactivated in Stage 1. These elements and loads would only be activated in subsequent stages.

Following the generation of initial stresses in Stage 1, the ‘wish-in-place’ 20 m long concrete pipeline was simulated in Stage 2 through the activation of plate elements. This was accompanied with the deactivation of soil elements and water conditions within the pipeline annulus. Rock-pipe interface elements were also activated concurrently with the plate elements.

In Stage 3, the user-defined displacements were prescribed horizontally in the y-direction and applied along the periphery of the tail end of the modeled pipeline. This was coupled

with the application of zero-valued prescribed displacements at the front end of the pipe to mitigate the effect of boundary fixities. These prescribed displacements were free in the y -direction (longitudinal to the modeled pipeline), thus allowing for the movement of the modeled pipeline beyond the model boundaries [21].

4. VALIDATION OF NUMERICAL ANALYSIS TECHNIQUE

In order to assess the applicability of direct shear test results to the assessment of pipe-jacking forces, it was necessary to validate the earlier described numerical analysis technique. The validation was also necessary due to the limited use of prescribed displacements in modeling the advance of a pipeline during pipe-jacking [12, 15, 16]. This validation was performed by applying the lower bound theorem to three baseline cases.

The lower bound theorem is widely used to verify that static equilibrium has been achieved in a system. This leads to a safe solution for the simulated geotechnical problem. Some geotechnical problems which have used the lower bound theorem include ground anchors [15], underground excavations [31-34], and pile performance [35-37]. According to Randolph and Houlsby [32], a lower bound solution is achieved when the following criteria have been met:

- The lower bound on the applied loads will cause collapse of perfectly plastic material with an associated flow rule;
- The stress field is in equilibrium with the applied loads and
- The stress field does not violate the yield criterion for the material.

In addition to these criteria, Potts and Zdravković [34] stated that such a lower bound solution should be found in a stress field that is statically admissible, i.e. a system in equilibrium. Applying these criteria to the earlier described numerical analysis technique would necessitate that the applied jacking loads were in equilibrium with the resulting pipe-rock interface shear stresses. This equilibrium needs to be fulfilled at all calculation stages.

For this validation, the lower bound theorem was applied to Drive A, under which three cases were assessed:

- Case 1 (parent) - No displacement or loading was applied to the modelled pipe.
- Case 2 (child) - Displacement was prescribed to the modelled pipe.
- Case 3 (child) - Loading was applied to the modelled pipe.

Case 1 was performed as a baseline for when displacements (Case 2) or loads (Case 3) were applied to the pipe. No interface shear stresses were expected to develop in Case 1. Case 2 was performed to verify that the modelling technique (including the earlier described elements, parameters and values, and the use of prescribed displacements to displace the modelled pipeline) was suitable for mobilizing interfacial shear stresses, as described in the aforementioned literature [17-19]. Case 3 was performed as a compliance check of Case 2, to verify that the numerical model was valid for the application of applied loads to displace the modelled pipeline. The successful modelling of Cases 2 and 3 would show that both prescribed displacements and applied loads were suitable in simulating the driving of the modelled pipelines. Cases 2 and 3 also served to demonstrate that the model was a lower bound solution [32].

4.1. Case 1 – No Displacements or Loading Applied to Modeled Pipe

In Case 1, displacements and loads were not applied to the modeled pipeline. This case was performed in order to establish a reference point for Cases 2 and 3, where displacements and loads would be applied. Inspection of the pipe-rock interface elements showed that the elements were all in an elastic state (see **Fig. 2**). This was verified by the distribution of constant interface shear stress, τ_1 longitudinally along (see **Fig. 3**) and radially around the pipeline (see **Fig. 4**). The pipe-rock interface shear stresses were uniformly distributed at $\tau_1 = 0$. Minimal shear stresses were observed at the ends of the modeled pipeline. This was due to end effects at the model boundaries.

With no displacements or loading applied, the pipe-rock interface elements exhibited the anticipated behavior. The results from Case 1 would form the baseline for subsequent validation of the modeling technique under prescribed displacements (Case 2) and applied loads (Case 3).

4.2. Case 2 – Displacements Prescribed to Modeled Pipe

Case 2 follows from the completion of Case 1. A uniform prescribed displacement of 100 mm was applied along the pipe circumference at the tail end of the modeled pipeline (see **Fig. 5**). The prescribed displacement was applied in the horizontal y -direction to allow for the modeled pipeline to displace as a rigid body longitudinally. The prescribed displacement of 100 mm was sufficient for the rock-pipe interface elements to reach a state of elastic yield, as shown by the pipe-rock interface elements achieving plasticity (see **Fig. 6**). During the backanalysis of jacking forces using the Pellet-Beaucour and Kastner [16] jacking force model, the rate of increase in jacking forces remained uniform under homogenous geological conditions. Hence, a modeled pipeline of a uniform diameter would exhibit uniform distributions of longitudinal and radial pipe-rock interface shear stresses, τ_1 .

