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Development of a prototype for modelling soil-pipe interaction and its application for predicting the uplift resistance to buried pipe movements in sand

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Abstract

This paper presents a testing rig for measuring the reactions on rigid pipes buried in sand during episodes of relative displacement. Following a detailed presentation of the 1-g prototype, the test preparation procedure, and the characterisation of the test sand’s shear strength and dilation potential under the low confining stresses pertinent to the problem, the paper focuses on the workflow devised to obtain accurate measurements of friction and arching effects, and accordingly normalise them to account for scale (stress level) effects. Emphasis is put on demonstrating the effectiveness of the sand deposition method for accurately controlling the density of the sample, and on quantitatively assessing its uniformity. Measurements obtained during a series of uplift tests, including reaction force-pipe displacement curves and images of the developing failure surface, facilitated by Particle Image Velocimetry and close-range photogrammetry techniques, are compared against published data and analytical methods. The results lead to the development of a new simplified formula for calculating the uplift resistance to buried pipe movements in sand. These are capable of accounting for scale effects, yet simple enough to be used for the analysis of pipes in practice.

Keywords

pipelines; physical modelling; uplift resistance; particle image velocimetry
1. Introduction

Buried pipelines are frequently subjected to differential ground movements, caused by surface subsidence, urban tunnelling works, seasonal moisture variations in expansive soils, slope instabilities or triggered from the activation of seismic faults. These ground movements may impose detrimental axial tension/compression and bending strain combinations on continuous pipes, the impact of which depends on the amplitude of the imposed displacement and the reaction from the soil surrounding the pipe. Pipe-stress analysis methods for ground movements are commonly based on beam-on-non-linear-spring models (ALA 2005, Karamitros et al. 2007), which require as input the reaction force per unit meter of the pipe along the direction of the relative movement, as a function of the amplitude of the latter. Design guidelines (e.g. ALA 2005, NEN3650) recommend calculating these input parameters via simple analytical models, or on the basis of published results from pipe pull-out and lateral drag tests which are performed to obtain the reaction force on the pipe as a function of the applied displacement.

Most experimental setups used to perform such benchmarking tests cannot account for pipe bending and section deformation effects on the developing soil resistance, and model the plane strain problem of dragging or pulling of a rigid cylinder buried in sand (e.g. Trautmann et al. 1985; White et al. 2001; Cheuk et al. 2008). The scale of 1-g tests, and therefore the maximum model pipe diameter and embedment depth, is limited by testing restrictions even in the simplest scenario of pipe uplift discussed here. One of the first rigs for performing such tests was developed at Cornell University, USA by Trautmann et al. (1985), who used a 4.24m$^3$ testing chamber to conduct uplift 1-g tests on a 102mm-diameter pipe with length 1.20m (length-over-diameter ratio $L/D=11.76$). Results of their pioneering study have been used extensively for the analysis of pipelines in practice, and are embraced by current design guidelines (ALA 2005). Much later, White et al. (2001) used a mini-drum centrifuge to model the uplift of a 22mm-diameter pipe ($D=220$mm in prototype scale) embedded at a normalised depth measured from the pipe springline of $H/D=3.14$. The length-over-diameter ratio of the pipe tested in the centrifuge was $L/D=5.45$. Cheuk et al. (2008) followed a different approach, and tested a pipe with $L/D<1$ in a small-scale 1-g prototype to investigate the sand deformation mechanisms developing during uplift of a 100mm pipe from a normalised depth of $H/D=3$. The study...
of Cheuk et al. is an exception, as the majority of pipe tests reported in the literature were performed on wide 1-g or centrifuge test chambers accommodating pipes with an $L/D$ ratio greater than 5, so as to minimise the end-boundary effects (pipe-chamber friction, sand arching) on the measured response. However, in large 1-g testing chambers, the sand density and uniformity are difficult to control reliably in a simple manner. This means that the accuracy of the measurements is limited, while the scope of the experimental testing programme is inevitably restricted by the time and labour required to prepare each test setup.

This paper presents the components of a small-scale 1-g testing prototype, developed for performing dragging and pulling tests on rigid pipes in sand with $L/D=1$ and a diameter equal to 75mm. The advantages of such a compact rig are not limited to shorter sample preparation times and lower equipment costs. Indeed, it allows tests on pipes embedded at varying depths ($H/D$) in sands of widely varying sand densities, while accurately tracking pipe movements and sand deformation mechanisms during the sample preparation and testing processes using Particle Image Velocimetry (PIV) and close-range photogrammetry. Two custom deposition methods were developed to accurately control the density of the sand bed, while achieving excellent sample uniformity and test repeatability. Sand bed uniformity is quantitatively assessed via a custom quality-control system, which is built around a miniature cone device. In addition to the above, the rig is unique in that it allows full control of the direction of the relative movement, without imposing any fictitious kinematic restraints on the pipe.

A significant part of this paper is devoted to the experimental and numerical methods for eliminating pipe end-boundary effects on the resistance measurements, as well as quantifying the effect of soil arching (due to friction at the glass walls) on the geostatic stress field developed during the process of sand deposition in the chamber. The effect of stress level on the mechanical behaviour of the sand is assessed in parallel via element tests under the low confining stresses which are pertinent to the physical model. Practical outcomes of this experimental study comprise force-displacement curves obtained during a series of uplift tests, and the documentation of the observed failure mechanisms.
Analysis of the results, in the light of existing models, leads to the development of a new formula for predicting the maximum resistance to pipe uplift in loose-to-very dense sands.

2. Testing rig and instrumentation equipment

The components of the testing rig are presented in Figure 1. The dimensions of the testing chamber are 1050mm (length) x 750mm (height) x 75mm (width), and allow testing of a rigid pipe with $L/D=1$. Numerical simulations using the methods described in Kouretzis et al. (2013, 2014) were employed during the early design stages to optimise the dimensions of the chamber. Their goal was to ensure that the expected failure mechanisms will not be affected by the location of the lateral boundaries.

Two 19mm-thick annealed glass windows form an observation plane perpendicular to the longitudinal axis of the pipe (in the plane of principal strains), and allow for digital imaging and Particle Image Velocimetry (PIV). Annealed glass was selected to form the chamber’s main sidewalls due to its low friction coefficient, hence reducing undesirable end-boundary effects. These two viewing windows are fitted into 20mm-wide grooves using a close sliding fit. The grooves were carved into two 25mm-thick aluminium plates, which form the cross walls of the chamber. These cross walls are also fitted into grooves, formed in a 25mm-thick aluminium base plate, allowing the chamber to act as a single rigid unit. The exterior interface between the two glass viewing windows and the aluminium walls was carefully sealed using silicone gel, while bracings were placed at the top and the bottom of the chamber to increase its in-plane stiffness. The chamber is fitted on a steel beam 400mm above the laboratory strong floor, and is fixed on the beam via friction plates at its corners.

Some additional features of the chamber allow lateral drag, pull-out and drag-down tests at various relative offset angles in the vertical principal strain plane to be performed. Five 40mm holes in one of the chamber’s cross walls allow the pipe to be connected to the actuator with a cable, and hence lateral and inclined pull-out tests can be done. Similar holes were also drilled at the half-length of the chamber base plate to allow the pipe to be moved downwards vertically or at an angle. O-ring-sealed PVC caps were fabricated to plug these holes flush with the inner surface of the chamber. Small nozzles (4mm in diameter) were drilled along the base plate, as shown in Figure 1, to allow for water drainage from the bottom of the chamber in future tests. These nozzles were sealed using machined 4mm nylon screws.
The testing chamber is supported by a 2.3m wide x 2.6m high steel frame, fabricated with built-up back-to-back double channel sections, with a 50mm web offset. This offset provides a path for passing the pulling cable and connection wires through the mid-depth of the frame. The frame is supported by cross legs, bolted to the strong floor to provide additional stability to the rig. The frame was designed to withstand dead loads, as well as a concentrated live in-plane load, of 4 tons.

The rigid pipe used in the tests is 75mm in diameter and 15mm in thickness, and has a length-to-diameter ratio \( L/D = 1 \) (Figure 2a). Its core is a PVC cylinder, which fits firmly into the chamber via two fabricated nylon endcaps, trimmed to match the outer diameter of the cylinder and surfaced with felt. Pilot tests revealed that significant frictional resistance occurred when individual sand grains became trapped in the nylon-glass interface. Surfacing the endcaps with felt prevented sand grains from becoming trapped in the pipe-glass interface, and also decreased the coefficient of friction at the pipe-glass wall interface. A 10mm eyebolt is fitted at the mid-length of the pipe (Figure 2a) to anchor the pulling cable via a 10mm ‘D-shackle’. This arrangement does not impose any kinematic constraints at the connection between the pipe and the cable. The total weight of the PVC pipe, the nylon caps and of the eyebolt is 340 grams. A 3mm flexible steel cable runs between the pipe and the actuator, while
the direction of loading is controlled using a custom-built ‘pulley-clamp’ mechanism (Figure 2b) that can be fixed on the frame at any desired position.

