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Interpretation of uplift pipeline-soil interaction behaviour using acoustic emission measurements

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ABSTRACT

The objective of this study was to develop quantitative acoustic emission (AE) interpretation for uplift pipeline-soil interaction behaviour, enabling early warning of serviceability and ultimate limit state failures in the field. A series of large-scale uplift experiments was performed on a steel pipe in sand with different burial depths (i.e., stress levels), and varying rates of deformation were imposed. A suite of AE parameters was compared with the pipe force and displacement behaviour. Image-based deformation measurements were used to monitor the soil displacement field. AE generation was proportional to the imposed stress level and pipe displacement rate and related to the evolution of the pipe/soil failure mechanism. Relationships have been quantified between AE generation and stress level (R^2 values of 0.99), and between AE generation rate and pipe velocity (R^2 values ranging from 0.95 to 0.98), enabling interpretation of accelerating deformation behaviour that accompanies progressive ground failure processes. An example interpretation framework demonstrates how AE parameters can be used to identify the mobilisation of peak uplift resistance and quantify accelerating deformation behaviour during post-peak softening.

KEYWORDS

Acoustic emission; Buried pipelines; Deformation; Ground movements; Monitoring; Soil-structure interaction.

INTRODUCTION

Proportions of the energy dissipated when materials deform are converted to heat and sound. The high-frequency (>10 kHz) component of this sound energy is called acoustic emission (AE). AE is generated by soil-pipe interaction through a suite of mechanisms: inter-particle friction; soil-pipe interface friction; particle contact network rearrangement; particle asperity degradation; pipe surface degradation; and particle crushing (Smith & Dixon 2019; Smith *et al.* 2019; Smith *et al.* 2020).

The purpose of AE research in geotechnical engineering over the past 60 years has been to quantify links between measured AE and strength and deformation behaviour.

Interpretation of AE measurements from geotechnical systems in the 1970s, 1980s and 1990s (e.g., Koerner *et al.* 1976; Koerner *et al.* 1978; Koerner *et al.* 1981; Koerner *et al.* 1984; Tanimoto and Nakamura 1981; Mitchell and Romeril 1984; Tanimoto and Tanaka 1986; Garga and Chichibu 1990; Shiotani and Ohtsu 1999) was limited to qualitative frameworks, for example: (a) no AE indicates stability; (b) moderate levels of AE indicates marginal stability; and (c) high levels of AE indicates that the geotechnical system is unstable.

More recently, quantitative interpretation frameworks are being developed for slope stability (e.g., Smith *et al.* 2014; Smith *et al.* 2017a; Deng *et al.* 2021a,b), pile penetration (e.g., Mao *et al.* 2018), triaxial testing (Smith and Dixon 2019), soil-steel interface shearing (Smith *et al.* 2020), and particle crushing (Lin *et al.* 2019). These frameworks monitor increasing AE generation rates (i.e. increasing energy dissipation within the geotechnical system per unit time) to detect, quantify, and interpret accelerating deformation behaviour that accompanies progressive ground failure processes for use in early warning by decision makers.

Smith *et al.* (2019) performed differential ground motion (normal faulting) experiments on full-scale buried pipes and demonstrated that the measured AE was proportional to both the magnitude and rate of differential ground movement. Extending this study, the objective of the investigation reported in this paper was to develop quantitative AE interpretation for uplift pipeline-soil interaction behaviour. For the first time, AE has been measured during uplift of buried pipes in sand, empirical relationships between AE activity and mechanical behaviour (load, displacement, and displacement rates) have been quantified, and a conceptual framework for interpretation of buried pipe uplift behaviour using AE measurements has been developed.

This work formed part of the *Listening to Infrastructure* project, which is developing AE technologies that can be retrofitted to geotechnical infrastructure assets. The study findings have significant implications for the development of a continuous, real-time AE monitoring system that can be distributed at discrete locations along buried pipes to sense pipe–soil interaction resulting from a range of deformation mechanisms and provide early warning that will enable targeted and timely interventions.

Nascent technologies, such as fibre optics, are beginning to provide valuable pipe asset health monitoring information; however, retrofitting hundreds of thousands of kilometres of assets with these sensors would be prohibitively intrusive and expensive. The advantage of the AE approach is that sensors could be installed at discrete locations to provide high spatial and temporal resolution information for use in early warning of adverse deformation behaviour. AE propagates tens to hundreds of metres in pipelines from the generation source, allowing a few sensors at appropriate spacings to monitor extensive lengths of pipeline assets (Smith *et al.* 2017b).

Pipeline networks cover vast geographical areas to transport water, oil, and gas, and hence are critical lifelines upon which society heavily relies. A significant proportion of

these assets are buried in soil for protection and support. Pipeline sections can experience upwards displacement through the surrounding soil due to: differential ground movement (e.g., fault-rupture or landsliding intersecting a buried pipeline); flotation of subsea pipelines (or pipelines buried under bodies of water) that have a very low combined pipe/fluid weight; the development of excess water pressures around the pipe; and upheaval buckling whereby axial thermal expansion forces, restrained by the pipe-soil interface friction and end connections, are relieved by pipe buckling (Cheuk *et al.* 2008; White *et al.* 2008; Byrne *et al.* 2013; Williams *et al.* 2013; Roy *et al.* 2018; Wong *et al.* 2021). Pipe uplift can result in significant strains and tensile or buckling failure, which can have catastrophic economic, environmental, and societal consequences, and the service of entire networks can be terminated (Karamitros *et al.* 2007; Vazouras *et al.* 2015; Robert *et al.* 2016).