After the application of prescribed displacements, the rock-pipe interface shear stresses, τ_1 were uniformly distributed radially (see **Fig. 7**) and longitudinally (see **Fig. 8**) at $\tau_1 = 65$ kN/m². From Eqs. 6 and 7, an assessment of τ_1 was performed. Based on $\sigma'_n = 172$ kN/m² and $\mu_{avg} = 0.356$, the resulting interface shear strength, τ_1 was 61 kN/m², which was approximately equal to the resulting interface shear stresses, $\tau_1 = 65$ kN/m² from numerical analysis. This reconfirmed the earlier observations from **Fig. 6** that the rock-pipe interface elements had yielded.

The reaction force at yield in the y -direction, $\Sigma F_{y,yield}$ was 7,233 kN. By integrating τ_1 along the outer surface area of the pipe, the total jacking force due to pipe-rock interface shear stresses, $F_{\tau,cal}$ can be given as

$$F_{\tau,cal} = \tau_1 \pi D_e L \quad (8)$$

where, L is the length of the modelled pipeline; and D_e is the external diameter of the pipe, which was 1.78 m for Drive A. This resulted in $F_{\tau,cal} = 7,270$ kN, which was comparable to the yield reaction force, $\Sigma F_{y,yield}$ of 7,233 kN. The slight discrepancy was attributed to the end effects (see **Fig. 8**), since τ_1 was measured at the mid-span of the modeled pipeline (see **Fig. 7**). Despite the minor discrepancy, it can be seen that the reaction forces due to the prescribed displacements were equivalent to the total jacking forces due to pipe-rock interface shear stresses, i.e. $\Sigma F_y \approx F_{\tau,cal}$. This implied that all the loads generated due to the prescribed displacements were borne by the pipe-rock interface elements and that there was no end bearing resistance incurred at the pipe ends, which was similarly observed in other simulations of pipe-jacking drives through highly weathered and fractured lithologies [20]. This further confirmed the successful release of model boundary fixities at the pipe ends. The equivalence between ΣF_y and $F_{\tau,cal}$ showed that equilibrium had been achieved between the prescribed displacements, the incurred reaction loads and the rock-pipe interface shear stresses, demonstrating compliance to the lower bound theorem. This meant that prescribed displacements could be used to obtain a safe and valid solution for the assessment of pipe-jacking forces.

4.3. Case 3 – Loading Applied to Modeled Pipe

Case 3 followed from the completion of Case 1. In contrast with Case 2 where prescribed displacements were used, Case 3 involved the use of applied loads to propel the modeled pipeline forward. Case 3 was performed as a compliance model to show that static equilibrium of the system could also be achieved through the use of applied loads. Successful modeling of Case 3 would also justify the use of reaction force, ΣF_y for the assessment of jacking forces. From the results in Case 2, the reaction force at yield was $\Sigma F_{y,yield} = 7,233$ kN.

Hence, in order to ensure model convergence, the applied load, $F_{y,app}$ in Case 3 could not exceed 7,233 kN, i.e. $F_{y,app} < \Sigma F_{y,yield}$. A uniform line load of 100 kN/m was applied along the circumference at the tail end of the pipe, corresponding with a total applied load, $F_{y,app}$ of 559 kN (see **Fig. 9**). As the Pellet-Beaucour jacking force model is based on the strength of the pipe-rock interface, sufficient displacement was determined by the attainment of elastic yield. Therefore, it was necessary to show that under the prescribed displacement, a uniform distribution of interface shear stresses was attained. The distribution of interface shear stresses longitudinally (see **Fig. 10**) and circumferentially (see **Fig. 11**) were uniform at $\tau_l = 5$ kN/m². Using Eqs. 6 and 7, integrating the pipe-rock interface shear stresses along the modeled pipeline resulted in a total jacking force, F_{total} of 559 kN, which was in equilibrium with $F_{y,app}$. The achievement of static equilibrium in Case 3 between the applied loads, the reaction loads and the pipe-rock interface shear stresses justified the use of ΣF_y in assessing pipe-jacking forces.

4.4. Post-analysis of Modeling Results with Consideration for Arching Effect

The use of a displaced ‘wish-in-place’ pipe in the numerical analysis has been reliably demonstrated. The effects of arching on pipe-jacking forces remain to be addressed. From the earlier companion paper [1], it was found that the accrual of pipe-jacking forces through highly fractured rock masses was dependent on arching. The excavation of a tunnel would instigate the occurrence of arching in the upper soil or ‘*soft rock*’ mass, causing a redistribution of in-situ soil stresses and resulting in the reduction of stresses acting on the pipeline. The use of a ‘wish-in-place’ pipeline in the numerical model implied that there was no overcut region separating the rock mass from the pipeline. Therefore, it was proposed to apply an arching ratio manually for the consideration of arching effects on the results from numerical analysis.

Arching ratios have been well-used to quantify the effects of arching. Based on the classical trapdoor experiment by Terzaghi [35], McNulty [36] expressed arching ratio as the ratio of load on the yielded trapdoor to that when the trapdoor is undeflected. The use of such an arching ratio can be useful in quantifying states of arching, i.e. an arching ratio of less than 1.0 would imply the occurrence of active arching, while an arching ratio of greater than 1.0 would indicate the presence of passive arching. This fundamental understanding led to the development of arching ratios primarily for studies conducted on embankment fills [41-44].