![Figure 2](image_url)

**Figure 2.** Details of the (a) pipe, and of the (b) pulley-clamp mechanism.

A 1.5ton electric winch is used to move the pipe, and was retrofitted in-house by adding a gearbox that allows displacement rates of between 1 and 200mm/hr to be applied. A clamping mechanism, similar to that of the pulley-clamp system shown in Figure 2b, is mounted at the bottom of the winch to secure its position on the frame beams.

The cable reaction force is measured with S-type tension-compression load cells. Load cells with capacities of 0.2, 0.5, 1.0, and 10 kN are used, depending on the magnitude of the expected pull-out force, in order to minimise the instrument noise. Measurement of the overall density of the sand bed is achieved via four 100kg button load cells, which are positioned beneath the testing chamber and provide continuous measurement of the soil mass inside the chamber. The pipe displacement is measured with UniMeasure JX-PA series linear position transducers (string potentiometers) which have a precision of 0.1mm. These are connected either directly to the pipe or to an adjacent point, and aligned parallel to the pulling cable so as to obtain the most accurate displacement readings. Instrument measurements were recorded using a DT80 data logger, capable of resolving 1 microvolt DC.

3. **Physical and mechanical properties of the test sand**
Stockton Beach Sand, referred to here as ‘STK sand’ for brevity, is a washed uniform silica sand containing more than 98% quartz, sourced from Stockton near the port of Newcastle, Australia. Its grain size distribution is shown in Figure 3. This fine sand has a mean diameter ($D_{50}$) and a maximum particle size ($D_{\text{max}}$) of 0.36mm and 0.62mm, respectively. It is classified as poorly graded silty sand (SP) according to USCS and as medium sand according to the Australian Standard AS-1289.3.6.1. Minimum and maximum dry densities (measured according to AS-1289.5.5.1) are 14.5kN/m$^3$ and 17.2kN/m$^3$, respectively. Its specific gravity was measured in a gas displacement pycnometer to be $G_s=2.65$. Table 1 lists other grading properties of STK sand.

![Particle Size Distribution Chart](image)

**Figure 3.** Grain size distribution curve of STK sand. The grain size distribution of CU Filter sand used in Trautmann et al. (1985) experiments is also depicted for comparison.

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Ajalloeian et al. (1996) characterised the mechanical behaviour of STK sand via direct shear tests (DST) and isotropically-consolidated drained and undrained triaxial compression tests. Direct shear tests on samples with relative densities between 13% and 92%, performed under normal stresses ranging from 37.9kPa to 379kPa, resulted in a residual friction angle of $\phi_{\text{res}} = 31.1^\circ$. Drained triaxial
tests were also performed on samples with a relative density between 30% and 90% and consolidated isotropically to confining pressures between 30kPa and 400kPa. Additional undrained compression tests were performed on samples with a relative density between 12.7% and 81%, consolidated to pressures between 10kPa and 150kPa. These tests resulted in constant-volume friction angles of $\phi_{cv} = 30.9^\circ$ and $\phi_{cv} = 31.4^\circ$ for drained and undrained conditions, respectively.

In view of the scale of the physical model used in this study, the initial confining stress in the sand bed can be as low as 1.5kPa. Thus, direct shear tests were carried out on STK sand to characterise the behaviour of the material during shearing at low normal stresses between 1.25-100kPa. These element tests and their results are presented in the following sections.

3.1 Modified shear box

Modifications were made to a conventional DST apparatus, to facilitate direct shear testing at normal stresses as low as 1.25kPa. A commercial strain-controlled DST system (Shear Trac II, Geocomp), with a rectangular shear box featuring dimensions of 100mm x 100mm x 42mm, was used for this purpose. The size of the shear box conforms to ASTM D3080-04 requirements (dimensions between $6D_{\text{max}}$ and $10D_{\text{max}}$). The minimum normal stress that can be applied during tests with the standard configuration of this apparatus is 7.5kPa; however, tests at such low normal stresses resulted in measurements of questionable accuracy with significant noise. To allow precise measurement of the sand’s shear strength parameters under the low normal stresses pertinent to the pipe test conditions, the apparatus was retrofitted as follows (Figure 4): i) The string potentiometer, used to measure the vertical offset of the top platen, was replaced by 10mm LVDT displacement sensors to reduce the dead load applied from the rig on the sample; ii) An aluminium bracket was fabricated to house three LVDTs on the top platen in an equilateral triangular pattern (Figure 4a); iii) the standard 4kN-load cell was replaced by a 0.5kN load cell to increase the accuracy of low shear force measurements; iv) a separate data acquisition system was installed to log and record data from the LVDTs and the 0.5kN load cell.

Figure 4b depicts the position of the three 10mm-LVDTs on the top platen. The rationale behind using three LVDTs was to improve the accuracy of the dilation measurements, as well as detect any
potential out-of-plane (perpendicular to direction of shear displacement) rotation of the top cap during
the tests. For all tests, the normal load on the failure plane was taken equal to the sum of the dead
loads from the top platen, LVDTs, loading ram and the self-weight of the sample above the slip plane.

Figure 4. DST apparatus modifications: (a) LVDTs and holding bracket, (b) position of LVDTs on the
top platen.

Loose ($D_r=19\%$) and medium-dense ($D_r=58\%$) samples were carefully prepared to their target dry
densities (equal to the sand bed densities achieved in the testing chamber) using similar pluviation
methods as the ones used to deposit the sand bed inside the chamber, described in section 6. Sand
was poured into the shear box using custom-made end attachments, while maintaining a constant
drop height of 20mm. Considering the scale of the shear box, a different procedure was undertaken to
prepare dense samples at the same density as that in the chamber ($D_r=92\%$), and sand was placed
inside the DST box in three 14mm lifts. After each lift the sample was compacted by dropping a 200g
weight, falling 50mm to the centre of a 95mm x 95mm fiberglass tamping plate. Calibration of the
sample preparation method consisted of determining the number of drops required to achieve the
target density, and resulted in 10 drops/lift. Once the box was full, any excess sand was screened out
using a straightedge, leaving a 2mm gap between the surface of the sample and the top of the box to
accommodate the top platen. All samples were sheared at a constant rate of 1mm/min, while the
vertical gap at the split point of the upper and lower frame was 1.0mm ($\sim 4D_{50}$).

3.2 Direct Shear Test Results

Loose, medium-dense, and dense samples were sheared under normal stresses of $\sigma_n=1.25, 2, 3.5, 5,$
7.5, 10, 30, 50 and 100 kPa. Figure 5a-c shows the stress ratio ($\tau_n/\sigma_n$) and the average vertical
displacement at the centre of the sample as a function of the shear displacement, measured during
tests at normal stresses $\sigma_n=1.25$, 5, 7.5, 50, and 100kPa. The ultimate shear resistance in the loose sand tests with $\sigma_n \leq 5$ kPa was mobilised at very large shear displacements ($\delta_h > 6$ mm). Also, the loose sand exhibited dilation when sheared at low normal stresses $\sigma_n \leq 10$ kPa, behaviour which has also been reported by others (Tatsuoka et al. 1986; Sture et al. 1998; Fannin et al. 2005). Interestingly, a constant-volume state was not reached by the loose sand samples sheared at very low normal stresses ($\sigma_n \leq 7.5$ kPa), even at shear displacements $\delta_h > 8$ mm where the dilation rate remained constant. The medium-dense and dense samples exhibited similar behaviour when sheared at low stress levels. As such, a residual friction angle $\phi_{res}$ was interpreted only from tests that clearly reached a constant volume state. Minor out-of-plane cap rotations were measured by the three LVDTs on the top platen during this set of tests, with a maximum cap rotation of 0.9° during the loose sample test under $\sigma_n=1.25$ kPa.
Figure 5. Results of direct shear tests under normal stress $\sigma_n = 1.25\text{kPa}, 5\text{kPa}, 7.5\text{kPa}, 50\text{kPa},$ and $100\text{kPa}.$