This paper describes a series of large-scale uplift experiments on a steel pipe in sand with different burial depths (i.e., stress levels), and with varying rates of deformation imposed to investigate the AE response to accelerating deformation behaviour that accompanies progressive failure processes. AE measurements are compared with force and displacement behaviour, and a conceptual framework is presented for the detection of the mobilisation of peak uplift resistance and interpretation of accelerating pipe/soil displacements.

METHODOLOGY

Apparatus and testing considerations

Figure 1(a) is a cross-section diagram of a typical buried pipe, showing the pipe outside diameter, D , the burial depth to the pipe springline, H , and the uplift resistance, P . Figures 1(b) and 1(c) show the uplift mechanisms used in widely adopted uplift resistance

prediction models, which comprise a sliding block bounded by a pair of shear surfaces. In the vertical slip model (Figure 1(b)), uplift resistance arises from the weight of the lifted block and the shear stress on the shear surfaces. In the inclined slip plane model, where the inclination, θ , of the shear surfaces is related to the soil dilation angle, the uplift resistance is equal to the lifted soil weight plus the resultant vertical force on the shear surfaces, which arises from both the shear and the normal stresses.

Observations from plane-strain experiments with short pipe sections moving upwards through dense soil show that the uplift failure mechanism comprises four stages (Cheuk *et al.* 2008): (1) mobilisation of peak resistance; (2) gap formation beneath the pipe invert and infilling; (3) post-peak shear band formation; and (4) flow-around soil displacement. A gap with sloping sides forms beneath the pipe during mobilisation of peak uplift resistance, which subsequently fail and infill the gap beneath the pipe. Shear strain concentrates into a pair of dilating shear bands, which initiate at the springlines and curve outwards and widen as they propagate to the surface. The flow-around mechanism is accompanied by a reduction in the uplift resistance due to post-peak shear band formation (Bransby *et al.* 2007; Cheuk *et al.* 2008).

The geometry of the test chamber employed in this study was selected to ensure full-scale pipe diameters could be tested under a range of burial depths without the dilating shear bands interacting with the lateral chamber boundaries. The test chamber is shown in Figure 2 (photograph) and Figure 3 (cross-section diagrams), which had internal dimensions of 1.5m (height) x 1.5m (width) x 0.985m (depth). The base and three walls were constructed from structural steel, while one wall was a 31.5mm-thick laminated and toughened glass plate. The glass plate enabled image-based deformation monitoring throughout the experiments.

The three steel walls were lined with wooden boards and covered with friction treatment to reduce mobilised shear stress at the soil/wall interface. The friction treatment comprised a base layer of polyethylene sheet, which was fixed in position at the top of the walls. This was coated in a thin film of silicone-based bearing grease to act as a lubricant, and then a second layer of polyethylene sheet was placed on top. The second sheet slides against the base sheet during the test, mobilising a peak angle of friction of less than 5° (Tognon *et al.* 1999).

A steel pipe was selected for this study for two key reasons: (1) steel pipelines are the most common choice for transporting water, oil and gas across large geographic areas and hence are likely to intersect a range of geohazards; and (2) AE propagates long distances in steel with minimal attenuation (<0.2 dB/m) (Smith *et al.* 2017b). The pipe was 0.985m long, had an outside diameter of 168.3mm and a wall thickness of 5mm. Plastic end caps (each 50mm long) covered with PTFE tape reduced friction on the pipe ends. Roughness average, R_a (μm), measurements taken from 10 positions on the pipe outside wall resulted in a mean of $6.2\mu\text{m}$ with a range of 4.4 to $8.6\mu\text{m}$, which confirm the pipe was relatively smooth.

In model pile penetration studies (e.g. Mao *et al.* 2020), sand has been observed to gradually fill the pile-wall interface gap and be crushed into fines. In this study, contact was established between the pipe ends, glass plate and rear box wall, to reduce the potential for sand to fill the pipe-wall interfaces. Experiments were performed without backfill to measure the uplift force associated with the interface friction at the pipe ends, and this was subtracted in the subsequent tests to quantify the presented net uplift force measurements. Despite contact with the box walls at the pipe ends, sand grains periodically entered the perimeter of the interface, which could have generated AE. However, the plastic caps at the pipe ends would have significantly attenuated this AE activity (e.g. AE attenuation in polymeric materials is orders of magnitude greater than in

steel due to viscoelastic behaviour), hence any extraneous AE generated at the pipe ends did not significantly contaminate the AE measurements.

Vertical steel bars (12mm diameter) were fixed to the pipe at two locations (Figure 3) and connected to a crossbeam to form a frame, which was in turn connected to the loading actuator. During the tests, the loading actuator moves vertically upwards, which lifts the buried pipe via the frame. Net uplift force and displacement were measured throughout each test. The initial pipe position was 0.2m above the base of the box and three burial depths were selected to fall within the typical range for gas pipes (Groves and Wijewickreme 2013).