For buried structures, Lee et. al [41] expressed arching ratio with the following equation

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

where AR is the arching ratio; $\Delta\sigma_v$ is the change in vertical stress due to tunneling; and σ_{vo} is the initial overburden pressure. From this, AR could be applied to pipe-jacking by considering the normal stresses acting on the pipeline. Thus, the arching ratio was proposed to be

$$AR = \frac{\sigma_{EV} + \frac{\gamma D_e}{2}}{\gamma \left(h + \frac{D_e}{2} \right)} \times 100\% \quad (8)$$

This arching ratio, AR was based on an average point, at the depth of the springline, denoted by the component $\frac{\gamma D_e}{2}$. By applying the proposed arching ratio, AR (Eq. 8) to the rock-pipe interface shear stresses, the jacking force determined from τ_1 is given as

$$JF_{\tau,cal} = AR \times \tau_1 \pi D_e \quad (9)$$

The jacking forces determined from τ_1 can be further verified against the jacking forces determined from the reaction loads, ΣF_y , which is given as

$$JF_{Fy,cal} = AR \times \frac{\sum F_y}{L} \quad (10)$$

In order to assess the accuracy of the numerical modeling technique, the calculated jacking forces, $JF_{\tau,cal}$ and $JF_{F_y,cal}$ were compared against the field measurements of jacking forces, JF_{meas} .

5. RESULTS OF NUMERICAL ANALYSIS

Table 6 shows the results of the numerical modeling of Drives A, B and C from the earlier companion paper [1], with the variations of these results against the measured jacking forces. In the modeling of the studied Drives A, B and C, prescribed displacements were used to drive the modelled pipelines. For Drive A, the back-analyzed σ_{EV} was 12.5 kN/m^2 . Using **Eq. 8**, the resulting arching ratio, AR was 8.00%. This meant that the strength of the rock mass contributed to a 92% reduction in normal loads acting on the pipe from the surrounding soil. From numerical analysis, the interface shear stress, τ_1 was 65 kN/m^2 . Using **Eq. 9**, the jacking forces computed from τ_1 , $JF_{\tau,cal}$ was 29.0 kN/m . Comparing this to the measured jacking force, JF_{meas} of 37.2 kN/m , the calculated $JF_{\tau,cal}$ underestimated JF_{meas} by 22.2%. The reaction force, ΣF_y measured from numerical analysis was $7,233 \text{ kN}$. Using **Eq.10**, the jacking forces calculated from ΣF_y , $JF_{F_y,cal}$ was 28.9 kN/m , which underestimated the measured jacking forces by 22.3%. Thus, the results from numerical analysis of Drive A agreed reasonably well with the measured jacking forces.

For Drives B and C, the back-analyzed σ_{EV} values were negative. For the determination of arching ratios, AR , the negative σ_{EV} were adjusted to be equal to zero [2, 42]. The AR values for Drives B and C were 3.75% and 2.90%, respectively. In comparison with the arching ratio for Drive A, the AR values for Drives B and C were smaller, indicating that arching was more significant in the latter drives. The heightened arching contributed to the measured jacking forces, which were smaller in Drives B and C as compared to those measured from Drive A (see **Table 6**).

From the numerical modelling of Drive B, the pipe-rock interface shear stresses, τ_1 were 104 kN/m^2 , resulting in a calculated jacking force, $JF_{\tau,cal}$ of 17.5 kN/m . This was a 15.4% underestimation of the measured jacking forces of 20.7 kN/m . The reaction load, ΣF_y was $8,753 \text{ kN}$. The resulting calculated jacking force, $JF_{Fy,cal}$ was 16.4 kN/m , which was an underestimation of field measured jacking forces of 20.8%. For Drive C, the pipe-rock interface shear stresses, τ_1 were 159 kN/m^2 . The jacking forces, $JF_{\tau,cal}$ calculated from these interface shear stresses was 20.8 kN/m , which was a 12.2% underestimation of the measured jacking forces. From the reaction force, ΣF_y of $12,850 \text{ kN}$, the calculated jacking force, $JF_{Fy,cal}$ was 18.7 kN/m , which underestimated the measured jacking forces by 21.1%.

It was useful to compare the results obtained from interface shear stresses, τ_1 and those obtained from reaction loads, $JF_{Fy,cal}$. The results from τ_1 were obtained from the mid-span of the modeled pipes, while $JF_{Fy,cal}$ was a measure of all the pipe-rock interface shear loads accrued along the pipeline, including end effects. Therefore, difference between the results obtained from interface shear stresses, τ_1 and those obtained from reaction loads, $JF_{Fy,cal}$ could be used to assess the influence of end effects. This difference is given by

$$\text{Difference in variations} = \frac{JF_{\tau,cal} - JF_{Fy,cal}}{JF_{meas}} \times 100\% \quad (11)$$

The differences in variations were generally small, with the highest difference exhibited in Drive C at 8.9%. This implied that end effects had minimal influence on the accuracy of the numerical analysis.

