SAND-PIPE INTERACTION AT FAULT CROSSINGS: EXPERIMENTAL AND NUMERICAL INVESTIGATION OF UPLIFT RELATIVE MOVEMENTS

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ABSTRACT

Physical models have been used since the 1980’s to measure the reaction from backfill soil to relative pipe movements. Outcomes of these studies form the basis of current guidelines for the stress analysis of pipelines crossing active faults via beam-spring models. Physical modelling of sand-pipe interaction is revisited in this paper, which presents a novel apparatus designed to measure with high accuracy the reaction developing on a rigid pipe while applying displacements along various directions in a vertical plane perpendicular to the pipe axis. The proposed setup allows accurate control of uniformity and density of the sand backfill surrounding the pipe, and visualisation of the developing failure surface via Particle Image Velocimetry, therefore rendering the measurements ideal for benchmarking numerical simulations. This concept is presented for the case of pipe uplift in dry sand, followed by a discussion on the capacity of finite element models in capturing the developing failure mechanism and in predicting soil reaction to pipe movements.

Keywords: soil-pipe interaction; uplift; finite element method; physical model

1. INTRODUCTION

Stress analysis of pipelines crossing seismically active areas, where permanent ground movements due to fault rupture may result in detrimental pipe strains, requires estimating the magnitude of soil reaction to pipe displacements relatively to the surrounding soil. A number of research groups (e.g. Trautmann et al. 1985; White et al. 2001, Cheuk et al. 2008) have attended to this problem since the 1980’s using small and large scale laboratory setups, analytical considerations and numerical simulations. Results of these studies are endorsed by standards and design guidelines (e.g. ASCE-ALA 2005; NEN3650) and form the basis of the current state-of-practice for the analysis of pipelines crossing active faults.

The present study was motivated by considerable scatter observed in experimental measurements and the disagreements identified in published works, as well as some questionable assumptions regarding soil behaviour embraced in numerical studies. To shed some light into these issues, we revisit here the problem of quantifying the reaction from dry sand backfill to pipe uplift movements, a case of interest for normal and reverse fault crossings. To obtain the reaction force as function of the imposed displacement while tracking the developing failure surface we use a novel small-scale physical modelling setup, developed in the University of Newcastle, Australia to simulate arbitrary soil-pipe relative displacement histories under controlled conditions.

To limit the extent of the presentation, we focus on dense sand backfills, which is the worst-case scenario for pipes crossing faults. We present first the physical modelling rig, the mechanical properties of the sand used in the experiments as gained from element tests, the elaborate sand deposition system developed to achieve excellent sand bed uniformity and test repeatability, the corrections applied to account for boundary effects, and the method used to track sand displacements during testing. Results
for a range of pipe embedment depths of practical interest are presented, while we attempt in parallel to perform “blind” numerical predictions of the same test results using an advanced constitutive model carefully calibrated on element tests pertinent to the stress levels of the problem. Comparison of experimental measurements against numerical results and state-of-practice formulas allows gaining some insights on the mechanisms governing the problem, and on the robustness of current stress analysis assumptions.

2. SMALL-SCALE PHYSICAL MODEL

2.1 Outline of the Experimental Setup

Pipe uplift tests were carried out in the narrow chamber depicted in Figure 1, designed to replicate plane strain conditions. The chamber is built from two annealed glass walls and three aluminium plates, two of which form the cross walls and the third forming the base of the chamber. Glass walls allow observing soil deformation to accommodate pipe movement, and documenting both with a digital camera. The aluminium and glass plates were locked into one another through slid-fit grooves and further stiffened using bracings at the bottom of the chamber. The chamber is supported by a steel frame which is mounted on the strong floor using cross legs. A schematic of the chamber and of the pipe prior to pull-out is shown in Figure 1.

The pipe used during the presented experiments was a hollow cylinder made of PVC, of external diameter 75mm and of wall thickness 15mm, which ensured that the pipe will behave as rigid during pull-out. Nylon caps were placed at the pipe ends, which were surfaced with acrylic felt sheets to minimise friction and prevent sand grains from trapping between the pipe and the glass. The pipe assembly had a total weight of 3.4N. The loading system was built around a 1.5tn geared winch actuator, providing winding speeds between 0.01-3.3 mm/min. The pipe was connected to the electric winch with a steel cable of diameter 3mm. The reaction force was measured using 0.25kN and 0.5kN S-shape load cells (depending on the expected force and to maximise accuracy) connected midway between the vertical section of the pulling cable and the pipe. Displacement of the pipe was measured using string potentiometers with accuracy of 0.1mm. Measured data are collected by a DT80 data logger and saved to a desktop computer which remotely controls both actuation and acquisition systems. A detailed description of the setup is provided in Ansari et al. (2018)