Backfill

Quartz Leighton Buzzard sand (LBS) was selected for backfill in this investigation as its mechanical behaviour has been studied extensively in the literature and its properties, including AE behaviour, quantified in previous studies (e.g., Smith and Dixon 2019; Smith *et al.* 2020). Figure 4 shows the particle size distribution of the LBS used as backfill and Table 1 summarises its properties. In each test, backfill was compacted in 0.1m lifts using a manual tamping plate to a target relative density, D_r , of 85%. The combined mass of soil placed in each layer and the overall compacted backfill volume was used to calculate the average density. Following compaction, the backfill dry density was 1.73 Mg/m³.

Testing programme

The burial depth and embedment ratio in each of the three tests are summarised in Table 2. A series of widely used uplift resistance prediction models were used to compare their outputs with the results from the experiments reported in this study (Table 2). Peak uplift force, P , and pipe displacement at peak, δ_p , predicted using the American Lifelines Alliance (ALA) (2005) method were the closest to the experimental results. For example,

for the three burial depths, the ALA (2005) prediction error in peak uplift force was less than $\pm 10\%$.

The influence of deformation rate (i.e., pipe/soil movement velocity) was a focus of this investigation because if AE can be used to quantify increasing/decreasing rates of pipe/soil movement, it could be used to provide an early warning of accelerating deformation behaviour that typically accompanies ground failure processes (e.g., post-peak softening). To achieve this, a pre-programmed displacement-controlled program was used to move the loading actuator (Figure 5), which consisted of 12 constant-velocity movement stages (Table 3), each separated by a 1-minute period of no movement (i.e., stationary pipe). The stationary pipe periods were included to allow mechanisms such as infilling beneath the pipe invert and flow-around time to achieve equilibrium before the subsequent pipe movement stage. The velocity was constant at 1 mm/min during the first four test stages to ensure peak uplift resistance was mobilised before increasing the rate of movement to simulate accelerating behaviour in the post-peak strength regime. The selected imposed velocities ensured an order of magnitude change in pipe displacement rate (i.e. from 1 mm/min to 10 mm/min).

Acoustic emission instrumentation and data acquisition

Two AE sensors were coupled to the inside wall of the pipe, 0.15m from the pipe end, using magnetic holders, with a layer of silicone gel to improve transmission of the stress waves propagating in the pipe wall. Both sensors were MISTRAS R3 α piezoelectric transducers, which are sensitive over the frequency range of 0-100 kHz and have a resonant frequency of 30 kHz. The sensors convert the mechanical AE to a voltage waveform that can be processed. Each sensor was connected to a separate and different AE measurement system (AE systems (1) and (2), described below).

AE system (1) was a device developed for field monitoring of slopes, named Slope ALARMS (e.g., Smith *et al.* 2014; Smith *et al.* 2017a), and comprised a band-pass filter to attenuate signals outside of the 20-30 kHz range and amplification of 70 dB to improve the signal-to-noise ratio. Focusing AE measurement to within 20-30 kHz removes environmental noise (e.g., construction activity and traffic). This system measures ring-down counts (RDC) per unit time, which are the number of times the AE waveform crosses a programmable threshold level (set to 0.25 V) within a predefined time interval and are a measure of the signal energy (e.g., Figure 6).

AE system (2) was a bespoke setup comprising a pre-amplifier (with a 10-50 kHz filter and 40 dB amplification) a main amplifier (with a 10-100 kHz filter and 21 dB amplification), an analogue-to-digital converter configured to sample at 500 kHz, and a laptop with a LabView program to condition, process and record the AE waveform. The two amplifiers were used to improve the signal-to-noise ratio. The LabView program included a further filter to attenuate signals outside of the 10-50 kHz range and record the full AE waveform within this frequency range. This AE measurement system measured the AE waveform continuously and recorded 500,000 samples per second in separate 1-second-long files, which were subsequently combined in post-processing.

Acoustic emission processing and parameter quantification

AE measured using system (2) during each test was post-processed by segmenting the full AE waveform into consecutive 1-second samples with no overlap in time. After detrending, each sample was denoised by discarding any measurements below a threshold of ± 0.1 V (i.e., low amplitude environmental and electronic noise). This produced a set of N real signals, $s_k(t)$, for each test, with $k = 1, \dots, N$, and $N = [4623, 4776, 4713]$ for the three laboratory tests indicated in Table 2, respectively.

The so-called analytic signal (Huang *et al.* 1998) was computed for each sample, $s_k(t)$:

$$x_k(t) = s_k(t) + i \hat{s}_k(t) \quad (1)$$

In Equation 1, the real part, $s_k(t)$, is the original k -th denoised sample, i is the imaginary unit, and the imaginary part, $\hat{s}_k(t)$, is the Hilbert transform of $s_k(t)$, defined as:

$$\hat{s}_k(t) = \frac{1}{\pi} p.v. \int_{-\infty}^{+\infty} \frac{s_k(\tau)}{t-\tau} d\tau \quad (2)$$

with $p.v.$ representing principal value. By considering each analytic signal and its polar coordinate presentations, according to the formula

$$x_k(t) = A_k(t) e^{i\theta_k(t)} \quad (3)$$

three significant features can be extracted for each sample, $s_k(t)$, which were used to compute the AE parameters investigated in this study:

- The instantaneous amplitude (or envelope), $A_k(t) = \sqrt{s_k(t)^2 + \hat{s}_k(t)^2}$;
- The phase function, $\theta_k(t) = \arctan\left(\frac{\hat{s}_k(t)}{s_k(t)}\right)$; and
- The instantaneous frequency, $f_k(t) = \frac{d\theta_k(t)}{dt}$.

