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CFD-Based Framework for Analysis of Soil-Pipeline Interaction in Reconsolidating Liquefied Sand

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1	A CFD-based framework for the analysis of soil-pipeline
2	interaction in re-consolidating liquefied sand
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14 Abstract

Submarine buried pipelines interact with shallow soil layers that are often loose and prone to fluidization/liquefaction. Such occurrence is possible consequence of pore pressure build-up induced by hydrodynamic loading, earthquakes and/or structural vibrations. When liquefaction is triggered in sand, the soil tends to behave as a viscous solid-fluid mixture of negligible shear strength, possibly

Pisanò et al., May 4, 2020

unable to constrain pipeline movements. Therefore, pipelines may experience 20 excessive displacement, for instance in the form of vertical flotation or sinking. 21 To date, there are no well-established methods to predict pipe displacement in the 22 event of liquefaction. To fill such a gap, this work proposes a computational fluid 23 dynamics (CFD) framework enriched with soil mechanics principles. It is shown 24 that the interaction between pipe and liquefied sand can be successfully analysed 25 via one-phase Bingham fluid modelling of the soil. Post-liquefaction enhancement 26 of rheological properties, viscosity and yield stress, can also be accounted for by 27 linking soil-pipe CFD simulations to separate analysis of pore pressure dissipation. 28 The proposed approach is thoroughly validated against the results of small-scale 29 pipe flotation and pipe dragging tests from the literature. 30

31 INTRODUCTION

Pipeline infrastructure is widely employed in offshore energy developments to 32 transport hydrocarbons from wells to plants for processing and distribution. When 33 directly laid on the seabed, pipelines are often exposed to harsh hydrodynamic 34 loads that may negatively impact their structural performance. Although pipelines 35 can usually withstand large displacements, the set-up of suitable stabilization 36 measures drives major costs in real projects (Cheuk et al., 2008; White and Cathie, 37 2010). A typical stabilization option is to lay pipelines in trenches back-filled with 38 rocks or sand. Pipe trenching can be very expensive, but allows to increase lateral 39 resistance and drastically reduce hydrodynamic forces (Teh et al., 2006; Bai and 40 Bai, 2014). 41

Pipelines buried in sandy backfill may suffer from the consequences of soil

2

liquefaction, since backfills are inevitably loose (uncompacted) and shallow (i.e., 43 at low effective stresses). Liquefaction can be triggered by a number of factors, 44 including structural vibrations, ocean waves, tidal fluctuations, and earthquakes 45 (Sumer et al., 1999; De Groot et al., 2006; Luan et al., 2008). Due to the low 46 strength and stiffness of fluidized soils, segments of buried pipelines may expe-47 rience excessive displacements, for instance in the form of vertical flotation or 48 sinking. In presence of light pipelines, the large unit weight of liquefied sand 49 is often the main flotation trigger. Reportedly, pipes may also float during/after 50 trench backfilling, due to soil liquefaction phenomena taking place behind the 51 backfill plough (Cathie et al., 1996). 52

Following the first pioneering studies in the United States (Pipeline Flotation 53 Research Council, 1966), North Sea offshore developments fostered in-depth re-54 search on how soil liquefaction can impact pipeline stability (Sumer et al., 1999; 55 Damgaard and Palmer, 2001). Relevant outcomes of these research efforts are 56 nowadays reflected by existing industry design guidelines (DNV, 2007a,b). As 57 pipeline routes can hardly avoid all liquefiable areas, geotechnical input to pipeline 58 design must include (i) assessment of liquefaction susceptibility (De Groot et al., 59 2006), and (ii) prediction of pipe displacement possibly induced by soil liquefac-60 tion (Bonjean et al., 2008; Erbrich and Zhou, 2017; Bizzotto et al., 2017). 61

This paper concerns the analysis of buried pipelines interacting with liquefied sand. A novel CFD-based approach is proposed to predict post-liquefaction pipe displacement, accounting for large deformations and re-consolidation effects in the soil. To prioritize applicability, large-deformation modelling of liquefied

Pisanò *et al.*, May 4, 2020

sand as a two-phase mixture was not pursued. Such endeavour was discouraged 66 by the many questions still open about applying traditional soil mechanics to 67 fluidized geomaterials. Instead, a one-phase approach was preferred, combining 68 Bingham CFD modelling and separate analysis of pore pressure dissipation. As 69 detailed in the following, the latter aspect enables to incorporate phenomenological 70 enhancement of rheological soil properties in the 'early' post-liquefaction phase. 71 While emphasis is on formulation and validation of the proposed framework, its 72 applicability to both submarine and onshore infrastructures is noted – a relevant 73 example of the latter case concerns, e.g., the seismic analysis of buried lifelines 74 (Akiyoshi and Fuchida, 1984; Ling et al., 2003; Yasuda and Kiku, 2006; Chian 75 and Madabhushi, 2012; Kruse et al., 2013). 76

77 CFD MODELLING OF LIQUEFIED SAND INTERACTING WITH BURIED PIPES

This section presents conceptual background and formulation of the proposed
 modelling approach, including critical discussion of relevant assumptions.

80 Conceptual background

Soil-structure interaction problems are usually tackled in the framework of continuum solid mechanics. Despite the particulate nature of soils, continuum theories have successfully supported general understanding of soil mechanics and its implications in geotechnical/structural design. Even the presence of pore fluid has been well accommodated in the same framework, owing to the notion of effective stress and the associated 'effective stress principle' (Terzaghi, 1943). When regarded as (continuum) solids, water-saturated soils exhibit frictional non-linear

Pisanò et al., May 4, 2020

⁸⁸ behaviour, and respond to external loads through deformations (both deviatoric and
⁸⁹ volumetric) that are strongly coupled with pore water flow. Typical design require⁹⁰ ments in civil engineering have determined the wide success of small-deformation
⁹¹ approaches along with soil plasticity modelling (Muir Wood, 2014).