Generally, the resulting τ_1 and ΣF_y from numerical analysis underestimated field measurements of jacking forces within a range of 15% to 22%. This was attributed to the use of the lower bound approach adopted in the current study for the numerical modeling of the studied pipe-jacking drives. Furthermore, the peak tangential strength parameters, $c'_{t,p}$ and $\phi'_{t,p}$ were developed from direct shear testing of reconstituted tunneling rock spoils in order

to simulate the highly fractured and highly weathered nature of the in-situ ‘*soft rock*’, as described in the earlier companion paper [1]. These rock spoils were crushed from the traversed lithologies during pipe-jacking, thus diminishing the natural cementation of the rock mass and resulting in lower bound solutions [20]. Additionally, the assessment of pipe-jacking forces depended on the use of the Pellet-Beaucour and Kastner [16] jacking force model, which required the assumption of average uniform conditions along the modeled pipelines. This assumption was applied to the strength and stiffness properties of the modeled rock mass, the pipeline and the pipe-rock interface elements. This idealization consequently required the use of the ‘wish-in-place’ technique for introducing the pipeline into the model, which contributed to the underestimation of jacking forces. Despite these underestimations, the jacking forces computed from the numerical analysis and the measured jacking forces correlated reasonably well, thus demonstrating the reliability of the interpreted strength parameters, $c'_{t,p}$ and $\phi'_{t,p}$ through numerical modelling, as well that of the back-analyzed parameters, μ_{avg} and σ_{EV} , through the consideration of arching effect.

6. CONCLUSIONS

From the earlier companion paper [1], three pipe-jacking drives traversing highly fractured lithologies were studied. The drives negotiated lithological units of shale (Drive A) and interbedded lithological units of metagraywacke – siltstone (Drive B) and greywacke – phyllite (Drive C) origins. From each of the studied pipe-jacking drives, tunneling rock spoils were collected, reconstituted and subjected to direct shear testing. Peak tangential strength parameters $c'_{t,p}$ and $\phi'_{t,p}$ were derived from the direct shear test results and used for the backanalysis of the measured jacking forces, from which μ_{avg} and σ_{EV} were developed.

Numerical modeling of the pipe-jacking drives was performed to ascertain the applicability of the parameters, $c'_{t,p}$, $\phi'_{t,p}$, μ_{avg} and σ_{EV} resulting from the earlier backanalysis of jacking forces. The developed strength parameters, $c'_{t,p}$ and $\phi'_{t,p}$ were applied to the

modelled rock mass surrounding the modeled pipelines. The concrete pipelines were modeled as ‘wish-in-place’, with an increased material modulus. It was necessary to model the pipelines as rigid bodies in order to segregate the influence of axial loads from the pipe-rock interface shear stresses. The pipe-rock interface elements were modeled as cohesionless frictional Mohr-Coulomb materials, with the strength parameters derived from the backanalyzed μ_{avg} in Ong and Choo [1]. A low material stiffness was used for the pipe-rock interface elements to facilitate the longitudinal displacement of the rigid pipeline. For the consideration of arching effects in subsequent post-analysis, an arching ratio was developed from σ_{EV} .

The modeling technique was validated in accordance with the lower bound theorem. When applying either prescribed displacements or loads for the propulsion of the modeled pipeline, the forces derived from the integration of interface shear stresses, τ_1 were in static equilibrium with the reaction loads, ΣF_y . Thus, the stress field in the model was considered to be statically admissible, allowing for the modeling technique to be used for subsequent simulation of the studied pipe-jacking drives.

The jacking forces derived from numerical analysis underestimated the measured jacking forces within a range of 15% to 22%. This was due to the use of the lower bound theorem for the assessment of the numerical modeling technique. Furthermore, the cementation of the tunneling rock spoils was reduced through crushing of the rock mass. The idealization of an average pipeline traversing homogeneous geological conditions contributed to the underestimation of jacking forces. However, the jacking forces obtained from τ_1 and ΣF_y in the numerical analysis were considered to be in reasonable agreement with the measured jacking forces.

The findings showed that reliable strength parameters can be developed from the direct shear testing of reconstituted tunneling rock spoils to be subsequently used for the assessment

of jacking forces for pipe-jacking drives negotiating highly weathered and fractured ‘soft rock’ such as those encountered in the Tuang Formation in Kuching, Malaysia. This reliability has been demonstrated through numerical modeling of the pipe-jacking drives using results obtained from direct shear tests from the earlier companion paper [1]. This method of back-analysis could allow for the prediction of jacking forces in highly weathered, highly fractured lithologies for future pipe-jacking drives, through the use of direct shear tests on reconstituted crushed rock.