2.2 Backfill Sand Properties

Washed silica sand from Stockton Beach (NSW, Australia), classified as SP according to USCS, was
used in the tests. Minimum and maximum void ratios of this sand are $e_{\text{min}} = 0.51$ and $e_{\text{max}} = 0.79$, respectively, and the specific gravity is $G_s = 2.65$. Ajalloeian et al. (1996) measured the critical state friction angle through triaxial tests to be $\phi_{cs} = 32^\circ$. Direct shear tests in dense ($D_r = 92\%$) Stockton Beach Sand (STK) samples at normal stress levels between $\sigma_n = 1.25\text{kPa}$ and 100kPa resulted in the peak friction and dilation angles listed in Table 1 (Ansari et al. 2018). This relative density corresponds to dry density of $\gamma_{\text{dry}} = 16.9\pm0.15\text{kN/m}^3$. Plane strain friction ($\phi_{ps}$) and dilation ($\psi_{ps}$) angles interpreted from the values of Table 1 according to Davis (1968) can be fitted with the following expressions:

$$\phi_{ps} = -6.06 \cdot \ln(\sigma_n) + 62.69 \quad \text{[deg, $\sigma_n$ in kPa]}$$  \hspace{1cm} (1)

$$\psi_{ps} = -1.575 \cdot \ln(\sigma_n) + 16.13 \quad \text{[deg, $\sigma_n$ in kPa]}$$  \hspace{1cm} (2)

Table 1. Direct shear test results on dense ($D_r = 92\%$) STK sand.

<table>
<thead>
<tr>
<th>Normal stress, $\sigma_n$ [kPa]</th>
<th>Peak secant friction angle, $\phi_{ds}$ [deg]</th>
<th>Peak dilation angle, $\psi$ [deg]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.25</td>
<td>49.82</td>
<td>18.30</td>
</tr>
<tr>
<td>2.00</td>
<td>46.03</td>
<td>13.89</td>
</tr>
<tr>
<td>3.50</td>
<td>44.00</td>
<td>11.98</td>
</tr>
<tr>
<td>5.00</td>
<td>42.61</td>
<td>11.14</td>
</tr>
<tr>
<td>7.50</td>
<td>42.61</td>
<td>14.81</td>
</tr>
<tr>
<td>10.00</td>
<td>40.50</td>
<td>13.72</td>
</tr>
<tr>
<td>30.00</td>
<td>36.50</td>
<td>11.63</td>
</tr>
<tr>
<td>50.00</td>
<td>34.50</td>
<td>9.19</td>
</tr>
<tr>
<td>100.00</td>
<td>33.34</td>
<td>8.98</td>
</tr>
</tbody>
</table>

The compressibility parameters of STK sand were obtained from incremental loading oedometer tests. The measured coefficient of volume compressibility $m_v$ values of dense sand samples with initial void ratio $e_0 = 0.54$ can be fitted with the following expression:

$$\frac{1}{m_v} = E \left[ (1-\nu)/(1+\nu)(1-2\nu) \right] = 4700 \cdot (\sigma_n/3.85)^{0.68} \quad \text{[}$\sigma_n$, $m_v$ in kPa$]$}$$  \hspace{1cm} (3)

where $E$ is the Young’s modulus and $\nu$ is the Poisson’s ratio of sand.

2.3. Sand Bed Deposition and Homogeneity

Dense STK sand was deposited into the chamber using a custom semi-mechanised sand deposition system. The sand bed was deposited in 30mm lifts, after being diffused through a bespoke rainer. The rainer comprises 3 rounds of sieves, cut to the inner section of the chamber as shown in Figure 2.

![Items in the image:
1. Stepper motor arm
2. Motor-rainer connector
3. Rainer
4. Perforated sheet (sieve #1)
5. Testing chamber
6. Supporting frame](image)
Lifting of the rainer during deposition was facilitated by a stepper motor connected to a linear actuator. Sample homogeneity was assessed via miniature cone penetration tests performed at various locations across the surface of the sand bed, using a custom 14mm diameter cone. Minor variations of cone tip resistance profiles at the different test locations confirmed the excellent uniformity of deposited sand beds inside the chamber. In addition, continuous resistance profiles with depth corroborate that staged deposition did not result in discontinuous properties at the interface between layers. Detailed results of this quality assessment system are presented in Ansari et al. (2018).