The applicability of solid mechanics, however, should be questioned when 92 external loading and hindered water drainage induce pore pressures that are large 93 enough for the mean effective stress (p') to vanish. The occurrence of the latter 94 event, most easily in shallow soil layers, has drastic implications: typical attributes 95 of solid behaviour (grain contacts, shear strength and stiffness) disappear, while 96 the soil begins to flow as a fluidized grain-water mixture. Such flow is nearly 97 incompressible, rate-dependent, and inevitably associated with large deformations 98 (Guoxing et al., 2016). It should be noted that the transition from solid-like to fluid-99 like state is not irreversible, as water drainage and pore pressure dissipation (so-100 called re-consolidation) can eventually re-establish grain contacts and frictional 101 solid-like behaviour. 102

Recent research efforts have been spent to unify the constitutive modelling of granular materials in their solid, 'transitional' and fluid states (Andrade et al., 2012; Prime et al., 2014; Vescovi et al., 2019). However, application of such approaches to boundary value problems is still far from trivial, also due to dearth of numerical methods and software able to cope with two-phase media and deformations of any magnitude.

¹⁰⁹ A practice-oriented approach is here proposed to analyse the interaction be-¹¹⁰ tween buried pipes and liquefied sand. The following simplifying assumptions

Pisanò *et al.*, May 4, 2020

were formulated in light of relevant experimental evidence:

- for practical purposes, it is possible to idealize liquefied sand as a one-phase,
 non-Newtonian viscous fluid, and analyse its flow using CFD (see the "CFD
 formulation and numerical solution" section, and equations therein);
- at the onset of post-liquefaction re-consolidation, even moderate dissipation of pore pressure can significantly affect the behaviour of liquefied sand.
 Although genuinely hydro-mechanical, such mechanism can be phenomenologically captured within the same one-phase fluid framework through suitable variations of rheological properties (see Equations (4)–(5));
- 3. Post-liquefaction pore pressures needed for the update of liquefied sand's
 Bingham rheological properties can be separately estimated through two phase, small-deformation analysis of re-consolidation (see Equations (11)–
 (12)).

124 Rheology of liquefied sand

The study of fluidized soils, including liquefied sand, has attracted numerous 125 researchers with an interest in earthquake engineering (Seed et al., 1976; Stark 126 and Mesri, 1992; Tamate and Towhata, 1999; Olson and Stark, 2002) and/or prop-127 agation of flow-slides and debris-flows (Pierson and Costa, 1987; Uzuoka et al., 128 1998; Parsons et al., 2001). Although their nature is intrinsically multi-phase, one-129 phase CFD modelling has gained wide popularity, e.g., for simplified simulation 130 of debris avalanches (Boukpeti et al., 2012; Pastor et al., 2014) or seismic lateral 131 spreading (Uzuoka et al., 1998; Hadush et al., 2000; Montassar and de Buhan, 132

¹³³ 2013). In fact, adopting a one-phase approach brings about significant modelling
¹³⁴ advantages while preserving, if properly implemented, features of behaviour rel¹³⁵ evant to engineering applications. The advantages of this approach include (i)
¹³⁶ simpler formulation of (one-phase) field equations and constitutive relationships
¹³⁷ (without 'two-way' hydro-mechanical coupling), (ii) reduced computational costs,
¹³⁸ and (iii) no numerical difficulties related to vanishing effective stresses when soil
¹³⁹ liquefaction occurs.

Soil-water mixtures with high solid concentration (i.e., beyond 35 % in volume) are most often modelled as non-Newtonian Bingham fluids (O'Brien and Julien, 142 1988). Accordingly, the relationship between deviatoric stress and strain rate 143 tensors is assumed to be linear above a so-called 'yield stress', below which no 144 flow occurs. In the case of one-dimensional shear flow, the Bingham model reads 145 as a simple uniaxial relationship between shear stress (τ) and shear strain rate ($\dot{\gamma}$):

$$\begin{cases} \tau = \tau_y + \eta \dot{\gamma} & \text{if } \tau > \tau_y \\ \dot{\gamma} = 0 & \text{otherwise} \end{cases}$$
(1)

where η and τ_y represent viscosity and yield stress of the fluidized soil, respectively. In case of 2D/3D flow problems, multi-axial representation of stresses and strain rates is necessary:

$$\begin{cases} \sigma_{ij} = s_{ij} + p\delta_{ij} \\ \dot{\varepsilon}_{ij} = \dot{e}_{ij} + \frac{\dot{\varepsilon}_{vol}}{3}\delta_{ij} \end{cases}$$
(2)

with the stress (σ_{ij}) and strain rate $(\dot{\varepsilon}_{ij})$ tensors decomposed into their deviatoric (s_{ij} and e_{ij}) and isotropic (p and $\dot{\varepsilon}_{vol}$) components – δ_{ij} is the second-order identity tensor. Accordingly, Equation (1) can be generalized as follows (Cremonesi et al., 2011):

$$\begin{cases} s_{ij} = \tau_y \frac{\dot{e}_{ij}}{\|\dot{e}_{ij}\|} + 2\eta \dot{e}_{ij} & \text{if } \|s_{ij}\| > \tau_y \\ \dot{e}_{ij} = 0 & \text{otherwise} \end{cases}$$
(3)