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8. REFERENCES

- [1] Ong, D.E.L. and Choo, C.S., *Assessment of non-linear rock strength parameters for the estimation of pipe-jacking forces. Part 1. Direct shear testing and backanalysis*. Engineering Geology, 2018, <https://doi.org/10.1016/j.enggeo.2018.07.013>.
- [2] Choo, C.S. and Ong, D.E.L. *Evaluation of pipe-jacking forces based on direct shear testing of reconstituted tunneling rock spoils*. Journal of Geotechnical & Geoenvironmental Engineering, 2015, **141**(10), [https://doi.org/10.1061/\(ASCE\)GT.1943-5606.0001348](https://doi.org/10.1061/(ASCE)GT.1943-5606.0001348).
- [3] Basarir, H., Ozsan, A., and Karakus, M., *Analysis of support requirements for a shallow diversion tunnel at Guledar dam site, Turkey*. Engineering Geology, 2005, **81**(2): p. 131-145, <https://doi.org/10.1016/j.enggeo.2005.07.010>.
- [4] Genis, M., Basarir, H., Ozarslan, A., Bilir, E., and Balaban, E., *Engineering geological appraisal of the rock masses and preliminary support design, Dorukhan Tunnel, Zonguldak, Turkey*. Engineering Geology, 2007, **92**(1): p. 14-26, <https://doi.org/10.1016/j.enggeo.2007.02.005>.

- [5] Pusch, R. and Börgesson, L., *Strain parameters of discontinuities in rock for finite element calculation*. Engineering Geology, 1998. **49**(3): p. 201-206, [https://doi.org/10.1016/S0013-7952\(97\)00050-1](https://doi.org/10.1016/S0013-7952(97)00050-1).
- [6] Zhu, W., Li, S., Li, S., Chen, W., and Lee, C.F., *Systematic numerical simulation of rock tunnel stability considering different rock conditions and construction effects*. Tunnelling and Underground Space Technology, 2003. **18**(5): p. 531-536, [https://doi.org/10.1016/S0886-7798\(03\)00070-1](https://doi.org/10.1016/S0886-7798(03)00070-1).
- [7] Li, C., Zhong, Z., Liu, X., Tu, Y., and He, G., *Numerical simulation for an estimation of the jacking force of ultra-long-distance pipe jacking with frictional property testing at the rock mass-pipe interface*. Tunnelling and Underground Space Technology, 2019. **89**: p. 205-221, <https://doi.org/10.1016/j.tust.2019.04.004>.
- [8] Barla, M., Camusso, M., and Aiassa, S., *Analysis of jacking forces during microtunnelling in limestone*. Tunnelling and Underground Space Technology, 2006. **21**(6): p. 668-683, <https://doi.org/10.1016/j.tust.2006.04.002>.
- [9] Itasca Consulting Group Inc., *FLAC Fast Lagrangian Analysis of Continua, version 4.0. User's Guide*. 2001, Minneapolis.
- [10] Itasca Consulting Group Inc., *UDEC Universal Distinct Element Code, version 3.10. User's Guide*. 1999, Minneapolis.
- [11] Shou, K.J., Yen, J., and Liu, M., *On the frictional property of lubricants and its impact on jacking force and soil-pipe interaction of pipe-jacking*. Tunnelling and Underground Space Technology, 2010. **25**(4): p. 469-477, <https://doi.org/10.1016/j.tust.2010.02.009>.
- [12] Yen, J.H. and Shou, K.J., *Numerical simulation for the estimation the jacking force of pipe jacking*. Tunnelling and Underground Space Technology, 2015. **49**: p. 218-229, <http://dx.doi.org/10.1016/j.tust.2015.04.018>.
- [13] Dounias, G.T. and Potts, D.M., *Numerical analysis of drained direct and simple shear tests*. Journal of Geotechnical Engineering, 1993. **119**(12): p. 1870-1891, [https://doi.org/10.1061/\(ASCE\)0733-9410\(1993\)119:12\(1870\)](https://doi.org/10.1061/(ASCE)0733-9410(1993)119:12(1870)).

- [14] Hu, L. and Pu, J., *Testing and modeling of soil-structure interface*. Journal of Geotechnical and Geoenvironmental Engineering, 2004. **130**(8): p. 851-860,
[https://doi.org/10.1061/\(ASCE\)1090-0241\(2004\)130:8\(851\)](https://doi.org/10.1061/(ASCE)1090-0241(2004)130:8(851)).
- [15] Merifield, R.S. and Sloan, S.W., *The ultimate pullout capacity of anchors in frictional soils*. Canadian Geotechnical Journal, 2006. **43**(8): p. 852-868, <https://doi.org/10.1139/t06-052>.
- [16] Pellet-Beaucour, A.L. and Kastner, R., *Experimental and analytical study of friction forces during microtunneling operations*. Tunnelling and Underground Space Technology, 2002. **17**(1): p. 83-97, [https://doi.org/10.1016/S0886-7798\(01\)00044-X](https://doi.org/10.1016/S0886-7798(01)00044-X).
- [17] Bieniawski, Z.T., *Engineering rock mass classifications : a complete manual for engineers and geologists in mining, civil, and petroleum engineering*. 1989 Wiley.
- [18] Bieniawski, Z.T., *Tunnel design by rock mass classifications*, in *US Army Corps of Engineers Technical Report GL-79-19*. 1979, U.S. Waterways Experimental Station: Vicksburg.
- [19] Barton, N., Lien, R., and Lunde, J., *Engineering classification of rock masses for the design of tunnel support*. Rock mechanics, 1974. **4**(4): p. 189-236, <https://doi.org/10.1007/BF01239496>.
- [20] Ong, D.E.L. and Choo, C.S., *Back-analysis and finite element modeling of jacking forces in weathered rocks*. Tunnelling and Underground Space Technology, 2016. **51**: p. 1-10,
<https://doi.org/10.1016/j.tust.2015.10.014>.
- [21] Plaxis bv, *PLAXIS 3D 2015 Manual*. 2013, Delft, The Netherlands.
- [22] Wang, D., Hu, Y., and Randolph, M.F., *Three-dimensional large deformation finite-element analysis of plate anchors in uniform clay*. Journal of Geotechnical and Geoenvironmental Engineering, 2009. **136**(2): p. 355-365, [https://doi.org/10.1061/\(ASCE\)GT.1943-5606.0000210](https://doi.org/10.1061/(ASCE)GT.1943-5606.0000210).
- [23] Zhang, X., Krabbenhoft, K., Pedroso, D.M., Lyamin, A.V., Sheng, D., da Silva, M.V., and Wang, D., *Particle finite element analysis of large deformation and granular flow problems*. Computers and Geotechnics, 2013. **54**: p. 133-142,
<https://doi.org/10.1016/j.compgeo.2013.07.001>.