In addition to RDC (Figure 6; measured using AE system (1)) and b -value (described below), three extracted numerical parameters were the focus of this study: (1) the variance of the instantaneous frequency, $\sigma_f(t_k)$; (2) the cumulative envelope area, $E_A(t_k)$; and (3) the mean envelope, $\mu_A(t_k)$.

The variance of the instantaneous frequency, $\sigma_f(t_k)$, was computed for each sample, s_k , using

$$\sigma_f(t_k) = \frac{1}{n_k-1} \sum_{i=1}^{n_k} f_{k,i}^2 \quad (4)$$

where n_k is the total number of measurements (500,000) for sample s_k , $f_{k,i}$ is the instantaneous frequency determined at the time instant t_i corresponding to the i -th

measurement in the k -th sample s_k , and t_k is the k -th second since the start of the test (i.e. $t_k = 1, 2, \dots, N$ seconds).

The cumulative envelope area was determined as:

$$E_A(t_k) = \sum_{i=1}^k \int_0^1 A_k(t) dt \quad (5)$$

The mean value for the envelope in each sample, s_k , was computed as:

$$\mu_A(t_k) = \frac{1}{n_k} \sum_{i=1}^{n_k} A_{k,i} \quad (6)$$

where $A_{k,i}$ is the instantaneous envelope at the time instant t_i corresponding to the i -th measurement in the k -th sample, s_k .

An additional AE parameter of interest that can be obtained from the full AE waveform data is the *b-value* (Pollock 1973), which is a convenient way to describe the amplitude distribution in a time-domain window using a single value. The *b-value* was computed using Equations 7 and 8.

$$\log N = c - bM \quad (7)$$

$$M = \log a \quad (8)$$

where a is the amplitude, M is a log-scale measure termed magnitude, N is the number of AE events with magnitude greater than M , c is the point that the $\log(N)$ vs. M relationship intersects the y-axis, and the coefficient b (*b-value*) is the negative slope of the $\log(N)$ vs. M relationship.

When the AE waveform is dominated by low amplitude events the *b-value* is large. As the proportion of higher amplitude events increases, indicating an increase in energy, the *b-value* reduces.

Image acquisition and image-based deformation analysis

Image-based deformation measurements were used to quantitatively study the pipe/soil failure mechanism and displacement field in the deforming soil. The opensource GeoPIV-RG MATLAB module was used to process the acquired test images. The software divides the initial image into a mesh of interrogation patches of pixels characterised by their brightness distribution. The displacement of each interrogation patch is calculated by comparison with subsequent images in the series (White *et al.* 2003; Cheuk *et al.* 2008; Stanier *et al.* 2016).

A consumer-grade digital camera (Nikon D7000) was centred in front of the glass-fronted box, positioned 2.3m away to ensure the entire glass plate was captured inside the camera frame, and fixed to a tripod (Figure 2). The camera lens was focused for this camera-to-object distance and fixed with electrical tape.

Calibration parameters were required to remove image distortion and enable accurate measurements using the consumer-grade digital camera (Wackrow *et al.* 2017). To perform the calibration procedure, a flat coded RAD (Ringed Automatically Detected) target field (Figure 7) was configured on the inner side of the glass and a set of calibration images were acquired at the distance of 2.3m, following a well-established camera calibration approach (Wackrow *et al.* 2008). PhotoModeler software was used to obtain the camera calibration parameters, which model the interior camera parameters but also refraction of the light rays passing through the 31.5mm thick glass plate (Luhmann 2010).

Scale bars were measured using a SmartScope Flash 200 metrology machine to obtain accurate (i.e., within 20 microns) distances of RAD coded targets attached to a Leica Geosystems GKNL4M Sectional Fibreglass Staff. These scale bars were then positioned on the inner side of the glass plate. Images of the box at 2.3m were acquired and processed in PhotoModeler to measure the target linear distances using photogrammetric

image processing. The linear distances established using photogrammetry were compared to the 'truth' metrology-measured distances, which resulted in a maximum deviation of 0.2mm. This demonstrates that the established camera calibration parameters remove the combined residuals introduced by the lens distortion and glass refraction, and hence the adopted methodology can be used for accurate image-based deformation measurements in this study.

Images were captured at 30-second intervals throughout each uplift test. A series of 10 control points fixed to the inside of the glass with known coordinates, measured using a total station from behind the glass to avoid glass refraction residuals, were used in image processing to convert from the image-space (pixels) to the object-space and output deformation measurements in mm units. PhotoModeler was used to apply the camera calibration and remove distortion from all test images. GeoPIV-RG was then used to process the corrected images and compute soil deformation vectors and contours (e.g., Figures 8 and 9).