where $||s_{ij}|| = \sqrt{(1/2) s_{ij} s_{ij}}$ and $||\dot{e}_{ij}|| = \sqrt{(1/2) \dot{e}_{ij} \dot{e}_{ij}}$ are the norms of deviatoric 153 stress and strain rate tensors, respectively. Total $(\dot{\epsilon}_{ij})$ and deviatoric (\dot{e}_{ij}) strain 154 rate tensors coincide in case of incompressible flow, i.e., when $\varepsilon_{vol} = 0$ at all times. 155 Decades of research have revealed broad variability of rheological parameters 156 (Tamate and Towhata, 1999; Parsons et al., 2001; Hwang et al., 2006), particularly 157 of viscosity. According to Montassar and de Buhan (2013), "obtained data for the 158 equivalent Newtonian viscosity coefficients range between 10^{-1} and 10^7 Pa·s". Not 159 only 'intrinsic' factors (e.g., soil mineralogy, porosity, and grain size distribution) 160 contribute to such variability, but also the lack of standard procedures for the 161 interpretation of laboratory tests (Della Vecchia et al., 2019). 162

163 Enhancement of rheological properties during re-consolidation

The large permeability of sandy soils often enables water drainage soon after liquefaction. As a consequence, pore pressure dissipation and concurrent increase in mean effective pressure (p') gradually bring the soil back to its solid-like

state (re-consolidation). The earliest stage of such transition is characterized by liquefied sand that still flows as a fluid, though with rheological behaviour directly affected by ongoing re-consolidation. Capturing this rapid process is relevant to the analysis of soil-structure interaction, for instance, during pipe flotation. To preserve the applicability of Bingham CFD modelling, quantitative information about post-liquefaction rheology (i.e., values and time evolution of rheological parameters) should be included in numerical calculations.

Data from experimental studies can be used in support of the above idea, i.e., 174 to describe the dependence of η and τ_y on p' when $r_u < 1$ (Nishimura et al., 175 2002; Gallage et al., 2005; Towhata et al., 2010; Guoxing et al., 2016; Chen 176 et al., 2013, 2014; Lirer and Mele, 2019) – r_u is the ratio between current pore 177 pressure and pre-liquefaction effective mean stress p'_0 . Particularly meaningful is 178 the work of Gallage et al. (2005), who inferred Bingham properties by subjecting 179 sand specimens at low p' to steps of axial compression at constant pore pressure. 180 Figure 1 displays values of η and τ_y measured for low mean effective stress, with 181 p' lower than 20 kPa – note that such low values are fully representative of soil 182 effective stresses near the onset of liquefaction. Small increments in p' produce 183 remarkable increase in η and τ_{y} , especially when compared to values extrapolated 184 for p' = 0 ($r_u = 1$). All the tests performed by Gallage et al. (2005) show 185 pronounced viscous behaviour at very low p', which corroborates the assumption 186 of fluid-like sand behaviour also in the early post-liquefaction phase. 187

188

As for CFD modelling, the data in Fig. 1 suggest that both τ_y and η may be

Pisanò *et al.*, May 4, 2020

¹⁸⁹ split into two components:

$$\tau_{y} = \tau_{y}^{0} \left(r_{u} = 1 \right) + \tau_{y}^{rec} \left(r_{u}, p_{0}^{\prime} \right) \tag{4}$$

$$\eta = \eta^{0} \left(r_{u} = 1 \right) + \eta^{rec} \left(r_{u}, p_{0}^{\prime} \right)$$
(5)

with τ_y^0 and η^0 material parameters related to fully liquefied conditions ($r_u = 1$), and τ_y^{rec} and η^{rec} variable components evolving during re-consolidation, i.e., as p'gradually increases from zero. τ_y^{rec} may be physically associated with recovery of shear strength:

$$\tau_y^{rec} = A_{\tau_y} p' \approx \frac{M}{\sqrt{3}} p' \tag{6}$$

Figure 1a supports the idea of linking the material coefficient A_{τ_v} to the critical 194 stress ratio M of the fully re-consolidated soil, which lies in the 0.9 - 1.4 range for 195 friction angles between 25° and 35°. The factor $1/\sqrt{3}$ in (6) is consistent with the 196 multi-axial formulation in (3) of a circular yield criterion in the deviatoric π -plane. 197 It should also be noted that, as r_u decreases, τ_v^{rec} quickly grows much larger than 198 au_y^0 , the latter being reported to be usually lower than 100 Pa in fully liquefied 199 sand (O'Brien and Julien, 1988; Uzuoka et al., 1998; Parsons et al., 2001; Pierson, 200 2005). 201

The (rare) data in Figure 1b hints to adopt, as a first approximation, linear p'-dependence for η^{rec} as well:

$$\eta^{rec} = A_{\eta} p' \tag{7}$$

Pisanò et al., May 4, 2020

in which the material parameter A_{η} is unfortunately difficult to identify on a micromechanical basis. Figure 1b indicate A_{η} values in the range of 5 – 15 Pa·s/Pa.