- [24] Tian, Y., Cassidy, M.J., Randolph, M.F., Wang, D., and Gaudin, C., *A simple implementation of RITSS and its application in large deformation analysis*. Computers and Geotechnics, 2014. **56**: p. 160-167, <https://doi.org/10.1016/j.compgeo.2013.12.001>.
- [25] Yin, Z.-Z., Zhu, H., and Xu, G.-H., *A study of deformation in the interface between soil and concrete*. Computers and Geotechnics, 1995. **17**(1): p. 75-92, [https://doi.org/10.1016/0266-352X\(95\)91303-L](https://doi.org/10.1016/0266-352X(95)91303-L).
- [26] Wang, X. and Wang, L.B., *Continuous interface elements subject to large shear deformations*. International Journal of Geomechanics, 2006. **6**(2): p. 97-107, [https://doi.org/10.1061/\(ASCE\)1532-3641\(2006\)6:2\(97\)](https://doi.org/10.1061/(ASCE)1532-3641(2006)6:2(97)).
- [27] Smith, C.C., *Limit loads for an anchor/trapdoor embedded in an associative Coulomb soil*. International Journal for Numerical and Analytical Methods in Geomechanics, 1998. **22**(11): p. 855-865, [https://doi.org/10.1002/\(SICI\)1096-9753\(199811\)22:11%3C855::AID-NAG945%3E3.0.CO;2-4](https://doi.org/10.1002/(SICI)1096-9753(199811)22:11%3C855::AID-NAG945%3E3.0.CO;2-4).
- [28] Smith, C.C., *Limit loads for a shallow anchor/trapdoor embedded in a non-associative Coulomb Soil*. Géotechnique, 2012. **62**(7): p. 563-571, <https://doi.org/10.1680/geot.10.P.136>.
- [29] Davis, E.H., Gunn, M.J., Mair, R.J., and Seneviratne, H.N., *The stability of shallow tunnels and underground openings in cohesive material*. Géotechnique, 1980. **30**(4): p. 397-416, <https://doi.org/10.1680/geot.1980.30.4.397>.
- [30] Augarde, C.E., Lyamin, A.V., and Sloan, S.W., *Prediction of undrained sinkhole collapse*. Journal of Geotechnical and Geoenvironmental Engineering, 2003. **129**(3): p. 197-205, [https://doi.org/10.1061/\(ASCE\)1090-0241\(2003\)129:3\(197\)](https://doi.org/10.1061/(ASCE)1090-0241(2003)129:3(197)).
- [31] Georgiadis, K., Sloan, S.W., and Lyamin, A.V., *Undrained limiting lateral soil pressure on a row of piles*. Computers and Geotechnics, 2013. **54**: p. 175-184, <http://dx.doi.org/10.1016/j.compgeo.2013.07.003>.
- [32] Randolph, M.F. and Houlsby, G.T. *The limiting pressure on a circular pile loaded laterally in cohesive soil*. Géotechnique, 1984. **34**, 613-623 DOI: <https://doi.org/10.1680/geot.1984.34.4.613>.