Figure 6 summarises the interpreted peak and residual direct shear friction angles, as a function of the normal stress level. As expected, the peak friction angle ($\phi_{ds,p}$) increases as the normal stress...
reduces, with the rate of increase depending on the initial density of the sample. Other researchers that measured the constant volume friction angle of sands (e.g. Wang and Lade 2001; Alshibli and Williams 2005) concluded that \(\phi_{cv}\) (and its direct shear test counterpart residual friction angle \(\phi_{res}\)), are material parameters which are independent of both the initial sand density and the level of confining stress. The results presented in Figure 6b confirm that \(\phi_{res}\) is not sensitive to the initial density. However, the direct shear tests on STK sand for \(7.5\text{kPa} \leq \sigma_n \leq 100\text{kPa}\) suggest that the residual friction angle increases with reducing normal stress, at a rate that is similar to that of the peak friction angle. According to Lehane and Liu (2013), the apparent stress-dependency of \(\phi_{res}\) observed at low stress levels can be attributed, to some extent, to second-order friction developing between the granular material and the shear box. This phenomenon becomes trivial at normal stresses higher than about 20kPa.
3.3 Interpreted plane strain friction angles

Considering the geometry of the problem at hand, plane strain strength parameters need to be interpreted from the DST data. The plane strain peak friction angle ($\phi_{ps,p}$) corresponds to the point of maximum stress obliquity on the Mohr’s circle of stresses. Assuming coaxiality of the principal stresses and strains (Cole 1967; Stroud 1971a,b), and taking the horizontal direction of shearing to be a zero extension line, allows the plane strain friction angle $\phi_{ps}$ to be correlated to the direct shear friction angle $\phi_{ds}$ as (Davis 1968):

$$\tan \phi_{ds} = \frac{\cos \psi \sin \phi_{ps}}{1 - \sin \psi \sin \phi_{ps}}$$

where $\psi$ is the dilation angle at the corresponding shear stress level $\psi = \tan^{-1}(\Delta\delta/\Delta\delta_h)$, plotted as a function of the normal stress in Figure 7a. Note that a nearly constant dilation angle of 3.6° is observed from tests in loose samples, whereas the rate of volumetric deformation at failure for the medium and dense samples clearly depends on the applied normal stress and decreases as this increases. The peak and critical state plane strain friction angles can be calculated using eq. (1) from their direct shear strength counterparts, $\phi_{ds,p}$ (at $\psi>0$) and $\phi_{res}$ (at $\psi=0$), respectively. The variation of $\phi_{ps,p}$ with normal stress for different sample densities is presented in Figure 7b.
Figure 7. Interpreted plane-strain strength and dilatancy parameters of STK sand.
The critical state friction angle (Figure 7c) can be estimated using eq. (1) when $\phi_{cs} = \phi_{res}$, but complications arise in the absence of an apparent residual state ($\psi=0$) at very low stress levels ($\sigma_n \leq 7.5$ kPa). As an alternative approach, $\phi_{cs}$ was inferred from the slope of a loosely-deposited sand heap whose toe was excavated, which is referred to in the literature as the ‘angle of repose’ (Cornforth 1973; Bolton 1986). The angle of repose of STK sand was measured inside the rig by forming a twin slope in the sand bed and vacuuming sand from the surface in the middle of the chamber. Multiple measurements of the angle of repose were obtained this way, and the mean measured values were $31.8^\circ$ in loose sand beds and $32.1^\circ$ in dense sand beds. This narrow band of values confirms that the angle of repose does not depend on the initial sand density, and is compatible with the critical state friction angle values obtained from element tests. Considering also that the critical state line gradient from the drained triaxial tests performed by Ajalloeian et al. (1996) at confining stresses as low as 30 kPa is $M = 1.261$, corresponding to $\phi_{cs} = 31.5^\circ$, an average value of $\phi_{cs} = 32^\circ$ was eventually assigned to the STK sand.

4. Particle Image Velocimetry

Particle Image Velocimetry (PIV) is a two-dimensional digital image processing technique that, by processing a specific number of sequentially captured images, allows non-intrusive measurement of particle movements along a viewing plane. Displacements are inferred by comparing images of the deformed state with a reference image corresponding to the undeformed (initial) state of the test. PIV was extended to geotechnical testing applications by White et al. (2003), and details on the most recent iteration of the method, GeoPIV-RG, which was used here, are provided in Stainer et al. (2015). The parameters employed in this study are provided in the following paragraph.

The glass sidewalls of the chamber form the viewing plane, perpendicular to the longitudinal axis of the pipe, on which the sand particle movements are tracked as the pipe is lifted. STK sand has a high quality speckle pattern, rated via the standard deviation of the subset pixel intensities ($\sigma_{ls}$) or by the sum of squares of the subset of intensity gradients (SSSIG) (see Stainer et al. 2015), rendering it ideal for PIV applications without the need for artificial seeding or sand dyeing. Digital images were captured with a CCD Canon EOS7D (DSLR) camera, fitted with a 28mm wide-angle prime lens and remotely controlled by a computer. Stable lighting conditions were facilitated by covering the area of
interest with black screens and using artificial lighting. Sensitivity of the matching algorithm to variations in illumination were reduced via the built-in processing algorithm, which used the zero-normalised cross-correlation coefficient (CCZNCC) subset matching available in the GeoPIV-RG software. Two image resolutions of 8 and 18 megapixels are used, depending on the distance between the camera sensor and the viewing window which controls the field-of-view (FoV) in each image. The distance between the camera and the area of interest, and the image resolution are the two factors affecting the captured pixel-per-grain size ratio, which influences the resolution of the sub-pixel (bi-quintic B-spline) interpolation. The optimal ratio depends on both the mean grain size ($D_{50}$) and the texture quality of the grains, and a ratio between 3 and 7 is recommended by Sutton (2008) and Stanier et al. (2016). Considering the texture of STK sand, an average pixel-per-grain size ratio of 4 was employed here. The optimal subset size was determined through an iterative procedure, in which the above constraints were satisfied while maintaining speckle pattern quality requirements ($\sigma_{ls}>15; SSSIG>10^5$) within the limits suggested by Pan et al. (2010) and Stanier et al. (2016). The average subset size of the 8-megapixel and 18-megapixel images was set to 70 and 100, respectively, using subset spacing equal to the selected subset size (zero overlapping). A five-second camera shutter triggering interval (0.2Hz) was used during the tests. This interval was decided via a trial-and-error procedure, and aimed to minimise the error in the measured displacement vectors which results from interpolation between sequential images and updating of the reference image.

5. Friction forces

The different components of friction interfering with the measured reaction force are:

$$ FF_{total} = FF_{pipe-glass} + FF_{pipe-sand} + FF_{cable-sand} + FF_{glass-sand} + FF_{pulley-cable} $$

where $FF_{pipe-glass}$ is the friction between the ends of the pipe and the glass sidewalls; $FF_{pipe-sand}$ and $FF_{cable-sand}$ are the friction between the pipe and the cable surfaces with the sand, respectively; $FF_{glass-sand}$ is the friction between the sand particles and the glass sidewalls; and $FF_{pulley-cable}$ is the pulley-cable friction force developing when the pulley is placed between the load cell and the pipe. All these components, except $FF_{pipe-sand}$, are related to components of the testing rig, and their contribution must be accounted for to estimate the net reaction force on the pipe during an episode of relative movement.
The frictional force $FF_{\text{pipe-glass}}$ develops between the pipe endcaps and the glass sidewalls, and can contribute substantially to the measured resistance. As discussed earlier, a felt layer is attached to the nylon endcaps (Figure 2a) to prevent sand particles from becoming trapped at the pipe-glass interface. Downdrag friction stress also develops along the part of the pulling cable that is buried in the sand ($FF_{\text{cable-sand}}$). However, the magnitude of $FF_{\text{cable-sand}}$ was measured during uplift tests by pulling a free-end cable out of dense sand, and was found to be negligible compared to $FF_{\text{pipe-glass}}$ due to the small diameter of the cable.