RESULTS

Pipe and soil behaviour

Figure 8 shows exemplar resultant sand displacement vectors in Test 1 (0.4m burial) after 110mm of pipe displacement (i.e., in post-peak conditions with reducing uplift resistance). The sand displacement field is consistent with findings reported in the literature (e.g., Cheuk *et al.* 2008), whereby the shear zones curve outwards towards the ground surface as the stress level reduces and dilation increases. In addition, this was accompanied by a flow-around mechanism and infilling beneath the pipe invert, which is shown by the downward soil movement around the pipe shoulders.

Figure 9 shows the resultant sand displacement contours for the three tests after 70mm of pipe displacement (i.e., in post-peak conditions with reducing uplift resistance). The flow-around mechanism was comparable in all three tests. The shear zones curve outwards in each test and the horizontal distance between where the two shear zones daylight at the ground surface increased with burial depth. Figure 9 demonstrates that a vertical shear zone model could be appropriate for predicting peak uplift resistance for the shallow burial depth (Test 1), whereas inclined shear zone models should be employed for the deeper burial depths (Test 2 and Test 3). The dashed lines inclined at 7 degrees in Figure 9 represent straight-line shear zones that result in an equivalent ($\pm 10\%$) volume of failed soil wedge.

The displacement contour measurements immediately adjacent to the pipes in Figure 9 are excluded because analysis produced erroneous values (e.g. due to significant distortion, errors and data gaps) in this zone in Tests 2 and 3. The purpose of the image-based deformation measurements was to monitor the uplift pipe-soil failure mechanism so that it could be compared to observations made by other researchers (e.g. Cheuk *et al.* 2008); the measurements in Figure 9 enable this purpose to be satisfied.

Figure 10 shows the load (net uplift force) versus displacement responses for each test. The load oscillations are due to successive infilling below the pipe invert that increased in magnitude with stress level (burial depth). The peak uplift forces were 1.87kN, 6.42kN and 13.02kN in Test 1, Test 2, and Test 3, respectively (Table 2). The dimensionless force is plotted against dimensionless displacement for the three tests in Figure 11a with results from published experimental programmes superimposed for comparison. The results are also compared with both these other studies and peak uplift resistance prediction models in Figure 11b, where the dimensionless peak uplift force is plotted against embedment ratio. These comparisons demonstrate the force-displacement responses and peak uplift resistances measured in this study are consistent with other researchers' findings and

prediction models used in practice, and hence that typical soil/pipe interaction behaviour has been produced.

Interpretation of AE measurements

Figure 12 compares the force-displacement response during the mobilisation of peak uplift resistance in each test with two AE parameters: *b-value* (Figures 12a-c) and the variance of instantaneous frequency, σ_f (Figures 12d-f). Both AE parameters significantly reduce during the mobilisation of peak uplift resistance and then stabilise (i.e., remain relatively constant with further increments of displacement). These general *b-value* and variance of instantaneous frequency trends (i.e. reducing during mobilisation of peak uplift resistance) are superimposed with fluctuations in activity because a suite of mechanisms is evolving concurrently, which are episodic at the micro-scale (e.g. stick-slip in soil assemblages and particle contact network rearrangement), such as: shear zone initiation at the pipe springlines, which propagate upwards; soil-pipe shearing as the pipe moves upwards through the soil; and gap formation beneath the pipe invert and infilling. The *b-value* measurements start at a very high value before pipe displacement initiates because the AE waveform at this time is dominated by low amplitude events. The *b-value* then reduces as the proportion of high amplitude events in the AE waveform increases. The variance of the instantaneous frequency rapidly increases at the onset of pipe displacement before reducing during mobilisation of peak uplift resistance. The *b-value* versus pipe displacement relationship is comparable for each burial depth indicating that the *b-value* measurements are insensitive to stress level, which is consistent with findings from triaxial testing reported by Smith and Dixon (2019). The *b-value* measurements reduced by 59%, 61% and 62% in Test 1, Test 2 and Test 3, respectively, between 0.5 mm and 15 mm of pipe displacement. In contrast, the magnitude of reduction in the variance of instantaneous frequency during the mobilisation of peak uplift increases with burial depth, indicating that this AE parameter is influenced by stress level. The variance of instantaneous frequency

measurements reduced by 40%, 70% and 85% in Test 1, Test 2 and Test 3, respectively, between 0.5 mm and 15 mm of pipe displacement.

Figures 13a-b show the time series measurements of cumulative RDC and cumulative envelope area, E_A , for the three tests. These AE parameters represent cumulative AE energy generation. It should be noted that all AE measurements are presented as equivalent per metre length of pipe values. The time series relationships of cumulative AE energy generation are analogous to the pipe displacement time series in Figure 5. Figures 13c-d show the cumulative RDC and cumulative envelope area measurements plotted against pipe displacement. Figures 13e-f show the average AE generated per mm of pipe displacement in each test stage, which increases in each test stage until a maximum is reached (e.g., stage four in Test 3), before gradually reducing. This demonstrates that maximum AE energy generation occurs when all uplift failure mechanisms are mobilised concurrently (i.e., inverted trapezoidal block movement along dilating shear zones, flow-around, and infilling below the pipe invert), rather than solely at mobilisation of peak uplift resistance.