2.4 Corrections for Boundary Effects

The length-to-diameter ratio of the test pipe that can be accommodated in the presented small-scale rig is $L/D = 1.0$ (or less). The facilities used in most previous studies were of larger scale, which allowed testing of longer pipes with $L/D \geq 5$ (e.g. Trautmann et al. 1985; Rowe and Davis 1982). Larger chambers come at increased commissioning costs and longer experiment preparation times; but also less control on the uniformity and target density of the sand bed, and decreased measurement accuracy. Tests performed in the presented setup exhibit excellent repeatability, however measurements need to be corrected to account for the different components of friction developing between different modules of the testing rig.

Resisting forces resulting from friction developing at the pipe-glass interface, between the cable and the pulley, and between sand and the embedded segment of the cable, had to be measured and deducted from the uplift resistance. Their values were measured experimentally by performing ‘friction tests’ in which sand was vacuumed from the chamber after pipe uplift was completed, and consequently the pipe was placed back to its original position and the test was repeated without any soil resistance. Friction between the buried segment of the cable and sand was measure to be negligible during pilot tests without the pipe.

Another mechanism associated with boundary effects in the narrow chamber is sand arching, due to friction between the sand bed and glass walls. This phenomenon was investigated extensively via a numerical model of sand deposition calibrated on close range photogrammetry and Particle Image Velocimetry (PIV) measurements. From that, we concluded that the stress field in the chamber can be recovered by considered geostatic conditions, but with reduced sand dry density by 23%. As such, interpretation of test results consider an effective dry sand unit weight $\gamma_{eff} = 0.77\gamma_{dry}$. The interested reader is referred to Ansari et al. (2018) for more details.

3. TRACKING SAND DISPLACEMENTS VIA PARTICLE IMAGE VELOCIMETRY

Precise tracking of the evolving shape of the failure surface during pipe uplift tests was facilitated by PIV analysis of digital images.

![Figure 3. Schematic of sand particle tracking during lateral dragging of a pipe in sand.](image-url)
Figure 3 above presents a schematic of the method employed to quantify sand particle deformations in a plane of interest, by processing a number of sequentially captured images. Images at the deformed state are compared one at a time with a reference image corresponding to the initial test conditions. Each digital image is represented by an integer matrix whose elements are defined via a function $f(x,y)$ with $f$ being the intensity (or grey level) at an image segment defined via coordinates $(x,y)$ in the Cartesian coordinate system of Figure 3. To enable particle tracking, a Region of Interest (RoI) is defined in the reference image and then gridded into a number of elements, known as ‘subsets’ or ‘test patches’. A cross-correlation criterion facilitates tracking the location of each one of the initial subsets in subsequent deformed-state images. Matching between two subsets is based on scouring for maximum correlation coefficients.

Images were captured during tests using a DSLR camera fitted with a 28-mm prime lens, at a frequency of 0.2Hz (5 sec intervals associated with 0.25mm of applied displacement at the winch). The sequence of images was then analysed using the geoPIV_RG algorithm (Stanier et al. 2015). Sand particle displacement vectors can be manipulated to provide displacement and engineering shear strain contours at the area of interest, from which the geometry of the failure surface can be inferred. Of particular interest are the contours corresponding to the pipe displacement where the peak resisting force develops, which are presented later in Section 5, for different initial pipe embedment depths.

4. SOIL REACTION TO PIPE UPLIFT

Figure 4a presents the evolution of normalised reaction forces to pipe uplift movements for tests starting from different initial embedment depths of practical interest. All tests were performed in dense sand beds, with relative density $D_r = 92\%\pm7\%$. The reaction force, $F_{vu}$ is normalised against the total overburden weight of sand acting on the pipe springline $\gamma_{eff}HDL$, where $H$ is the depth to the springline measured from the sand bed surface, $D = 75\text{mm}$ is the pipe diameter and $L = 75\text{mm}$ is the out-of-plane pipe length. This normalisation allows obtaining a measure of the shear resistance to pipe movement. In addition, pipe displacement is normalised against its diameter. Notice that softening characterises the response in all tests post-peak. The considerable oscillations observed post-peak suggest that as the rate of pulling is kept constant, a drop in the measured reaction is recorded every time the tension in the cable overcomes the static friction of the system, reducing the friction coefficient to its kinetic value (stick-slip phenomenon). These oscillations are therefore associated with the developing failure mechanism, and not with instrument noise.