The magnitude of $FF_{\text{pipe-glass}}$ and $FF_{\text{pulley-cable}}$ developed during an uplift test, as a function of the pipe displacement, was measured explicitly by performing “friction” tests immediately after each actual buried pipe pull-out test (Figure 8). During each friction test, sand is removed from the chamber, the pipe is placed back to its original position, and is forced to travel along the trajectory of the corresponding pull-out test while measuring the reaction force. The reaction force measured during each friction test is the sum of i) the weight of the attachments between the load cell and the pipe, including the weight of the pipe itself ($W_{\text{pipe}}$); ii) the pipe-glass interface friction ($FF_{\text{pipe-glass}}$); and iii) any friction between the pulley and the cable ($FF_{\text{pulley-cable}}$). A schematic of the procedure is shown in Figure 8, together with buried pipe pull-out and friction measurements from a test in medium-dense sand where the pipe was initially embedded at a depth of $H/D=3$. 
Frictional resistance between sand and glass ($F_{\text{glass-sand}}$) depends on a number of factors, including the density of the sand, microscopic roughness between sand particles and the contacting surface, the relative hardness of the contacting materials, the particle angularity, and magnitude of the normal stresses acting on the interface. Studies on the frictional resistance developing at the interface of sand and glass surfaces (e.g. Butterfield and Andrawes 1972) suggest that the average interface friction angle $\phi_i$ ranges between $\phi_i = 9^\circ$ and $\phi_i = 14^\circ$ for loose and dense sand respectively, while the maximum shear resistance is reached at very small displacements, of the order of 0.08mm.
Friction developing between the mobilised sand grains (located in the failure wedge) and the glass boundaries ($FF_{\text{glass-sand}}$) has a twofold effect on the test results: i) it contributes some resistance to particle movement for the grains in contact with the glass that are located in front of the advancing pipe, and ii) it affects the initial stress field in the chamber due to arching effects.

![Figure 9. Schematic of the failure wedge developed during uplift tests for estimating the friction resistance at the sand-glass interface.](image)

Focusing on the first, the additional resistance to particle movement can be calculated from the geometry of the evolving failure wedge at various stages of testing which, in turn, can be found by means of PIV. For the uplift case examined herein, eq. (3) below is used to calculate the sand-glass friction at peak resistance. This assumes that the stress field in the chamber is geostatic and is based on the wedge-shaped failure mechanism shown in Figure 9:

$$FF_{\text{sand-glass}} = 2\mu'\gamma_{\text{eff}}K_0 \int_0^{H_{\text{ref}}} \left( H - z \right) \left( D + 2z\tan(\theta) \right) dz$$

where $\mu = \tan \varphi_i$ is the friction coefficient at the sand-glass interface with $\varphi_i = 10^\circ$, $12^\circ$ and $14^\circ$ for loose, medium and dense sand, respectively; $\gamma_{\text{eff}}$ is the effective unit weight of the sand bed inside the chamber, defined in section 8; $K_0$ is the at-rest lateral earth pressure coefficient calculated as $K_0 = 1 - \sin(\varphi_{cs})$; $D$ is the pipe diameter; $H$ is the initial pipe burial depth measured from its springline, $H_{\text{ref}}$ is the height of the wedge measured from the pipe springline; and $\theta$ is the inclination of the failure planes relative to the direction of the pipe movement. Linear interpolation is used to estimate the values of $FF_{\text{sand-glass}}$ for smaller pipe displacements, before the at-peak friction resistance is reached.
Arching effects due to friction developing at the sand-glass interface during deposition are a well-known phenomenon, and affect the vertical and horizontal stress distribution during backfilling of fascia retaining walls or the stress field in narrow containers, particularly when a surface surcharge is present (e.g., Paik and Salgado 2003; Terzaghi 1943). The effect of arching on the initial stress field developing in the chamber during the layer-by-layer deposition process is discussed in detail in section 8.

The pipe-sand interface friction ($FF_{\text{pipe-sand}}$) depends on the roughness of the pipe surface, and the interface friction angle varies between $\varphi_p/2$ and $\varphi_p$, with the latter being associated with rough, rusty pipes. Results of various numerical studies (e.g. Yimsiri et al. 2004, Kouretzis et al. 2013) suggest that the effect of the assumed interface friction angle on the peak uplift force is small (≤8%) for the complete range of interface friction angles mentioned above.

6. Sand bed preparation method

Obtaining a wide range of target sand bed densities during deposition, while maintaining sample uniformity, is a very challenging task. It is, however, particularly important for obtaining meaningful, high-quality results. Two semi-mechanised sample preparation methods were developed during this study, tailored to the restrictions imposed by the dimensions of the chamber and the need to accommodate the pipe, pulling cables and connections in the setup without inducing sand disturbance. Target relative densities ranged between $D_r=15\%$ and $D_r=95\%$, to cover the whole range of sand densities of practical interest. The classical definition of the relative density $D_r$ in eq. (4) is used here:

$$D_r = \frac{e - e_{\min}}{e_{\max} - e_{\min}} = \frac{\gamma_{\max}}{\gamma} \times \left(\frac{\gamma - \gamma_{\min}}{\gamma_{\max} - \gamma_{\min}}\right)$$

where $e_{\max}$ and $e_{\min}$ are the maximum and minimum void ratios associated with the minimum ($\gamma_{\min}$) and maximum unit weight ($\gamma_{\max}$) of STK sand, respectively (AS1289.5.5.1; ASTM D4254).

Amongst the soil deposition methods described in the literature, the air (or dry) pluviation technique is commonly employed in studies involving the preparation of large calibration chamber specimens (Vaid and Negussey 1984; Rad and Tumay 1987), and is suitable for preparing sand beds at a wide range of densities.
range of densities. The density of the sand bed deposited in the chamber depends on both the falling pattern and impact (kinetic) energy of sand grains which, in turn, can be controlled by adjusting the deposition intensity (defined as the mass of sand dropped per unit area of the pourer over unit time), the drop height and the travelling speed of the sand pourer. Increasing the drop height results in sand beds of higher relative density, provided that the terminal velocity of the grains is not exceeded (Vaid and Negussey 1988). On the contrary, increasing the deposition intensity results in looser sand beds, as at higher flow rates the frequency of grain collision increases, leading to lower deposition energy and a lower probability of dense packing of the grains. There is less consensus on the influence of the travelling velocity of the pourer on the deposited sand density. Both an increase in density with the travel velocity and insignificant effects have been reported in the literature (Oliveira et al. 2012).

Two semi-mechanised deposition methods were developed during this study, inspired by travelling and stationary air pluviation techniques, and used to prepare samples of loose-to-medium and dense-to-very dense sand, respectively. The methods are described in the following two sections.

6.1 Pouring deposition method

The deposition mechanism, depicted in Figure 10 and Figure 11, consists of a 75-Lt conical hopper acting as a feeder, with a shutter valve at its lower section, hung over the testing rig by a three-axis 1.5-ton roof crane. A 43mm flexible tube allows sand to flow from the hopper to a 34.5mm sand pourer (a rigid PVC tube), with a ball valve is installed between the flexible tube and the sand pourer to control the flow. The sand pourer’s diameter is slightly smaller than the half-width of the chamber, so that each sand bed layer is deposited in two separate runs along the length of the chamber. This sequential side-by-side deposition of each layer facilitates sand deposition around the pipe and the pulling cable. A tailor-made mechanical ‘traveller’, designed to move along the top edges of the chamber, provides support for the sand pourer and allows a constant deposition height to be maintained as the sand builds up in the chamber.
Figure 10. Picture of the sand deposition setup with the sand pourer inside the test chamber.
Figure 11. Schematic of the deposition setup for preparing loose-to-medium dense sand beds.

The particle drop height is fine-tuned through a number of PVC end-attachments of varying lengths, connected to the bottom end of the sand pourer via threaded connectors. The thickness of each layer was decided through a trial-and-error procedure. During these trials it was observed that friction at the
glass sidewalls inevitably influenced the density of the sand bed, both by reducing the impact kinetic energy of the falling grains and by altering the falling pattern of those grains that come in contact with them. This leads to some degree of horizontal non-homogeneity as the density of the sand bed reduces towards the glass boundaries. Another phenomenon influencing homogeneity is the uncontrolled lateral flow of free-falling grains immediately after they reach the surface of the sand bed. These lateral mini-slope failures influence the density in both the horizontal and vertical directions. To mitigate these effects, the sand is deposited in 30mm lifts. At the expense of increased test preparation time, this small thickness minimises the glass-sand friction effects and reduces the lateral spreading of the sand particle to a minimum, leading to improved homogeneity in the horizontal and vertical directions.

The deposition intensity is controlled using custom-made diffuser sieves of varying porosity (opening areas, Table 2), which are fixed between the end-attachments and the rigid pourer. These sieves were built from thin-cut (t=2.5mm) Acetal sheets and were inserted in two separate rows below and above the threaded connector. Details of the end-attachments and of the diffuser sieves used for the calibration of the deposition process are outlined in Table 2 and Table 3. The travelling speed was a function of the deposition intensity, which means the traveller is displaced along the chamber at a variable speed as each 30mm lift is deposited.