207 CFD formulation and numerical solution

The interaction between buried pipe and liquefied sand has been studied 208 throughout this work as a fluid-structure interaction problem. CFD simulations 209 were performed using the Particle Finite Element Method (PFEM), in the ver-210 sion developed by Cremonesi et al. (2010, 2011) after Idelsohn et al. (2004). 211 The PFEM has been widely applied to engineering applications, such as fluid 212 dynamics (Idelsohn et al., 2004; Oñate et al., 2014a), fluid-structure interaction 213 (Idelsohn et al., 2006; Franci et al., 2016; Zhu and Scott, 2014), bed erosion (Oñate 214 et al., 2008), manufacturing processes (Oñate et al., 2014b), landslides (Cremonesi 215 et al., 2017) and granular flows (Zhang et al., 2014), and recently simulation of 216 cone penetration in water-saturated soils (Monforte et al., 2017). The PFEM 217 adopts a fully Lagrangian description of free-surface fluid flow, especially suitable 218 for fluid-structure interaction problems. 219

In a fully Lagrangian framework, conservation of linear momentum and mass must be fulfilled over the moving fluid volume Ω_t during the time interval (0, T):

$$\rho \frac{Dv_i}{Dt} = \sigma_{ij,j} + \rho b_i \quad \text{in } \Omega_t \times (0,T)$$

$$v_{i,i} = 0 \quad \text{in } \Omega_t \times (0,T)$$
(8)

where Dv_i/Dt represents material time differentiation applied to components of

local velocity v_i , while σ_{ij} , ρ , and b_i stand for total (Cauchy) stress tensor, mass density, and external body force vector, respectively.

Following the PFEM, governing equations were discretized in space with linear interpolation functions for velocity and stress variables; backward Euler time integration was performed along with Newton-type step iterations. The inevitable mesh distortion associated with large deformations was remedied through a remeshing procedure based on Delaunay tessellation (Cremonesi et al., 2010). A plane-strain 2D version of the above method was adopted.

The pipe was modelled as a rigid body, whose translation in time is governed by the following equilibrium equation:

$$\rho_p A_p \ddot{w}_i = \underbrace{W_i^p}_{\rho_p g_i A_p} + \underbrace{F_i^{fluid}}_{\int_{\Gamma_p} \sigma_{ij} n_j \, \mathrm{d}\Gamma_p} + \underbrace{F_i^{struct}}_{-K_{struct} w_i}$$
(9)

where w_i is the displacement vector of the pipe centroid, ρ_p and A_p the mass density 233 and cross-section area of the pipe, and $[g_i] = [0 \ 0 \ -9.81]$ m/s² the gravity 234 acceleration vector. The force terms on the right-hand side relate to pipe weight 235 (W_i^p) , interaction with the fluidized soil (F_{fluid}) , and other structural restoring 236 forces (F_i^{struct}), respectively. F_i^{fluid} represents the integral of fluid stresses (σ_{ii}) 237 along the lateral surface of the pipe (Γ_p , with n_j its normal unit vector), and 238 includes both buoyancy and drag effects. Whenever applicable, F_i^{struct} reflects the 239 considered structural system, and was assumed to linearly depend on w through 240 a (case-specific) elastic stiffness K_{struct} . The rotational degree of freedom is not 241

12

relevant to the applications addressed in the following, and therefore not considered
in Equation (9).

The interaction between pipe and liquefied sand was captured via a staggered 244 Dirichlet-Neumann scheme (Cremonesi et al., 2010). At each time step, the 245 velocity of the rigid body was applied to the fluid interface as a Dirichlet boundary 246 condition; after solving the CFD problem in the surrounding fluid (Equation (8)), 247 stresses along the pipe boundary were integrated to obtain the F_i^{fluid} term in 248 Equation (9), and then update location and velocity of the pipe in the PFEM 249 model. This staggered procedure was performed iteratively for each time-step 250 until convergence (Figure 2). Overall, the proposed approach relies on the time-25 domain solution of Navier-Stokes equations (8) for an incompressible Bingham 252 fluid, whose yield stress and viscosity are updated in space/time through Equations 253 (4)–(7). Such update is based on current p' values obtained by separately solving 254 the re-consolidation model described in the following. A synopsis of the proposed 255 approach is provided in Figure 2. 256

²⁵⁷ Pore pressure dissipation during re-consolidation

The numerical solution of system (8) requires a suitable constitutive relationship between stresses and strain rates in the liquefied sand. To this end, Bingham modelling with evolving rheological parameters was adopted to capture re-consolidation effects in the early post-liquefaction phase. According to Equations (6)–(7), the enhancement of τ_y and η , depends on the current effective mean stress p', which is in fact not a variable in the one-phase CFD model. The analyses of soil-pipe interaction and pore pressure dissipation were therefore decoupled,

with the latter reduced in practice to a 1D problem. This choice corresponds to assuming that the presence of the pipe does not severely affect the pore pressure field (as well as p') in the re-consolidating soil.

Pore pressure dissipation (re-consolidation) in a horizontal soil layer was simulated using Terzaghi's effective stress 1D theory (Terzaghi, 1943). Accordingly, the recovery of p' occurs at expense of the excess pore pressure u_e :

$$p'(z,t) = [1 - r_u(z,t)] p'_0 = -\Delta u_e(z,t)$$
(10)

for any time (t) and depth below the soil surface (z), starting from the initial 271 condition p'(z,0) = 0 (fully liquefied soil layer). While the bulk of Terzaghi's 272 theory was held valid, some changes were motivated by the highly non-linear 273 behaviour of sand at very low p'. Indeed, a number of experimental studies show 274 that, during re-consolidation, both hydraulic conductivity k and 1D oedometer 275 stiffness E_{oed} (= $1/m_v$, oedometer compressibility) depend strongly on the current 276 effective stress level and void ratio (Brennan and Madabhushi, 2011; Haigh et al., 277 2012; Adamidis and Madabhushi, 2016). 278

The evolution of the excess pore pressure field $u_e(z, t)$ was simulated by solving the following diffusion equation (Adamidis and Madabhushi, 2016):

$$\frac{\partial u_e}{\partial t} = \frac{E_{oed}}{\gamma_w} \frac{\partial}{\partial z} \left(k \frac{\partial u_e}{\partial z} \right) \tag{11}$$

where γ_w represents the unit weight of pore water. Along with u_e , the evolution of the void ratio *e* (ratio of the volume of the voids to the volume of solids, and

14

related to porosity as $\phi = e/(1+e)$ was also obtained as:

$$\frac{\partial e}{\partial t} = \frac{1+e}{E_{oed}} \frac{\partial u_e}{\partial t}.$$
(12)

The empirical relationship proposed by Adamidis and Madabhushi (2016) was adopted for the hydraulic conductivity:

$$k = C_T \frac{e^3}{(1+e)} \left[1 + 0.2 \exp(-100\sigma'_{\nu}) \right]$$
(13)

in which C_T is a constitutive parameter, σ'_{ν} the vertical effective stress (in kPa), and *k* is expressed in *m/s*. In agreement with empirical evidence (Haigh et al., 2012), explicit dependence of *k* on σ'_{ν} appears in Equation (13).