The results in Figure 13 clearly show that AE energy generation increases proportionally with stress level, as the magnitude of measured AE increased proportionally with burial depth. This is exemplified in Figure 14a where the AE generated per mm during stage 7 (i.e., post-peak) in each test is plotted against embedment ratio. A power function best represents the relationship between AE energy generation and embedment ratio, analogous to the way peak uplift resistance increases with embedment ratio. The AE generated per mm during stage 7 is plotted against the peak uplift force in each test in Figure 14b, which results in a linear function best representing this relationship for the range of data reported.

A particular focus of this investigation was the influence of pipe/soil movement velocity on AE generation. If AE can be used to quantify increasing/decreasing rates of pipe/soil movement, it could be used in early warning of accelerating ground failure processes. The rate of AE energy generation is plotted as time series in Figures 15a-b. RDC rate was measured over 10-second intervals and converted to equivalent RDC per minute values (Figure 15a). The mean AE envelope, μ_A , was computed over each 1-second sampling window and Figure 15b shows 10-second moving average values. The time series relationships of AE energy generation rate (Figures 15a-b) are analogous to the pipe velocity time series in Figure 5, demonstrating that the AE energy generation rate increased proportionally with the rate of pipe movement throughout each test. The AE oscillations in Figures 15a-b are a result of mechanisms such as particle contact network stick-slip and successive infilling beneath the pipe invert; the magnitude of the oscillations increased with stress level (burial depth).

Figure 15c shows the average AE rate (RDC/min) plotted against the applied pipe velocity in each test stage for each test. Figure 15d shows the mean AE envelope plotted against the applied pipe velocity in each test stage for each test. Empirical relationships between the rate of AE energy generation and pipe velocity have been established as power functions, which best describe the relationships (highest R^2 values of between 0.95 and 0.98 from systematic curve fitting) over the range of data reported. The relationship for 0.7m burial depth is 2.8 times greater than the relationship for 0.4m burial depth, and the relationship for 1.0m burial depth is 2.2 times greater than the relationship for 0.7m burial depth.

DISCUSSION

A programme of uplift displacement experiments with a steel pipe buried in sand have been performed to establish quantified relationships for use in the interpretation of pipe/soil interaction behaviour from AE measurements. The mechanical pipe/soil interaction behaviour was consistent with findings reported in the literature, which confirms the adopted experimental approach was able to replicate consistent and realistic uplift failure behaviour. The soil displacement fields captured with image-based deformation monitoring (Figures 8 and 9) revealed the same uplift failure mechanisms that have been identified by other researchers: the formation of a sliding block bounded by a pair of distributed dilating shear zones, infilling beneath the pipe invert, and flow-around displacement (Bransby *et al.* 2007; Cheuk *et al.* 2008). Comparisons of the dimensionless force, dimensionless displacement, and embedment ratio (Figure 11) demonstrated that the results obtained in this study fit within the range of experimental data reported by other researchers and agree well with the commonly used prediction models used in practice (Trautmann *et al.* 1985; Schaminée *et al.* 1990; ALA 2005; Chin *et al.* 2006; DNV 2007; Cheuk *et al.* 2008).

A suite of AE parameters (*b-value*, variance of instantaneous frequency, cumulative RDC, RDC rate, cumulative signal envelope area and mean signal envelope) has been measured from buried pipe uplift experiments for the first time. The results show that AE generation during pipe uplift is proportional to the imposed stress level and pipe displacement rate and related to the evolution of the failure mechanism (i.e., magnitude of pipe displacement). Both the *b-value* and variance of instantaneous frequency measurements can be used to interpret the onset of pipe displacement and mobilisation of peak uplift resistance (Figure 12). The variance of instantaneous frequency was influenced by stress level, whereas the *b-value* measurements were not. This is an important difference to be considered in the selection of which of these AE parameters to use for the

detection of peak uplift mobilisation in a field monitoring system; for example, if the burial depth is variable across the pipe length, *b-value* measurements would be preferable.

All measurements of AE energy generation magnitudes and rates have been presented as equivalent per metre length of pipe values, which permits simple factoring to be applied to scale the results for pipe length. The amount of AE energy generated per increment of pipe displacement evolved over time with the failure process (Figures 13e-f), increasing after the onset of pipe displacement, reaching a maximum when all failure mechanisms were mobilised in post-peak conditions (movement along dilating shear zones, infilling beneath the pipe invert, and flow-around), and then reducing along with uplift force as the mechanism transitions from primarily heave of the soil block to primarily flow-around. The mechanism evolution from primarily heave to primarily flow-around was progressive, which agrees with the findings of Cheuk *et al.* (2008) who also found this change had no distinct transition.