<table>
<thead>
<tr>
<th>Type</th>
<th>Number of holes</th>
<th>Hole diameter (mm)</th>
<th>Opening area (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#1</td>
<td>31</td>
<td>1.2</td>
<td>26.4</td>
</tr>
<tr>
<td>#2</td>
<td>19</td>
<td>1.5</td>
<td>25.3</td>
</tr>
<tr>
<td>#3</td>
<td>19</td>
<td>1.75</td>
<td>34.4</td>
</tr>
<tr>
<td>#4</td>
<td>19</td>
<td>2</td>
<td>44.9</td>
</tr>
</tbody>
</table>

Table 3. Details of end-attachments shown in Figure 11.

<table>
<thead>
<tr>
<th>Type</th>
<th>Effective drop height (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1)</td>
<td>50</td>
</tr>
<tr>
<td>(2)</td>
<td>100</td>
</tr>
<tr>
<td>(3)</td>
<td>150</td>
</tr>
<tr>
<td>(4)</td>
<td>200</td>
</tr>
</tbody>
</table>
The global density of the sand bed is estimated using mass measurements from four ‘button’ load cells, placed at the base of the chamber, with an accuracy of 0.1kN/m$^3$. Relative densities between 11% and 67% were achieved for the samples through careful calibration of the travelling deposition mechanism.

6.2 Rainer deposition method

An alternative pluviation mechanism was devised to prepare sand beds of higher density. A ‘rainer’ (Figure 12), comprising three vertically-stacked diffuser sieves spaced 65mm to 75mm apart, was fabricated from 1mm-thick galvanised perforated sheets, having a porosity of 40%. To reduce the deposition intensity of the setup, each sieve was built from two layers of perforated sheets, offset horizontally to achieve a combined porosity of 25%. A stepper motor allows the rainer to be driven inside the chamber. A passage for the pulling cables is provided through a 5mm-diameter tube. Sand is poured into the top sieve from the flexible tube shown in Figure 11 to a height of 60mm, while the sand flow rate is adjusted by the ball valve. Pilot tests were performed to confirm that the rate of sand flow into the rainer does not affect the sand bed density, which depends only on the deposition intensity controlled by the rainer geometry. Nonetheless, the rate at which sand was poured into the chamber was carefully monitored to avoid sand piling up during deposition. The drop height was kept constant by lifting the rainer via the stepper motor after each 30mm layer was deposited. Relative densities of between $D_r=85\%$ and 96% were obtained with this process by calibrating the drop height. The denser sand grain packing achieved by this method is attributed to the unrestricted lateral movement of grains during deposition.

The two deposition methods were calibrated to target three relative density values, corresponding to loose, medium-dense and dense sand, as shown in Table 4. Both deposition methods exhibited excellent repeatability, with the maximum global density offset being 0.3kN/m$^3$ in dense sand. The samples prepared with offsets higher than 0.3kN/m$^3$ from the average densities reported in Table 4 were considered inappropriate for testing, and were disregarded.
Figure 12. Rainer mechanism for preparing dense-to-very dense sand beds.

Table 4. Summary of the deposition calibration and targeted density states.

<table>
<thead>
<tr>
<th>Density</th>
<th>Min deposited unit weight (kN/m³)</th>
<th>Max deposited unit weight (kN/m³)</th>
<th>Average deposited unit weight (kN/m³)</th>
<th>Average relative density (%)</th>
<th>Deposition method</th>
<th>Deposition components</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loose</td>
<td>14.8</td>
<td>15</td>
<td>14.9</td>
<td>19</td>
<td>Pouring</td>
<td>#4</td>
</tr>
<tr>
<td>Medium</td>
<td>15.85</td>
<td>16.1</td>
<td>15.98</td>
<td>58</td>
<td>Pouring</td>
<td>#2</td>
</tr>
<tr>
<td>Dense</td>
<td>16.8</td>
<td>17.1</td>
<td>16.95</td>
<td>92</td>
<td>Raining</td>
<td>-</td>
</tr>
</tbody>
</table>

7. Evaluation of sand bed uniformity

The use of a quantitative method to evaluate the uniformity of the sample along the length of the chamber is an important part of assessing and controlling the quality of the obtained measurements. For this purpose, a 14mm-diameter cone with a tip angle of 60° was used to perform miniature cone penetration tests (mCPT) inside the chamber.

The details of this custom-made miniature cone penetrometer are shown in Figure 13. A 220N mini-load cell is housed inside an aluminium shaft and is water-sealed using 1.5mm rubber sealant O-rings behind the shoulder of the cone. A 2mm rubber sealant quad-ring seals the gap between the cone
and the cone shaft. A 240lb (1kN) linear actuator, attached to a servo-controlled stepper motor, allows penetration tests to be performed at a constant rate of 1.8mm/sec, while resistance measurements are recorded at a frequency of 8Hz via a DT800 data logger. The displacements are measured using a string potentiometer with an accuracy of 0.1mm. A digital inclinometer, with an accuracy of 0.1°, is used to ensure that the cone penetration is vertical.

It should be noted that, owing to the proximity of the cone to the rigid sidewalls of the chamber, the tip resistance measurements are going to be affected by boundary effects and cannot be correlated directly to sand properties. Results from past published studies (Ghionna and Jamiolkowski 1991; Schnaid and Houlsby 1991) suggest that to eliminate calibration chamber boundary effects, the distance to the nearest rigid boundary $L_w$ normalised against the cone diameter $D_c$ must such that $L_w/D_c = 38$ to $60$. Later, Bolton et al. (1999) recommended $L_w/D_c > 10$ to eliminate boundary effects. In our case $L_w/D_c = 37.5/14 ≈ 2.7$, which is well below the threshold values proposed in the literature. However, as the effect of rigid boundaries is the same, regardless of the location of the test along the axis of the chamber, the goal of the mCPT tests is to ensure that the measurements obtained at various sites result in the same resistance profiles with depth, and that the vertical variation of cone resistance does not exhibit inconsistencies that can be attributed to local density changes or layering effects.
The locations of tests inside the chamber are illustrated in Figure 14. These are spaced 150mm apart (>10Dc) to ensure negligible interference. Tests are also performed at the left and right corners of the chamber, 75mm apart from cross walls. Figure 14 depicts the cone tip resistance profiles in loose, medium, and dense samples. No layering effect due to staged deposition is evident in these profiles, while excellent agreement is observed for tests mCPT-1 to mCPT-5. Some deviation is observed in the mCPT results at corner points in loose and medium sand. This is no unexpected, however, and is related to the sequential (side-by-side) layer deposition process which causes the sand pourer to be lifted at the corners in order to change direction. However, these corners are sufficiently far from the regions where the failure surface develops, and their effect on the measured response is negligible.
Figure 14. mCPT penetration resistance profiles in loose (top), medium (mid) and loose (bottom) sand beds. Tests in dense sand beds are halted after penetration of $17 D_c$, when the load cell capacity is reached.
8. Quantification of arching effects on the geostatic stress field

As mentioned earlier, friction shear stresses are mobilised at the sand-glass interface as the layered deposition progresses, resulting in non-uniform sand bed settlements across the narrow width of the chamber. These side-wall shear stresses result in lower vertical (and horizontal) normal stresses, compared to geostatic conditions, as part of the overburden weight is transferred to the side walls. The degree of divergence of the stress field from the geostatic ideal depends on the geometry of the chamber, the properties of the materials, and the sand deposition method used. To consider these effects in the analysis of results, PIV and close range photogrammetry were used in conjunction with numerical simulations of the deposition process. The results of these analyses were validated via experimental measurement of the normal stresses in the chamber.

8.1 Tracking of sand particle movements during deposition

PIV was employed during the deposition process to measure the settlement of sand grains at the side walls due to increasing overburden stress. For this exercise, a loose sand bed of total thickness 732mm was prepared in 24 lifts. The thickness of each layer was 30mm, as mentioned earlier, while the thickness of the initial layer was slightly larger (42mm). Digital photos were captured during the deposition process, covering the region of interest (RoI) shown in Figure 15. The reference point for PIV analysis was taken as the point where the first 11 layers (bed thickness 342mm) were deposited, and the sand surface reached the top of the field-of-view (FoV). As shown in Figure 15, the FoV lies between 130mm and 310mm from the bottom of the chamber.
PIV analysis provides the vertical displacement of the sand grains as the weight of layers 12 to 24 is added in the RoI. Final (after the deposition of all 24 layers) displacement vectors and contours obtained with this method are shown in Figure 16. The lateral component of the displacements is rather small (<8%) and the displacement vectors are predominantly vertical as is shown in Figure 16b. The maximum vertical displacement is 0.98mm at the top boundary of the FoV, while the settlement of sand particles located at the bottom boundary is negligible. Figure 16c presents volumetric strains in the RoI estimated via PIV, after the deposition of 13 layers of sand. The average compressive volumetric strain due to the overburden pressure from the overlying layers is 1.4%. Considering that $e_0=0.738$, the as-deposited relative density is estimated to be $D_r = 24.2\%$. 