A number of 'compression models' are available in the literature for the 1D oedometer stiffness, typically implying a power-law dependence on the vertical effective stress σ'_{ν} . Among all, the well-established relationship proposed by Janbu (1963) and reappraised by Muir Wood (2009) was adopted:

$$\frac{E_{oed}}{\sigma'_{ref}} = \chi \left(\frac{\sigma'_{\nu}}{\sigma'_{ref}}\right)^{\alpha}$$
(14)

where σ'_{ref} is a reference effective stress value, and α and χ two dimensionless material parameters $-0 \le \alpha \le 1.5$ and $10^0 \le \chi \le 10^6$ (Muir Wood, 2009).

Equation (11) was solved in combination with common initial/boundary conditions:

- fully liquefied soil layer:
$$u_e(z,0) = (\gamma_{sat} - \gamma_w) z \Rightarrow \sigma'_v(z,0) = 0$$

- perfectly draining top boundary: $u_e(0,t) = 0$

- impervious bottom boundary:
$$\frac{\partial u_e}{\partial z}(H,t) = 0$$

where γ_{sat} and *H* are the saturated unit weight of the soil and the depth of the lower boundary, respectively.

302 SIMULATION OF PIPE FLOTATION IN LIQUEFIED SAND

Especially relevant to model validation are the recent tests performed at 303 Deltares (Delft, The Netherlands) to study post-liquefaction pipe flotation (Horsten, 304 2016). Pipe flotation experiments were executed in a large container (length: 4 m, 305 width: 2.5 m, depth: 1.2 m), equipped with a fluidization system at the bottom to 306 create sand samples of low relative density, in the range $D_r = 20 - 40\%$. Ittebeck 307 sand was used for this purpose, a uniform fine sand characterized by $G_s = 2.64$ 308 (specific grain gravity), $D_{50} = 0.165$ mm (median grain diameter), $e_{max} = 0.868$ 309 (maximum void ratio), $e_{min} = 0.527$ (minimum void ratio). Three different high-310 density polyethylene (HDPE) flexible pipes were employed, with different outer 311 diameter and thickness. The experimental set-up sketched in Figure 3 featured 312 a fixed-end pipe buried in a saturated sand layer – the clamped edge was intro-313 duced to more realistically represent a pipeline connected to an existing structure. 314 Geometrical and mechanical properties of the three pipes are listed in Table 1. 315 More details about the experimental set-up can be found in Horsten (2016) – see 316 https://repository.tudelft.nl. 317

Pisanò *et al.*, May 4, 2020

Calibration of re-consolidation model

In the original experimental work (Horsten, 2016), sand re-consolidation tests 319 were performed prior to flotation experiments. Such tests were performed in a 320 0.6 m diameter cylindrical container filled with a 1.2 m thick layer of saturated 32 loose sand, and liquefaction was induced by means of single peak vibrations 322 brought about by a falling weight. Pore pressures were measured by five bespoke 323 transducers placed along depth with 0.2 m regular spacing. Specific reference is 324 made here to Sample #2, reportedly characterized by zero initial relative density 325 (initial void ratio $e_0 \sim e_{max}$). The considered re-consolidation tests provided 326 data useful for calibrating the pore pressure dissipation model described above. 327 Required soil properties and model parameters were directly inferred from Horsten 328 (2016) whenever possible – see Table 2, set 1. 329

Setting the parameter C_T in Equation (13) is crucial in that it governs the 330 reference hydraulic conductivity $k_0 = k(\sigma'_v = 0)$, not directly measurable. A value 331 of $C_T = 4 \cdot 10^{-4}$ m/s was selected (yielding $k_0 = 1.68 \cdot 10^{-4}$ m/s) to reproduce 332 the timescale of pore pressure diffusion in the experiment. This value of C_T is 333 about 1/5 of that suggested by Adamidis and Madabhushi (2016) for Hostun sand, 334 reflecting the fact that the latter soil is significantly coarser ($D_{50} = 0.47$ mm, see 335 Haigh et al. (2012)) and more permeable than Ittebeck sand ($D_{50} = 0.17$ mm, see 336 Horsten (2016)). 337

Regarding the choice of σ'_{ref} , χ and α in Equation (14), Muir Wood (2009) provides some broad guidance. Suggested ranges for sand are $10^2 \le \chi \le 10^3$, while α varies from 0.2-0.3 (over-consolidated) to 0.4-0.8 (normally consolidated).

Pisanò *et al.*, May 4, 2020

Reference stress $\sigma'_{ref} = 100$ kPa (recommended by Muir Wood (2009)) and exponent $\alpha = 1.15$ were set for Ittebeck sand. A mid-range value of $\chi = 5.2 \cdot 10^2$ was selected to complete parameter calibration.