Exemplar relationships have been established to quantify the influence of embedment ratio and peak uplift force on AE generation (Figure 14; R^2 values of 0.99). AE generation increased linearly with peak uplift force and as a power function with embedment ratio. Another original contribution of this study is new quantified relationships between AE energy generation rates and pipe velocity (Figure 15; R^2 values ranging from 0.95 to 0.98), enabling interpretation of accelerating deformation behaviour that typically accompanies progressive ground failure processes (e.g., quantification of increasing relative pipe/soil velocity that occurs in a stress-controlled system using the relationships shown in Figure 15). Waveform envelope and RDC parameters have been used as measures of AE energy generation, and both have resulted in relationships with mechanical behaviour that have strong correlation (R^2 values ranging from 0.95 to 0.99). The selection of which of these parameters to use for the quantification of pipe movement velocity in a field monitoring system should therefore focus on practical aspects of implementation rather than

differences in accuracy; for example, RDC can be implemented in hardware components using a comparator, which means the full AE waveform does not need to be digitally reconstructed, saving processing power and storage requirements (e.g., Smith *et al.* 2017a).

A novel conceptual framework for the interpretation of uplift pipeline-soil interaction behaviour using AE measurements is shown in Figure 16. Warning criteria could comprise: (1) initiation of AE activity showing the onset of relative pipe/soil displacement; (2) reducing trend of *b-value* and/or variance of instantaneous frequency measurements signifying mobilisation of peak uplift resistance; and (3) increasing rates of AE energy generation showing accelerating deformation behaviour. These could trigger engineers to inspect the asset and prioritise maintenance/remediation, or terminate service to prevent catastrophic consequences, facilitating serviceability and ultimate limit state assessments.

Synthesis of this new framework for interpretation of uplift pipeline-soil interaction behaviour with the findings reported in Smith *et al.* (2019) for pipeline-soil interaction-generated AE during differential ground movement, and with additional future research to investigate AE activity resulting from axial pipeline-soil interaction behaviour, would lead to a holistic interpretation strategy for AE monitoring of buried pipelines in the field.

The research reported here has demonstrated the potential for AE monitoring systems to detect and interpret pipe/soil interaction behaviour to facilitate targeted and timely interventions. A field-viable monitoring system will also need to locate the position of distress along the pipeline. Strategies that compute the arrival times of AE propagating in the pipe to sensor locations (e.g., Smith *et al.* 2017b; Heather-Smith *et al.* 2018) are in development and will allow the location of the source to be quantified. Moreover, the design of a field monitoring system and approach should consider sensor spacings, power

supplies in remote locations, wireless networks, real-time signal processing and alerts/warnings via telemetry.

CONCLUSIONS

This study has gone beyond the state-of-the-art and demonstrated that AE can be used to detect, quantify, and interpret uplift pipeline-soil interaction behaviour. This new knowledge will enable use of AE monitoring to provide early warning of serviceability and ultimate limit state failures in the field. The AE generated by uplift pipe/soil interaction has been investigated for the first time in a programme of large-scale experiments with different stress levels (via burial depth) and with varying rates of imposed deformation, which has led to the following principal findings:

- AE generation during pipe uplift is proportional to the imposed stress level and pipe displacement rate and is related to evolution of the failure mechanism (i.e., magnitude of pipe displacement).
- AE *b-value* and variance of instantaneous frequency measurements can be used to interpret the onset of pipe displacement and mobilisation of peak uplift resistance.
- Exemplar relationships have been established to quantify the influence of stress level on AE generation (R^2 values of 0.99); AE generation increased linearly with peak uplift force and as a power function with embedment ratio.
- Relationships have also been quantified between AE generation rate (per metre length of pipe) and pipe uplift velocity (R^2 values ranging from 0.95 to 0.98), enabling interpretation of accelerating deformation behaviour that typically accompanies progressive ground failure processes.

- An example interpretation framework has been presented that combines AE parameters to identify the mobilisation of peak uplift resistance and quantify accelerating deformation behaviour during post-peak softening.

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LIST OF SYMBOLS

a	amplitude
A_k	instantaneous amplitude
b -value	measure of the proportion of low and high magnitude events in an AE waveform
C_c	coefficient of curvature
C_u	coefficient of uniformity
D	pipe diameter
D_r	relative density
d_{50}	median particle size
E_A	cumulative envelope area
e_{max}	maximum void ratio
e_{min}	minimum void ratio
f_k	instantaneous frequency
G_s	specific gravity of particles
H	burial depth
L	pipe length
M	log-scale measure of AE magnitude
P	uplift force
p'	mean effective stress
RDC	ring-down counts
s_k	waveform sample (1-second long)
$\hat{s}_k$	Hilbert transform of s_k
x_k	analytic signal
δ'	soil/steel interface friction angle
δ_p	pipe displacement at peak uplift
θ_k	phase function
μ_A	mean value for the envelope
σ_f	variance of the instantaneous frequency
ϕ'_{crit}	critical state friction angle

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TABLES

Table 1. Physical properties of the quartz Leighton Buzzard Sand (LBS)

Property	LBS 2.0-3.35
Particle size range (mm)	2.0-3.35
Median particle size, d_{50} (mm)	2.65
Coefficient of uniformity, C_u	1.34
Coefficient of curvature, C_c	0.96
Minimum void ratio, e_{min}	0.517
Maximum void ratio, e_{max}	0.690
Specific gravity of particles, G_s	2.67
Roundness of particles [‡]	0.18-0.73 (mean = 0.48)
Sphericity of particles [‡]	0.36-0.86 (mean = 0.63)
Critical state friction angle, ϕ'_{crit} (deg)*	34.0
Soil/steel interface friction angle, δ' (deg) [†]	30.3

[‡] The reported particle shape parameters (range and mean) are the result of measurements from 40 particles: roundness and sphericity (circle ratio) were computed using two-dimensional microscope images and the method described in Zheng and Hryciw (2015).