Figure 4b compares the peak uplift resistance gained from the reaction force-displacement curves against the results of Trautmann and O’Rourke for dense sand, which are commonly used in practice as
input when preparing stress analysis models of pipes crossing seismic faults. Notice that the latter consistently underestimate the peak resistance. One would argue that the sand beds prepared in the study at hand were denser compared to the experiments of Trautmann and O’Rourke. However Trautmann and O’Rourke do not explicitly report the relative density achieved in their tests, due to inconsistencies between the densities achieved in their chamber and those measured according to ASTM D2049 (Trautmann et al. 1988). Based on the dry densities reported in Trautmann et al. (1985), it can be inferred that the relative density to which the thick black line in Figure 4b corresponds to is $D_r = 75$ to $80\%$. The presented comparison suggests that the method endorsed by ASCE-ALA (2005) does not provide a conservative upper bound of forces applied on pipes during uplift in very dense sands, and exemplify the importance of accurately controlling sand density in benchmark experiments.

5. NUMERICAL “BLIND PREDICTIONS”

A number of numerical studies (e.g. Jung et al. 2013, Yimsiri et al. 2004, Kouretzis et al. 2014) have successfully reproduced the experimental force-displacement curves obtained by Trautmann and O’Rourke (1983). However, this required some form of calibration of soil constitutive model parameters to the specific experimental data. Especially when an elastoplastic constitutive model is used to describe soil behaviour, an equivalent deformation modulus compatible with the soil strength mobilised at the peak resistance needs to be considered.

Here we attempt to replicate the experimental results with finite element simulations based on a variation of the Hardening Mohr Coulomb (HMC) model developed by Doherty and Muir Wood (2013) and implemented in the computer code OptumG2 (Krabbenhoft 2017). All model parameters are calibrated on direct shear and incremental loading element tests, performed at low stress levels pertinent to the problem (hence the term “blind predictions”). The advantage of this model is that the secant soil stiffness, $E_{50}$ is defined separately to the unloading/reloading stiffness $E_u$, thereby offering more flexibility in terms of defining the sand’s stiffness characteristics. The ratio $E_u/E_{50}$ is usually $E_u/E_{50} \approx 2$ to 5 or higher. Here we take $E_u$ to be equal to the stiffness interpreted from incremental loading tests, while given the pressure dependence of sand mechanical properties, we assume that the stiffness parameters vary with stress levels as:

$$E_{50} = E_{50,ref} \left( \frac{\sigma_3}{p_{ref}} \right)^{m} \quad (4)$$

$$E_{ur} = E_{ur,ref} \left( \frac{\sigma_3}{p_{ref}} \right)^{m} \quad (5)$$

where $\sigma_3$ is the minor principal stress, $p_{ref}$ is the reference pressure corresponding here to the mean stress at the pipe springline (different for each simulation), and $m = 0.68$ is a fitting parameter calibrated on incremental loading oedometer tests (Eq. 3). $E_{ur,ref}$ values were gained from Eq. (3), while we assumed the secant stiffness to be $E_{50,ref} = 0.1E_{ur,ref}$. Peak friction and dilation angles were calculated from Eqs. (1) and (2) respectively, for the vertical stress $\sigma_n$ acting on the pipe springline. We use Taylor’s stress-dilatancy law to describe volume change during shearing, in which dilation is not constant but increases as the ultimate state is approached, where it is quantified via $\psi_{ps}$ in Eq. (2). Analyses were performed while considering the effective unit weight of STK sand from Section 2.4 equal to $\gamma_{eff} = 0.77 \cdot 17.2 = 13.2 \text{kN/m}^3$. The earth pressure coefficient at rest was calculated with Jaky’s formula as $K_0 = 1 - \sin \phi_{cs} = 0.47$ using the critical state friction angle reported earlier, while the Poisson’s ratio of STK sand was assumed to be $v = 0.25$ and the effective cohesion was taken equal to $c' = 0 \text{kPa}$. A schematic of the analysis model is presented in Figure 5. Each analysis comprised 10,000 6-node Gauss elements, while adaptive meshing was used to capture strain localisations in dense sand emerging due to the non-associative flow rule considered. The boundaries of the mesh correspond to the borders of the chamber, and the pipe is simulated with rigid elements. The peak friction stress that can develop at the interface between the rigid pipe elements and sand is taken equal to 50\% of the sand shear strength...
at the same normal stress, hence the contact conditions are representative of an interface between “smooth” and “rough”. Uplift of the pipe is simulated by applying a prescribed vertical displacement at its centre, which in each simulation is taken equal to the displacement where the peak resistance was observed during the experiments; the employed analysis method cannot simulate softening observed in the experiments post-peak.

Numerical results are compared against experiments in terms of normalised resistance-displacement curves in Figure 6 and in terms of the shape of the failure surface at the displacement where the peak resistance is reached in Figures 7-10. Figure 6 also depicts the range of displacement $\delta_{vu}$ where the peak resistance would develop according to ASCE-ALA (2005) guidelines, and compares the peak resistance gained from the numerical analyses against experimental data and the formula proposed in ASCE-ALA (2005). Note that the resistance is again normalised against the weight of the soil column acting on the pipe springline, following the presentation introduced in Figure 4.