In Figure 4a numerical simulations of u_e isochrones are compared to ex-344 perimental measurements, while Figure 4b shows simulated and measured time 345 evolution of u_e at four different depths. Both plots exhibit good agreement be-346 tween computed and measured values. Further insight can be gained from Figure 347 5, showing computed isochrones of permeability (Figure 5a) and 1D oedometer 348 stiffness (Figure 5b), respectively. In line with Adamidis and Madabhushi (2016), 349 the overall change in k during re-consolidation is rather small, whilst E_{oed} ex-350 periences large variations. Computed stiffness values appear reasonably close to 35 expected small-stress values for clean sand (cf. Lauder and Brown (2014), Haigh 352 et al. (2012)). The performance of the non-linear pore pressure dissipation model 353 is further discussed in Appendix I with respect to test results provided by Adamidis 354 and Madabhushi (2016). 355

356 Pipe flotation tests

The three pipes in Table 1 were subjected to separate flotation tests (Horsten, 2016). In all cases, liquefaction of loose Ittebeck sand was achieved through the impact of a weight falling on the sidewall of the rigid container. Resulting displacements of the pipes were measured in time at several locations along their length. As explained in Appendix II, raw flotation measurements had first to be post-processed to eliminate the effects of spurious rotations caused by imperfect clamping (Horsten, 2016).

Flotation tests were numerically simulated using the proposed CFD framework. 364 2D plane-strain PFEM models were set up, with the soil domain discretized using 365 linear triangular elements – see mesh in Figure 6. Velocity no-slip boundary 366 conditions were imposed along all rigid walls, along with zero pressure at the 367 top surface. Measured/simulated displacements in Figures 7–9 relate to the mid-368 section of each pipe (section 1 in Figure 3). Following Equation (9), the 3D effect 369 of the clamped edge (Figure 3) was incorporated in 2D simulations as an elastic 370 restoring force. The structural stiffness $K_{struct} = (17/384) \cdot L_p^4 / E_p I_p$ associated 37 with the mid-section of a cantilever pipe was identified based on standard structural 372 analysis. 373

Figure 7 shows how the upward displacement of the 200 mm pipe evolved in time during the test on pipe 3 (line with square markers). As expected, the general flotation trend features gradual decrease in pipe velocity until full arrest, after about 15 seconds. The dashed horizontal line in the same figure ('no-soil equilibrium') represents the equilibrium that the same elastic cantilever would theoretically attain under self-weight and fluid buoyancy only. Such equilibrium allows to appreciate the influence of shear drag.

³⁸¹ While the total mass density ρ was directly obtained from available measured ³⁸² soil data (Table 2, set 1), enhanced Bingham parameters (τ_y^0 , η^0 , A_{τ_y} , A_{η}) were ³⁸³ calibrated against the experimental flotation curve in Fig. 7:

- to reduce arbitrarity in calibration, default values $\tau_y^0 = 0$ and A_{τ_y} (M = 1.2) were set. The former reflects the dominance of re-consolidation over the low shear strength at $r_u = 1$, the latter relates to an average (critical state)

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³⁸⁷ friction angle of 30°;

- initial viscosity $\eta^0 = \eta (r_u \approx 1) = 2200$ Pa·s was selected to capture pipe velocity at the onset of flotation;

³⁹⁰ – the last parameter A_{η} was identified to match general trend and final equi-³⁹¹ librium of flotation during re-consolidation.

A very satisfactory agreement between experimental and numerical results was achieved for $\eta^0 = 2200$ Pa·s and $A_\eta = 20$ Pa·s/Pa. The influence of A_η was also parametrically studied to highlight the influence of viscosity enhancement on the timing of pipe flotation (Figure 7). It is worth noting the good consistency between the set of identified parameters (Table 3) and previous inferences from Gallage et al. (2005)'s test results (Figure 1).

Comparing the timing of pipe flotation (Figure 7) and pore pressure dissipation (Figure 4) leads to recognize the substantial influence of early re-consolidation on the final displacement of pipe 3. Even though pore pressures dissipate only slightly in the first 30 seconds of the experiment (by about 100 Pa), non-negligible regains in yield stress and viscosity emerge from Equations (6)–(7).

With the same set of calibrated parameters, similar PFEM simulations were performed to predict the uplift experienced by the mid-sections of pipes 1 and 2. The corresponding plots in Figures 8–9 confirm very satisfactory agreement between experimental and numerical results. The proposed CFD model appears capable to accommodate different degrees of re-consolidation effects for pipes of different size, weight and stiffness.

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409 SIMULATION OF LATERAL PIPE DRAGGING IN LIQUEFIED SAND

The proposed CFD framework was further validated against the lateral pipe 410 dragging experiments presented by Towhata et al. (1999). Reference is made to 411 a 1g physical model test in which a pipe embedded in extremely loose saturated 412 sand was laterally dragged at constant elevation after full liquefaction induced 413 by strong shaking of the container (see Section 2 of Towhata et al. (1999) for 414 details). Towhata et al. (1999)'s experiment was carried out on Toyoura sand, 415 reportedly characterized by $G_s = 2.65$, $D_{50} = 0.17$ mm, and initial void ratio 416 $e_0 = 1.04$. A 30 mm diameter, 300 mm long model pipe was embedded at 300 mm 417 depth (constant during pipe dragging) in a sand stack of 400 mm thickness. Pipe 418 dragging was enforced during post-liquefaction pore pressure dissipation, while 419 pure re-consolidation experiments on Toyoura sand (such as those in Fig. 4) were 420 not performed. 421

Despite high experimental uncertainties and limitations in reported data (Towhata 422 et al., 1999), the 1D re-consolidation model was rather easily calibrated, by de-423 ducing the initial soil's unit weight from e_0 and G_s , and selecting for Toyoura sand 424 a value of $C_T = 4 \cdot 10^{-4}$. This is consistent with the value chosen for Ittebeck 425 sand, which has the same particle mean diameter, and likely similar permeability. 426 Soil parameters in Equation (14) were set within typical ranges after Muir Wood 427 (2009) – see Table 2, set 3. Figure 10 shows the time evolution of simulated and 428 measured excess pore pressure (at the top of the pipe), starting from initial full 429 liquefaction. The beginning and end of pipe dragging are marked on the exper-430 imental curve. Pore pressure dissipation is globally well reproduced, although a 431

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slight offset between simulated and experimental curves is noticeable near when
pipe dragging is arrested.