Figure 15. PIV deposition setup and close-range photogrammetry area.
8.2 Numerical simulation of the deposition process

The displacements obtained above were used to calibrate a finite element model built in PLAXIS to simulate the deposition process. An outline of this model is shown in Figure 17. For simplicity, 2D plane strain conditions are assumed as the length-to-width ratio of the chamber is equal to 14, and a
section perpendicular to the length of the chamber is considered. The assumption of plane strain conditions is supported by the fact that the lateral displacements depicted in Figure 16 are negligible. The initial 11 layers are deposited (activated) first in the model in 11 distinct steps, and the displacements are subsequently set to zero. Next, thirteen 30mm-deposition lifts are simulated in discrete steps, to replicate the deposition process corresponding to the PIV measurements. The glass sidewalls are considered to be linear elastic. The sand is modelled as an elastic-perfectly plastic material obeying the Mohr-Coulomb failure criterion, with the strength parameters presented in section 3.3 for the average normal stress of 4.5kPa. The modulus of elasticity of the sand was obtained from oedometer tests, which are not presented here for brevity, and is again compatible with the level of confining stress. Assuming an elastic response, the Poisson’s ratio of the sand was taken equal to 0.2 and the lateral earth pressure coefficient equal to 0.25. The glass-sand interface friction angle was determined via back-analysis of PIV data, and a best match was achieved for $\phi = 10.2^\circ$. The material model parameters are listed in Table 5.

![Figure 17. Schematic of the finite element model used to simulate the layer-by-layer deposition process.](https://mc06.manuscriptcentral.com/cgj-pubs)
Table 5. Material parameters used in the numerical simulation of the deposition.

<table>
<thead>
<tr>
<th>Material</th>
<th>Modulus of Elasticity (kPa)</th>
<th>Poisson's ratio</th>
<th>Friction angle (deg)</th>
<th>Dilation angle (deg)</th>
<th>Unit weight (kN/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loose Sand</td>
<td>1700</td>
<td>0.2</td>
<td>37.6</td>
<td>3.6</td>
<td>15</td>
</tr>
<tr>
<td>Glass</td>
<td>30x10⁶</td>
<td>0.2</td>
<td>-</td>
<td>-</td>
<td>24.5</td>
</tr>
</tbody>
</table>

Figure 18 compares the calculated settlement profiles against PIV measurements when the overburden stress from 5, 9, and all 13 layers is applied in the RoI. The shape of the settlement profiles is captured by the numerical model, with the observed absolute differences being acceptable given the scale of the displacements (<1mm). The vertical stress distribution at the centre of the sand bed is also obtained from the numerical model, and is compared against the theoretical geostatic stress distribution in Figure 19.

As expected, the vertical stresses in the chamber are lower than the theoretical ones. A linear fit to the distribution of vertical stresses with depth obtained from PLAXIS suggests that the effective density (considering arching effects) of the sample inside the chamber is $\gamma_{\text{eff}} = 11.5 \text{kN/m}^3$ from which an arching weight factor of $11.5 \text{kN/m}^3/15 \text{kN/m}^3 = 0.77$ can be inferred. In other words, the soil pressure on the pipe must be reduced by a factor 0.77 from its free-field value in order to account for the effect of sand-glass friction on the pipe uplift measurements. The numerical results also suggest that the average principal axis rotation from free-field geostatic conditions is of the order of $\pm 4^\circ$ at the glass boundaries of the chamber. One must bear in mind that the above measurements correspond to the initial sand stress state inside the chamber, before pulling of the pipe is commenced. Once pipe movement begins and the failure wedge is mobilised, friction between sand and glass is released, and an increase in the effective vertical stress around the pipe is expected to take place.

Independent validation of the numerical results was sought by placing a 10mm steel plate, supported by four button load cells, inside the chamber to measure directly the total load applied by the sand bed during deposition. Load cells were positioned evenly along the steel plate to limit plate bending (Figure 20). As the steel plate was narrower than the chamber, a 10mm layer of foam was placed above it to prevent sand grains from escaping from the sides and causing friction effects. Figure 20
depicts this setup, while the measured pressure values during deposition are plotted in Figure 19. Some deviation between the measured soil pressure and the back-analysis of PIV measurements is observed for large relative depths, therefore the maximum test embedment depth was limited to $7D$.

**Figure 18.** Comparison of numerical and measured (PIV) settlement profiles at the RoI during layer-by-layer deposition.

**Figure 19.** Distribution of in-situ vertical stress in the chamber.
9. Soil reaction during pipe uplift tests

This section presents results of a series of uplift tests performed to investigate the effects of sand density and embedment depth on the developed resistance, as function of the pipe displacement. Tests were carried out in loose, medium, and dense sand (Table 4), at four nominal embedment depths of $1.5D$, $3D$, $4D$, and $7D$ measured from the pipe springline. A sand layer of minimum thickness equal to $1.5D$ was first deposited inside the chamber, up to the level of the pipe invert, and the pipe was then placed on the surface of the sand bed. Sand deposition was then continued until the desired embedment depth was achieved.

![Figure 20. Test setup for soil pressure measurements.](image-url)
Repeatability of the experimental procedure was assessed through multiple duplicate tests at different densities. The measured net force-displacement curves were nearly identical, with the most prominent discrepancies between duplicate tests being observed for tests in loose sand (Figure 21). The maximum difference in the measured peak uplift resistance was 6.7%, and corresponded to a test in loose sand with the pipe originally embedded at $H/D=4$.

9.1 Load-Displacement response

Normalised force-displacement curves in loose, medium, and dense sand are presented in Figure 22a to 22c, respectively. The net reaction force values ($F_{vu-net}$) presented in the following are corrected for friction (according to the methodology described in section 5) and for arching effects (according to the mentioned in section 8) by considering a reduced, effective sand unit weight of $\gamma_{eff} = 0.77\gamma$. The pipe uplift movement $\delta_z$ is normalised against its diameter $D$, and the dimensionless force $N_{vu}$ is defined as:

$$N_{vu} = \frac{F_{vu-net}}{\gamma_{eff} HDL}$$

Figure 21. Repeatability of tests in loose sand.
The testing details including the measured unit weight of the sand, the pipe burial depth, the maximum measured dimensionless force and the corresponding displacement at-peak \( \delta_{z, \text{peak}} \), are tabulated in Table 6. In addition, Table 6 includes information about the observed failure mode and the geometry of the failure surface, which are further discussed in section 10.

**Table 6.** Uplift test results.

<table>
<thead>
<tr>
<th>Test number</th>
<th>Sand unit weight, ( \gamma ) (kN/m(^3))</th>
<th>Measured ( H/D )</th>
<th>( \delta_{z, \text{peak}} ) (mm)</th>
<th>( N_{uu} )</th>
<th>Failure mode</th>
<th>Extent of failure zone above pipe crown</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>15.03</td>
<td>1.54</td>
<td>6.8</td>
<td>1.7</td>
<td>Wedge-type</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>15.2</td>
<td>2.93</td>
<td>14.4</td>
<td>2.2</td>
<td>Flow-type</td>
<td>1.7( D )</td>
</tr>
<tr>
<td>3</td>
<td>15.11</td>
<td>3.97</td>
<td>12.8</td>
<td>2.7</td>
<td>Flow-type</td>
<td>1.7( D )</td>
</tr>
<tr>
<td>4</td>
<td>15.18</td>
<td>7.02</td>
<td>49.7</td>
<td>3.6</td>
<td>Flow-type</td>
<td>1.8( D )</td>
</tr>
<tr>
<td>5</td>
<td>16.1</td>
<td>1.45</td>
<td>0.7</td>
<td>1.8</td>
<td>Wedge-type</td>
<td>-</td>
</tr>
<tr>
<td>6</td>
<td>16.01</td>
<td>2.98</td>
<td>2.5</td>
<td>2.8</td>
<td>Wedge-type</td>
<td>-</td>
</tr>
<tr>
<td>7</td>
<td>16.05</td>
<td>3.97</td>
<td>13.7</td>
<td>3.5</td>
<td>Wedge-type</td>
<td>-</td>
</tr>
<tr>
<td>8</td>
<td>15.9</td>
<td>6.94</td>
<td>56.3</td>
<td>5.7</td>
<td>Flow-type</td>
<td>4.5( D )</td>
</tr>
<tr>
<td>9</td>
<td>17</td>
<td>1.52</td>
<td>3.2</td>
<td>2.4</td>
<td>Wedge-type</td>
<td>-</td>
</tr>
<tr>
<td>10</td>
<td>17.14</td>
<td>2.9</td>
<td>5.8</td>
<td>4.0</td>
<td>Wedge-type</td>
<td>-</td>
</tr>
<tr>
<td>11</td>
<td>17.1</td>
<td>3.95</td>
<td>8.8</td>
<td>5.4</td>
<td>Wedge-type</td>
<td>-</td>
</tr>
<tr>
<td>12</td>
<td>17.13</td>
<td>7</td>
<td>15.5</td>
<td>9.0</td>
<td>Wedge-type</td>
<td>-</td>
</tr>
</tbody>
</table>