Observe that the comparison between numerical and experimental results is rather poor. To start with, numerical simulations predict a much stiffer initial response compared to experiments, despite using as input a rather low secant stiffness value. This could be attributed to inevitable slack in the test setup, the effect of which on recorded pipe displacements is stressed out in other similar experimental studies (e.g. Trautmann et al. 1985). Note that fitting of the experimental results is possible by using a reduced sand deformation modulus compared to the one measured in incremental loading tests, however the merit of such an adjustment in the numerical model is questionable, as it does not address the mechanisms that lead to this divergence.

Apart from the initial stiffness, considerable differences are also noticed in the peak normalised soil resistance. Good agreement is achieved only for shallow embedment experiments ($H/D = 1.5$), while experimental and numerical results diverge as the initial embedment increases, with numerical simulations consistently under-predicting the measured peak soil resistance. The maximum difference reaches 50% for $H/D = 7$. Interestingly, numerical results appear to match predictions of ASCE-ALA (2005), despite the fact that the constitutive model parameters are representative of a denser sand. To investigate this discrepancy, we compare contours of shear strains interpreted from images captured during the experiments using PIV, against the corresponding numerical results. Observe in Figures 7-10 that during pipe uplift in the chamber a wedge-type failure mechanism is formed, with the inclination of the shear planes being about 20° on average. In experiments with low initial embedment, where the failure wedge outcrops at the sand bed surface, the inclination of the shear planes increases towards the surface as a result of the increase in the dilation potential at very low stress levels. Infilling of the gap under the pipe due to the flow-type mechanism is observed for pipe burials larger than $4D$, for which the pipe displacement at peak resistance is sufficient to activate this secondary mechanism.
Shear strain contours drawn from numerical analysis results suggest a similar wedge-type failure mechanism comprising two main shear planes, whose shape is very similar to the one observed in the experiments. Localisation, a direct consequence of non-associativity, leads to the development of multiple (and non-symmetric) secondary shear bands (Figures 7-10). Their geometry and thickness is captured in the analyses by employing element adaptivity, which leads to mesh refinement at zones of high shear strain gradients. However, finite element analysis is a continuum analysis method, hence fails to capture the secondary flow-type mechanism observed at deeper pipe embedment depths, for larger pipe displacements. We believe that this is reason for failing to predict the ultimate pipe resistance in our “blind” predictions, and the differences observed correspond to the contribution of this mechanism.
to the resistance developing on the pipe. This decreases as the pipe embedment decreases, and becomes trivial for shallow embedment depths where the peak resistance develops for pipe displacements of the order of 3% of its diameter.

Figure 8. Qualitative comparison of (a) experimental and (b) numerical shear strain contours and finite element mesh at the displacement where the peak resistance was recorded in the experiments. Embedment $H/D = 3$.

Figure 9. Qualitative comparison of (a) experimental and (b) numerical shear strain contours and finite element mesh at the displacement where the peak resistance was recorded in the experiments. Embedment $H/D = 4$.

Figure 10. Qualitative comparison of (a) experimental and (b) numerical shear strain contours and finite element mesh at the displacement where the peak resistance was recorded in the experiments. Embedment $H/D = 7$.  

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6. CONCLUSIONS

Estimating the maximum backfill resistance to pipe uplift and the relative displacement at which this resistance develops is essential for the stress analysis of buried pipes crossing potentially active normal and reverse faults. We have shown that formulas endorsed by current guidelines may provide unconservative estimates of the peak resistance, particularly for pipes embedded deeper than $H/D = 4$, which is not uncommon in practice. Analysts should also be aware that the displacement at peak resistance interpreted from physical modelling experiments is potentially affected by the inevitable slack in testing equipment, the role of which may be prominent given that the displacement required to mobilise the failure mechanism is rather small. We have also shown that even state-of-the-art finite element analysis tools, incorporating advanced sand constitutive models and adaptive meshing techniques, fail to properly capture the mechanics of this problem. There appears to be a secondary flow-type mechanism that contributes about 50% of the resistance at deep embedment cases, which cannot be captured by continuum analysis methods. Finally, we recommend against fitting finite element analysis parameters on experimental resistance-displacement curves, as the applicability of the analysis method depends on the initial geometry of the problem. Indeed, for shallow embedment depths “blind” predictions successfully captured the peak resistance to pipe uplift.

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7. REFERENCES