After calibrating the pressure dissipation model, enhanced Bingham parameters were identified for liquefied Toyoura sand. For this purpose, the experimental force-time curve obtained by Towhata et al. (1999) for a lateral dragging velocity of 8 mm/s and the same (pre-liquefaction) void ratio $e_0 = 1.04$ was used. The same values as above of τ_y^0 and A_{τ_y} were re-used to limit freedom in calibration, while η^0 and A_η were identified as follows:

- the initial viscosity $\eta^0 = \eta (r_u \approx 1) = 300$ Pa·s was selected to capture drag force values at the beginning of lateral dragging;

- the last parameter A_{η} was identified to reproduce the increase in drag force during re-consolidation.

PFEM simulations were set up with a pipe initially still for the first 4 s, allowing for 444 some re-consolidation to occur before lateral dragging (Figure 10). In the absence 445 of any structural connections, $F_i^{struct} = 0$ was set in Equation (9) for the laterally 446 dragged pipe. Figure 11a shows satisfactory agreement between experimental 447 and numerical curves in terms of drag force per unit length. The relevance of 448 re-consolidation stands out when considering the result of a purely Newtonian 449 simulation ($\tau_y^0 = A_{\tau_y} = A_{\eta} = 0$ and $\eta^0 = 300$ Pa·s): without regain in shear 450 resistance, the drag force during pipe dragging at constant velocity would barely 451 vary. 452

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Identified Bingham parameters proved again consistent with existing knowl-

edge on liquefied sand rheology. Particularly, the viscosity enhancement coeffi-454 cient ($A_{\eta} = 13 \text{ Pa} \cdot \text{s/Pa}$) falls exactly within the range indicated by Gallage et al. 455 (2005)'s data in Figure 1b, also very close to the value calibrated to reproduce 456 Horsten (2016)'s flotation tests. The influence of A_{η} on the increase in drag force 457 is parametrically demonstrated in Figure 11b. The same figure also shows that 458 the effect of increasing viscosity (η^{rec} , Equation (5)) prevails over the regain of 459 shear strength, as shown by the relatively low force associated with $A_{\eta} = 0$ (i.e., 460 with increase in τ_y only). Although no specific calibration of A_{τ_y} was attempted, 46 the tentative value in Table 3 is of the same order of magnitude as suggested by 462 Gallage et al. (2005)'s data (Figure 1a). 463

The data in Towhata et al. (1999) provided for further model validation, regarding the relationship between drag force and dragging velocity. Experimental tests were performed for sand samples with $e_0 = 1.03 - 1.05$, and three different velocities – namely, 4, 8, 12 mm/s. Figure 12 illustrates the comparison between experimental and numerical results, showing satisfactory simulation of rate effects.

469 CONCLUDING REMARKS

This work presented a CFD-based approach to analyse the interaction between buried pipelines and liquefied sand, accounting for transient re-consolidation effects. Advanced PFEM simulations were performed in combination with enhanced Bingham modelling of the fluidized soil. The rheological enhancement consisted of an update in space and time of both viscosity and yield strength, based on separate non-linear analysis of pore pressure dissipation. The result was a Lagrangian CFD framework capable of dealing with large deformations and re-consolidation

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without explicit modelling of the transition from fluid-like to solid-like behaviour. The soundness of the proposed approach and related calibration procedures were investigated with reference to the experimental literature regarding the interaction of buried pipes with liquefied sand. It was shown that capturing the regain in yield stress and viscosity induced by re-consolidation impacts positively the evaluation of interaction forces and/or displacements experienced by pipes moving through liquefied sand.

The main novelty of this work is the development of a practice-oriented, simpli-484 fied numerical framework for the analysis of pipeline-soil interaction in the event 485 of soil liquefaction, without the need to model phase transitions in multi-phase ge-486 omaterials. The main model limitations can be considered to be (i) the fact that the 487 pore pressure diffusion model is one-dimensional, and (ii) the phenomenological 488 nature of the proposed law expressing the variation of rheological parameters with 489 pore pressure. Hence, further improvements may be achieved by (i) using 2D/3D 490 pore pressure diffusion models to deal with more complex geometries and bound-491 ary conditions, and (ii) reinforcing the micromechanical link between viscosity 492 enhancement and pore pressure dissipation. 493

The underlying large deformation approach is also expected to suit other flotation triggering mechanisms, e.g., those associated with underwater backfilling of pipeline trenches.

497 DATA AVAILABILITY

All data, models, or code that support the findings of this study are available from the corresponding author upon reasonable request. These include:

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- ⁵⁰⁰ numerical simulation results plotted in the manuscript;
- numerical code for soil-pipe CFD simulations;
- ⁵⁰² numerical code for pore pressure dissipation analysis.