- [33] Finno, R.J., Lawrence, S.A., Allawh, N.F., and Harahap, I.S., *Analysis of performance of pile groups adjacent to deep excavation*. Journal of Geotechnical Engineering, 1991. **117**(6): p. 934-955, [https://doi.org/10.1061/\(ASCE\)0733-9410\(1991\)117:6\(934\)](https://doi.org/10.1061/(ASCE)0733-9410(1991)117:6(934)).
- [34] Potts, D.M. and Zdravković, L., *Finite element analysis in geotechnical engineering: theory*. Vol. 1. 1999: Thomas Telford Services Limited.
- [35] Terzaghi, K., *Stress distribution in dry and in saturated sand above a yielding trap-door*, in *Proc., 1st Intl. Conf. on Soil Mechanics and Foundation Engineering*. 1936: Graduate School of Engineering, Harvard Univ., Cambridge, Massachusetts, USA. p. 307-311.
- [36] McNulty, J.W., *An experimental study of arching on sand*, in *US Army Corps of Engineers Technical Report I-674*. 1965, U.S. Waterways Experimental Station: Vicksburg.
- [37] Han, J. and Gabr, M., *Numerical analysis of geosynthetic-reinforced and pile-supported earth platforms over soft soil*. Journal of Geotechnical and Geoenvironmental Engineering, 2002. **128**(1): p. 44-53, [https://doi.org/10.1061/\(ASCE\)1090-0241\(2002\)128:1\(44\)](https://doi.org/10.1061/(ASCE)1090-0241(2002)128:1(44)).
- [38] Ye, G., Zhang, Z., Han, J., Xing, H., Huang, M., and Xiang, P., *Performance evaluation of an embankment on soft soil improved by deep mixed columns and prefabricated vertical drains*. Journal of Performance of Constructed Facilities, 2012. **27**(5): p. 614-623, [https://doi.org/10.1061/\(ASCE\)CF.1943-5509.0000369](https://doi.org/10.1061/(ASCE)CF.1943-5509.0000369).
- [39] Borges, J.L. and Marques, L. O., *Geosynthetic-reinforced and jet grout column-supported embankments on soft soils: Numerical analysis and parametric study*. Computers and Geotechnics, 2011. **38**(7): p. 883-896, <https://doi.org/10.1016/j.compgeo.2011.06.003>.
- [40] Deb, K., *A mathematical model to study the soil arching effect in stone column-supported embankment resting on soft foundation soil*. Applied Mathematical Modelling, 2010. **34**(12): p. 3871-3883, <https://doi.org/10.1016/j.apm.2010.03.026>.
- [41] Lee, C.J., Wu, B.R., Chen, H.T., and Chiang, K.H., *Tunnel stability and arching effects during tunneling in soft clayey soil*. Tunnelling and Underground Space Technology, 2006. **21**(2): p. 119-132, <https://doi.org/10.1016/j.tust.2005.06.003>.
- [42] Terzaghi, K., *Theoretical soil mechanics*. 1943: John Wiley & Sons, Inc.

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Table 1. Parameters used in pipe-jacking force model for backanalyses of μ_{avg} [1]

Drive		Drive A	Drive B	Drive C
Geology		Shale	Metagraywacke – siltstone	Graywacke – phyllite
Length of rock cores extracted from shaft locations (m)		6.0	12.0	13.0
RQD (%)	Maximum	23	20	20
	Minimum	0	0	0
	Average	14.0	5.8	2.0
Length of rock cores with RQD = 0 (m)		0.7	8.9	11.0
External pipe diameter, D_e (m)		1.78	1.43	1.78
Soil unit weight, γ_{soil} (kN/m ³)		18	18	18
Depth to rock level, h_{soil} (m)		20	9	17
Rock unit weight, γ_{rock} (kN/m ³)		22	22	22
Height of rock cover, h_{rock} (m)		1	11	10
Tangential MC parameters obtained from direct shear testing	$c'_{t,p}$ (kN/m ²)	50.8	57.8	29.0
	$\phi'_{t,p}$ (°)	47.8	44.3	38.7
Average measured jacking forces, JF_{meas} (kN/m)		37.2	20.7	23.6
Vertical stress acting on pipe crown, σ_{EV} (kN/m ²) (see Eq. 3)		12.5	-19.9	-38.5
Back-analyzed μ_{avg} , using Eq. (1)		0.356	0.491	0.581
R^2		0.86	0.70	0.77

Table 2. Parameters used for estimating Q rating [17]

Parameter	Description	Value
Rock quality designation, RQD	Very poor	0 – 25
Joint set number, J_n	Random, heavily jointed	15
Joint roughness number, J_r	Rough	1.5
Joint alteration number, J_a	Unaltered joint walls	1.0
Joint water reduction factor, J_w	Medium inflow or pressure occasional outwash of joint fillings	1.0
Stress reduction factor, SRF	Medium stress	1.0

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

	Loose sand	Very soft clay	Loose sand	Dense sand	Medium dense sand	Very soft clay	Very hard silt	Shale	Rock-pipe interface
Materials used	Soil	Soil	Soil	Soil	Soil	Soil	Soil	Soil	Interface
Material model	MC	MC	MC	MC	MC	MC	MC	MC	MC
Depth (m)	0 to 6	6 to 10.5	10.5 to 11.5	11.5 to 13.5	13.5 to 14.9	14.9 to 19.3	19.3 to 20	20 to 30	-
Unit weight, γ (kN/m ³)	18	18	18	18	18	18	18	22	-
cohesion, c' (kPa)	6	2	6	34	19	2	107	-	-
cohesion, c' (kPa)	5	5	5	15	10	5	15	21.7 ⁽¹⁾	0
Friction angle, ϕ' (°)	28	28	28	30	28	28	33	38.7 ⁽¹⁾	19.60 ⁽²⁾
Young's modulus, E' (kPa)	10,430	2,200	10,430	59,130	33,040	2,200	117,700	7.1×10^6	1,000
Poisson's ratio, ν	0.35	0.35	0.35	0.35	0.25	0.35	0.35	0.35	0.35
Interface pipe diameter, D_e (m)	-	-	-	-	-	-	-	-	-

Note: 1. From generalized tangential interpretation of earlier direct shear tests.