Notice that in Figure 22 the response is generally characterised by post-peak softening. This is expected, since at very large displacements the reaction force will tend to zero as the pipe is pulled out from soil. During the early stages of the tests the reaction force appears to increase linearly with pipe displacement. This pseudo-elastic behaviour (Rowe and Davis 1982) is more prominent in dense sand tests, where it is observed up to the displacement where the peak resistance is reached (\( \delta_z = 0.05 \) to 0.2\( D \)). Abrupt softening follows this point due to the formation of shear bands. A softer pre-peak response is observed for deeper embedment depths and for loose-to-medium dense samples, where the dilation potential of the sand is suppressed and a “flow-type” failure mode develops (Table 6). The large post-peak oscillations observed in Figure 22 are not associated with instrument noise, but rather indicate a stick-slip resistance mechanism, where each drop in the measured reaction corresponds to the point where the static resistance is exceeded and the friction coefficient reduces from its static to its dynamic value. Similar oscillations were observed by Cheuk et al. (2008), who associated these with the periodic flow-around and infilling mechanism developing around the pipe post-peak. Interestingly, the force-displacement curves are bounded by an envelope, suggesting that
this flow-around phenomenon does not affect the peak resistance. At large displacements, softening is associated with a gradual decrease in the weight of the soil wedge acting on the pipe as failure progresses to the soil surface where it eventually outcrops (with the formation of miniature surface flows).
Figure 22. Load-displacement response measured during uplift tests in (a) loose, (b) medium, and (c) dense sand.

9.2 Peak uplift resistance

In addition to the classical definition of the dimensionless reaction force (eq. 5), where the uplift force is normalised against the weight of the soil acting at the pipe springline, we introduce here the
normalisation of the uplift force against the maximum shear strength of soil at the pipe springline, via
the modified dimensionless reaction force $N_{vu-mod}$:

$$N_{vu-mod} = \frac{F_{vu-net}}{\gamma H \tan(\varphi_{ps,p})}$$

This allows us to account for the fact that the peak friction angle $\varphi_{ps,p}$ depends on the stress level. The
maximum dimensionless uplift force, calculated using eqs. (5) and of (6), is plotted in Figure 23a and
Figure 23b, respectively, as function of the initial pipe embedment ratio $H/D$.

Figure 23 also includes the experimental data obtained by Trautmann et al. (1985) from full-scale pipe
tests in Cornell filter sand CU (Figure 3). Trautmann et al. prepared sand beds at three different
densities with $\gamma = 14.8$, $16.4$, and $17.7$ kN/m$^3$, corresponding to loose ($\varphi=31^\circ$), medium ($\varphi=36^\circ$) and
dense ($\varphi=44^\circ$) CU sand, respectively and tested pipes embedded at $1.5D$, $4D$, $8D$, and $13D$. For
medium and dense CU sand, stress-dependent values of $\varphi_{ps,p}$ from Jung et al. (2016) are used
together with eq. (6) while for loose CU sand, a constant $\varphi_{ps,p}=37.5^\circ$ is considered. Results from the
current study compare well against the results of Trautmann et al., despite a number of differences in
the two experimental setups (pipe diameter, sample preparation methods, sand grain size distribution,
target densities). Notice that the $N_{vu}$ values measured during this study are consistently higher than
the values measured by Trautmann et al. (Figure 23a), which can be attributed to the different pipe
diameters tested. This is corroborated by the comparison of $N_{vu-mod}$ values in Figure 23b.
Measurements from both setups are now confined in a narrow band, as scale effects are accounted
for in eq. (6). An exception is the results of Trautmann et al. for pipes buried in loose sand deeper
than $H=4D$. However, Trautmann et al. reported that these particular measurements were
incompatible with all the published models they reviewed.
Figure 23. Normalised peak uplift resistance plotted against the embedment ratio $H/D$ (a) normalisation based on soil pressure, (b) normalisation based on soil shear strength.

The at-peak displacement $\delta_{z,\text{peak}}$, i.e. the pipe displacement required to reach the peak resistance, can be interpreted using the data in Figure 22. However, one should bear in mind that the initial linear segment of the force-displacements curves in Figure 22 may be influenced by slack in the loading system, leading to an overestimate of $\delta_{z,\text{peak}}$. This issue was identified by Trautmann et al. (1985), who mentioned that the interpreted peak displacements should be considered as upper bounds to the actual values of $\delta_{z,\text{peak}}$. Almahakeri et al. (2013) also acknowledge the effect of initial slack in the pulling system they used to perform lateral tests. To account for this issue, they installed strain gauges at the connection point between the pulling cables and the pipe and estimated the displacement at which the pulling force was acting on the tested pipe. In this study, a small initial slack was allowed between the pipe and the pulling cable, to provide more flexibility for the operator during the preparation of tests in loose and medium sand beds. For embedment depths $H/D \leq 4$, tests were conducted using two string potentiometers, one of which was directly attached to the pipe to obtain slack-free readings of pipe vertical movements. For tests with larger pipe embedments, this approach was deemed inappropriate as friction between sand and the buried segment of the string potentiometer could obstruct the retraction of this cable, and hence readings of the pipe’s stroke.

10. Documentation of sand failure mechanisms during pipe uplift

This section attempts to shed some light on the mechanisms associated with the development of soil resistance to uplift pipe movements, using displacement vectors and shear strain contours depicting the boundaries of the failure surface obtained via PIV (Figure 24 to Figure 26). The presented results correspond to the at-peak pipe displacement, where the maximum resistance is reached. Note that
the distance between the camera and the target-viewing window varied in each test, so as to achieve a FoV of sufficient detail. As such, different image scales are used in each test. Notice the following:

- A localised, flow-type mechanism develops in the loose sand tests (Figure 24), which doesn’t reach the surface at the point where the maximum resistance develops, apart from the shallowest embedment case $H/D=1.5$. This failure mechanism is associated with densification of the sand in the zone above the crown of the pipe, where the displacements are predominantly vertically upwards. The sand displacement vectors gradually rotate as we move away from the crown, towards a collapsing zone, as sand flows to fill the gap formed below the pipe invert as the pipe is pulled upwards. Consequently, a clear wedge-type mechanism is not formed, rendering analytical modelling of this failure mode complicated.

- A clear wedge-type mechanism is observed in the medium-dense (Figure 25) and dense sand tests (Figure 26), which extends all the way to the surface even in cases corresponding to deep embedment ratios. It therefore appears that the predominant mode of failure (localised or general) depends mainly on the mechanical properties of the sand surrounding the pipe, rather than the initial embedment depth. This is further corroborated by the shear strain contours defining the two failure planes, the inclination of which depends predominantly on the sand density (Figure 26).

- The observed inclination angle of the shear bands (relative to the vertical), $\theta$, is of the same order of magnitude as the dilation angle of the sand, $\psi$, a finding which is in line with the sliding block model proposed by White et al. (2001). The inclination of the planes changes towards the surface of the sample, owing to the stress gradient and the dependence of the dilation angle on the confining stress (section 3). Considering the dilation angle $\psi$ that corresponds to the stress level at the pipe springline, the ratio $\theta/\psi$ ranges between 1.2 and 1.77 for the medium-dense sand tests and 1.25 and 1.89 for the dense sand tests. The $\theta/\psi$ ratio consistently increases with $H/D$ as, unlike $\theta$, the dilation angle decreases with the mean stress level.
Figure 24. Sand shear strain contours and displacement vectors at peak uplift resistance. Tests in loose sand at initial embedment depth (a) \(H/D=1.5\), (b) \(H/D=3\), (c) \(H/D=4\), (d) \(H/D=7\).
Figure 25. Sand shear strain contours and displacement vectors at peak uplift resistance. Tests in medium sand at initial embedment depth (a) $H/D=1.5$, (b) $H/D=3$, (c) $H/D=4$. 
Figure 26. Sand shear strain contours and displacement vectors at peak uplift resistance. Tests in dense sand at initial embedment depth (a) $H/D=1.5$, (b) $H/D=3$, (c) $H/D=4$, (d) $H/D=7$.