* The reported critical state friction angle was obtained from triaxial testing (initial sample D_r of approximately 85%) described in Smith and Dixon (2019).

[†] The interface friction angle was obtained from interface shearing tests (initial sample D_r of approximately 65%) performed in Smith *et al.* (2020).

Table 2. Summary of test conditions, uplift resistance prediction models and key results

Test		1	2	3
Test conditions	Burial depth, H (m)	0.4	0.7	1
	Embedment ratio, H/D	2.38	4.16	5.94
	Effective stress at pipe springline, p' (kPa)	6.79	11.89	16.98
Uplift resistance prediction models	ALA (2005): peak uplift force, P (kN)	2.07	6.33	12.92
	ALA (2005): pipe displacement at peak, δ_p (mm)	4.0	7.0	10.0
	DNV (2007): peak uplift force, P (kN)	2.73	6.89	12.85
	DNV (2007): pipe displacement at peak, δ_p (mm)	2.5	4.9	7.3
	Schaminée <i>et al.</i> (1990): peak uplift force, P (kN)	2.55	6.70	12.66
Results in this study	Peak uplift force, P (kN)	1.87	6.42	13.02
	Pipe displacement at peak, δ_p (mm)	3.5	7.0	16.0
	Normalised peak mobilisation displacement, $\frac{\delta_p}{H}$ (%)	0.9	1.0	1.6

Note: The experimental results and prediction model outputs are presented for the pipe length in this study (0.985m).

Table 3. Pipe displacement and displacement rate applied in each test stage

Stage	Velocity (mm/min)	Displacement (mm)
1	1	10
2	1	10
3	1	10
4	1	10
5	2	20
6	3	15
7	4	20
8	5	15
9	6	12
10	8	8
11	10	10
12	10	10

Note: Test 1 (0.4m burial) ended after stage 8.

FIGURES

Figure 1. Uplift mechanisms of buried pipes in sand: (a) cross-section diagram of a typical buried pipe with nomenclature; (b) sliding block with vertical shear surfaces; and (c) sliding block with inclined shear surfaces

Figure 2. Annotated photograph of the buried pipe uplift apparatus

Figure 3. Diagrams of the buried pipe uplift apparatus: (a) transverse cross-section; and (b) longitudinal cross-section. The zones with different colour shades show the three burial depths

Figure 4. Particle size distribution of the quartz Leighton Buzzard sand

Figure 5. Displacement rate-controlled program applied to the pipe in each test

Figure 6. Example AE waveform with a suite of time-domain AE quantification parameters superimposed

Figure 7. Flat coded RAD target field used in camera calibration

Figure 8. Resultant displacement vectors after 110mm of pipe displacement for a burial depth of 0.4m (Test 1)

Figure 9. Resultant displacement contours after 70mm of pipe displacement: (a) 0.4m burial (Test 1); (b) 0.7m burial (Test 2); and (c) 1.0m burial (Test 3). Note the figures are to scale and the dashed box around each has dimensions 1.5m x 1.5m

Figure 10. Load (net uplift force) versus displacement responses during pipe uplift

Figure 11. The force-displacement responses measured in this study compared with those obtained by other experimental programmes and outputs from uplift prediction models: (a) dimensionless force versus dimensionless displacement; and (b) dimensionless peak uplift force versus embedment ratio

Figure 12. Load and AE versus pipe displacement during the first 15mm of movement (mobilisation of peak uplift resistance): (a), (b) and (c) show load and AE b -value versus displacement for 0.4m, 0.7m and 1.0m burial, respectively; and (d), (e) and (f) show load and AE variance of instantaneous frequency versus displacement for 0.4m, 0.7m and 1.0m burial, respectively

Figure 13. AE measurements throughout the tests: (a) cumulative AE (RDC) versus time; (b) cumulative area under the AE waveform envelope (V.s); (c) cumulative AE (RDC) versus displacement; (d) cumulative area under the AE waveform envelope (V.s) versus displacement; (e) AE (RDC) generated per mm of imposed displacement in each test stage; and (f) area under the AE waveform envelope per mm of imposed displacement in each test stage. Note that the AE measurements are presented as equivalent per metre length of pipe values

Figure 14. (a) AE generated per mm versus embedment ratio. (b) AE generated per mm versus peak uplift force. Example relationships showing AE measurements from stage 7. Note that the AE measurements are presented as equivalent per metre length of pipe values

Figure 15. AE measurements throughout the tests: (a) AE rate (RDC/min) versus time; (b) mean AE envelope (V) versus time; (c) AE rate (RDC/min) versus applied pipe velocity relationships; and (d) mean AE envelope (V) versus applied pipe velocity relationships. Note that the AE measurements are presented as equivalent per metre length of pipe values

Figure 16. Example use of AE for interpreting mobilisation of peak uplift resistance and accelerating pipe/soil deformation behaviour