2. Interface friction angle, $\phi_i = \tan^{-1}(c'_i/\sigma'_{vi})$

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

Material	Soft clay	Soft silt	Very stiff silt	Hard silt	Meta-graywacke – siltstone	Rock-pipe interface	Pipe
Elements used	Soil	Soil	Soil	Soil	Soil	Interface	Plate
Material model	MC	MC	MC	MC	MC	MC	Linear elastic
Depth (m)	0 to 2.7	2.7 to 5.5	5.5 to 7	7 to 9	9 to 25	-	-
Unit weight, γ (kN/m ³)	18	18	18	18	22	-	24
SPT 'N'	3	3	29	54	-	-	-
Cohesion, c' (kPa)	5	5	10	15	44.6 ^a	0	-
Friction angle, ϕ' (°)	28	28	28	33	39.9 ^a	26.15 ^b	-
Elastic modulus, E' (kN/m ²)	3,300	3,300	31,900	58,360	7.1×10^6	1,000	27×10^9
Poisson's ratio, ν	0.35	0.35	0.35	0.35	0.35	0.35	0.15
External pipe diameter, D_e (m)	-	-	-	-	-	-	1.43

Note: a. From generalized tangential interpretation of earlier direct shear tests.

b. Interface friction angle, $\phi_i = \tan^{-1}(\mu_{avg})$

Table 5. Material properties for finite element modeling of Drive C

Material	Loose sand	Stiff silt	Hard silt	Hard silt	Graywacke – phyllite	Rock-pipe interface	Pipe
Elements used	Soil	Soil	Soil	Soil	Soil	Interface	Plate
Material model	MC	MC	MC	MC	MC	MC	Linear elastic
Depth (m)	0 to 5	5 to 8	8 to 12.5	12.5 to 17	17 to 35	-	-
Unit weight, γ (kN/m ³)	18	18	18	18	22	-	24
SPT ‘N’	3	9	35	66	-	-	-
Cohesion, c' (kPa)	5	5	15	15	60.8 ^a	0	-
Friction angle, ϕ' (°)	28	28	30	33	41.8 ^a	30.16 ^b	-
Elastic modulus, E' (kN/m ²)	5,217	9,900	38,500	72,600	7.1×10^6	1,000	27×10^9
Poisson’s ratio, ν	0.35	0.35	0.35	0.35	0.35	0.35	0.15
External pipe diameter, D_e (m)	-	-	-	-	-	-	1.43

Note: a. From generalized tangential interpretation of earlier direct shear tests.

b. Interface friction angle, $\phi_i = \tan^{-1}(c'_i / \sigma'_i)$

Table 6. Analysis of finite element modeling results

Drive	A	B	C
Geology	Shale	Metagraywacke – siltstone	Graywacke – phyllite
Prescribed displacement (mm)	100	100	100
Vertical stress acting on pipe crown, σ_{EV} (kN/m ²)	12.5	-19.9 ^a	-38.5 ^a
Arching ratio, AR (%) ^b	8.00	3.75	2.90
Measured jacking forces, JF_{meas} (kN/m)	37.2	20.7	23.6
Interface shear stress, τ_1 (kN/m ²)	65	104	159
Jacking forces calculated from interface shear stresses, $JF_{\tau,cal}$ (kN/m) ^c	29.0	17.5	20.8
Variation, $\frac{JF_{\tau,cal} - JF_{meas}}{JF_{meas}} \times 100$ (%)	-22.2	-15.4	-12.2
Reaction force, ΣF_y (kN)	7,223	8,759	12,850
Jacking forces calculated from reaction force, $JF_{Fy,cal}$ (kN/m) ^d	28.9	16.4	18.7
Variation, $\frac{JF_{Fy,cal} - JF_{meas}}{JF_{meas}} \times 100$ (%)	-22.3	-20.8	-21.1
Difference in variations, $\frac{JF_{\tau,cal} - JF_{Fy,cal}}{JF_{meas}} \times 100$ (%)	0.1	5.4	8.9

Note: a. Negative value of σ_{EV} was readjusted to zero.

b. From Eq. 8.

c. From Eq. 9.

d. From Eq. 10

Graphical abstract

Highlights

- Jacking forces were backanalyzed using tangential parameters, $c'_{t,p}$ and $\phi'_{t,p}$.
- The reliability of $c'_{t,p}$ and $\phi'_{t,p}$ was assessed using finite element modeling.
- Backanalyzed μ_{avg} was used to model the strength of pipe-rock interface elements.
- Arching ratios were developed from σ_{EV} for post-analysis of modeling results.
- The simulated results were in reasonable agreement with measured jacking forces.

Declaration of interests

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Figure 1

Figure 2

Figure 3

Figure 4

Figure 5

Figure 6

Figure 7

Figure 8

Figure 9

Figure 10

Figure 11