11. **Comparison with analytical models**

Majer (1955) was the first to propose a limit equilibrium solution to calculate the maximum pull-out capacity of anchor plates in sand by considering vertical slip surfaces. More robust solutions (Merifield *et al.* 2001; White *et al.* 2001), that account for shear band kinematics, consider the failure prism as an inverted trapezoidal block with straight legs and two rectangular sections at its lower and upper bases. As mentioned above, the model proposed by White *et al.* is compatible with the inclined failure planes identified via PIV analysis in Figure 25 and Figure 26. The following equations derived by White *et al.* account for both the weight of the soil contained within the lifting block and the mobilised shear resistance on the sliding planes:

\[
F_w = \gamma_{\text{eff}} H D + \gamma_{\text{eff}} H^2 \tan \psi - \left( \frac{\pi \gamma_{\text{eff}} D^2}{8} \right)
\]

(7)

\[
F_{sh} = \frac{\gamma_{\text{eff}} H^2}{2} \left( \tan \phi_{\text{peak}} - \tan \psi \right) \left[ 1 + K_0 - (1 - K_0) \cos(2\psi) \right]
\]

(8)

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where $F_w$ and $F_{sh}$ are the weight of the soil block and the mobilised shear resistance on the slip planes, respectively. The normalised uplift resistance (per unit length of the pipe) is calculated here as:

$$ N_{vu-mod} = \frac{F_w + F_{sh}}{\tau_{\text{max}} D} $$

The derivation of eqs. (7) and (8) was based on the simplifying assumption that the stress field remains geostatic, thus ignoring the increase in normal stresses acting on the slip plane due to densification of sand in the wedge as the pipe moves upwards. In eqs. (7) and (8), $\psi$ is the inclination of the slip planes which is assumed to be equal to the dilation angle of the sand in the original White et al. paper. According to the PIV results discussed above, the dilation angle $\psi$ provides a lower bound on the inclination of the failure planes presented in Figure 25 and Figure 26.

Application of eqs. (7) and (8) to predict the experimental results obtained in this study is presented in Figure 27. Two different options are considered: i) the inclination of the failure wedge planes is taken equal to the dilation angle of soil at the pipe springline (from Figure 7a); ii) the inclination of the failure wedge planes is interpreted from PIV tests (Figure 25 and Figure 26 and Table 6). In addition the peak friction angle is estimated from the DST results presented in Figure 7b considering the vertical stress level at the pipe springline, and $K_0 = 1 - \sin(\varphi_{cs})$ with $\varphi_{cs}=32^\circ$, as before. Good agreement is obtained in both cases, even for loose sand where a local flow-type mechanism was observed, which suggests that the collapse of the sand grains towards the pipe invert does not contribute significantly to the development of resistance on the pipe. This finding is in line with the observation of an envelope of developing resistance which is shown in Figure 22.
12. Simplified formula for estimating the ultimate uplift resistance

As stated earlier, two components contribute to the resistance to uplift movement viz. the weight of the failure wedge and the mobilised shear resistance. The use of PIV allows us to infer the dimensions of the failure wedge at the pipe displacement where the peak resistance develops, and knowing the unit weight of the soil we can estimate the contribution of the weight of the trapezoidal failure wedge ($\theta > 0$) to the total resistance as $N_{vu,mod}/N_{vuw,\theta > 0}$, where:

$$N_{vu,\theta > 0} = \frac{[\text{Area of failure wedge}]}{HD}$$

and $N_{vu,mod}$ is given by eq. (6). Figure 28a presents the variation of $N_{vu,mod}/N_{vuw,\theta > 0}$ with the normalised embedment depth, for all the experiments performed. As the sand density increases, the contribution of the weight of the failure wedge to the total resistance becomes smaller, or in other words, the shear component increases. It must be noted here that $N_{vuw,\theta > 0}$ can be defined explicitly only when a wedge-type failure is depicted in the PIV displacement patterns. Tests where flow-type failure was observed were, however, included in 28a by ignoring the contribution of the flow-around mechanism and considering an average inclination angle for each soil density. As discussed before, the inclination angle is not particularly sensitive to the embedment of the pipe, therefore average $\theta$ values of $\theta = 4^\circ$. 

Figure 27. Comparison of the results of the limit equilibrium model of White et al. against experimental data considering $\psi$ to be equal to the dilation angle of sand (left) and interpreting directly the inclination angle from PIV results (right).
\(\theta=11^\circ\), and \(\theta=21^\circ\) can be used effectively to estimate the area of wedges in loose, medium, and dense sand beds, respectively.

The results plotted in Figure 28a, suggest that the total normalised uplift resistance can be expressed as:

\[
N_{vu-mod} = N_{vuw, \theta=0} \times \left( a + b \frac{H}{D} \right)
\]

where \(a\) and \(b\) are density-dependent correlation coefficients, depicted in Figure 28a.

**Figure 28.** Contribution of the weight of the mobilised wedge on the total uplift resistance for (a) \(\theta>0\), and (b) \(\theta=0\).

Alternatively, the ultimate uplift resistance \(N_{vu-mod}\) can be correlated to the weight of the vertical column of soil above the pipe only, via

\[
N_{vu, \theta=0} = \frac{[\text{Area of soil column}]}{HD}
\]

as (Figure 28b):

\[
N_{vu-mod} = N_{vuw, \theta=0} \times \left( a' + b' \frac{H}{D} \right)
\]

where \(a' = 1.175\) and \(b'=0.711\) are again correlation coefficients which are independent of the sand density. This approach is facilitated by the fact that the proposed modified normalisation factor \(N_{vu-mod}\) incorporates the non-linear dependence of the shear resistance on the confining stress via the (stress dependent) plane-strain peak friction angle (Figure 7b).
The results obtained with eq. (13) are compared against the experimental data in Figure 29a and against the measurements obtained by Trautmann et al. (1985) in Figure 29b. Despite its simplified nature and its calibration on one dataset only, this equation is able to predict reasonably accurately the results of Trautmann et al. too for a 102mm pipe with $L/D=11.76$. The predictions are less good for the loose sand samples where $H/D > 4$ but, as discussed earlier, these cases are perhaps of questionable accuracy.

13. Concluding remarks

A number of key issues affecting the measurement of sand resistance to buried pipe movements, under controlled laboratory conditions, were addressed in this study, including:

- The (non-linear) dependency of the shear resistance and dilation potential of sands on the confining stress level, which is particularly important for pipes that are buried in shallow trenches.
- Arching effects in narrow testing chambers, which are accounted for here by correcting the geostatic stress field on the basis of PIV tracking of sand particle movements during deposition in the chamber, numerical back-analyses and direct measurements of the transferred stresses.
- Boundary friction effects, which may comprise a considerable portion of the measured resistance when testing pipes with low length to diameter ratios.
- The development of precise sand deposition methods, which are capable of matching the target sand density with an accuracy of 0.3kN/m$^3$ while maintaining sand uniformity, as confirmed quantitatively via miniature cone penetration tests.

Comparing the measurements from the study with published data and available analytical methods showed excellent agreement, provided the uplift resistance values are properly corrected for the influence of friction and arching and are normalised to account for scale (stress level) effects on the reaction force. This led to the development of a new, simple formula for calculating the maximum sand resistance to pipe uplift, but also allows the full potential of the new testing rig to be unlocked. This rig can measure the resistance to pipe movements along practically any direction in the vertical plane which is perpendicular to the pipe axis. The main limitations of the design are: i) the ability to test only rigid, continuous pipes, hence ignoring joint flexibility, pipe bending and section deformation effects on the developing resistance, and ii) its applicability to shallow pipe tests which are not subjected to surface loads. Findings from this study can be used for the analysis of buried onshore (and offshore) pipes in practice, but the excellent repeatability of test results also renders them appropriate for the calibration of methods for simulating the movement of rigid objects in granular materials in general.

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