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Appendix I. FURTHER VALIDATION OF THE PORE PRESSURE DISSIPATION MODEL

The above pore pressure dissipation model was further tested against the mea-706 surements recorded by Adamidis and Madabhushi (2016) during re-consolidation 707 centrifuge tests on Hostun sand - experiment OA2-EQ2. Selected parameters for 708 this case are given in Table 2 - set 2, most of which taken from published values. 709 Mid-range values for sand were assigned to χ and α following Muir Wood (2009). 710 Simulated pore pressure isochrones and time profiles are compared in Figure 13 711 to experimental data. Despite the simplicity of the 1D stiffness model (14), all key 712 features of re-consolidation are adequately captured. 713

Although all lying within expected ranges, the two parameter sets in Table 2 exhibit differences due to the sand type and, likely, to the adopted physical modelling strategy (1g vs centrifuge modelling).

717 Appendix II. CORRECTION OF RAW FLOTATION DATA

The original work of Horsten (2016) reported imperfect clamping of the pipe 718 cantilever (Figure 3). As a consequence of such imperfection, all pipes experi-719 enced a component of rigid rotation during flotation, on average of about 0.9° – 720 i.e., approximately 20 mm of additional displacement at the mid-section. This 721 effect is readily visible in the raw displacement data provided by Horsten (2016) 722 and plotted in Figure 14. In order to simplify PFEM simulations, it was decided 723 to post-process the raw measured data and eliminate the effect of undesired rigid 724 rotation. In all cases, it was straightforward to identify and remove the affected 725 branch in each flotation curve, indicated in Figure 14 as 'end of clamp rotation'. 726 Relevant bending was assumed to begin for each pipe at the end of rigid ro-727 tation, and corresponds with the corrected experimental data plotted in Figures 728 7–9. To approximate actual experimental conditions, PFEM simulations were set 729 up with initial conditions consistent with the after-rotation configuration - i.e., 730 including higher initial elevation of the pipe, non-zero initial velocity and sand 731 re-consolidation already developed to some extent. 732

LIST OF SYMBOLS 733

Latin symbols 734 A_p = pipe cross-section area

- A_{τ_y} = constitutive parameter accounting for yield stress enhancement during 736 re-consolidation 737
- A_{η} = constitutive parameter accounting for viscosity enhancement during 738
- re-consolidation 739
- b_i = body force vector 740

$$C_T$$
 = hydraulic conductivity parameter

$$D_p = pipe diameter$$

$$D_r$$
 = relative density

- D_{50} = median soil particle diameter 744
- e =void ratio 745
- e_{min} = minimum void ratio 746
- e_{max} = maximum void ratio 747
- $E_{oed} = 1D$ oedometer stiffness 748
- E_p = pipe Young modulus 749
- \dot{e}_{ij} = deviatoric strain rate tensor 750

751	g_i = gravity acceleration vector
752	F_i^{fluid} = fluid force on the pipe (per unit length)
753	F_i^{struct} = structural restoring force on the pipe (per unit length)
754	G_s = relative unit weight of soil grains
755	h_p = pipe elevation
756	H = thickness of the consolidating layer
757	I_p = moment of inertia of pipe cross-section
758	k = hydraulic conductivity
759	L_p = pipe length
760	M = soil critical stress ratio
761	$m_v = 1D$ oedometer compressibility
762	n_i = unit vector normal to lateral surface of the pipe
763	p = mean total stress
764	p' = mean effective stress
765	p'_0 = initial mean effective stress
766	r_u = ratio between current pore pressure and initial mean effective stress
767	s_{ij} = deviatoric stress tensor
	38 Pisanò <i>et al.</i> , May 4, 20

768	t = time
769	t_p = pipe thickness
770	T = end time of soil-pipe simulations
771	u_e = excess pore water pressure
772	v_i = velocity vector in the soil domain
773	w_i = pipe displacement vector
774	W_p = pipe weight (per unit length)
775	z = depth below soil surface
776	Greek symbols
777	α = soil stiffness parameter
778	χ = soil stiffness parameter
779	δ_{ij} = Kronecker identity tensor
780	$\dot{\varepsilon}_{ij}$ = strain rate tensor
781	$\dot{\varepsilon}_{vol}$ = volumetric strain rate
782	$\dot{\gamma}$ = shear strain rate
783	γ_w = water unit weight
784	Γ_p = pipe perimeter

Pisanò et al., May 4, 2020

785	$\eta = \text{viscosity}$
786	η^0 = viscosity of fully liquefied soil
787	η^{rec} = viscosity enhancement during re-consolidation
788	$\phi = \text{porosity}$
789	ρ = soil mass density
790	ρ_p = pipe mass density
791	σ_{ij} = Cauchy stress tensor
792	σ'_r = radial component of the effective stress
793	σ'_v = vertical component of the effective stress
794	σ'_{ref} = reference effective stress
795	$\tau = \text{shear stress}$
796	τ_y = yield stress
797	τ_y^0 = yield stress of fully liquefied soil
798	τ_y^{rec} = yield stress enhancement during re-consolidation
799	Ω_t = moving fluid volume

800 List of Tables

801	1	Pipe geometrical/mechanical properties – h_p = pipe elevation, L_p	
802		= length, t_p = cross-section thickness, D_p = outer diameter, A_p	
803		= cross-section area, I_p = cross-section moment of inertia, ρ_p =	
804		HDPE mass density, E_p = HDPE Young's modulus	42
805	2	Re-consolidation model parameters used to reproduce experimen-	
806		tal measurements from Horsten (2016) (set 1), Adamidis and Mad-	
807		abhushi (2016) (set 2) and Towhata et al. (1999) (set 3)	43
808	3	Enhanced Bingham parameters used to reproduce measurements	
809		from pipe flotation (Horsten, 2016) and pipe dragging (Towhata	
810		et al., 1999) tests	44

	h_p	L_p	t_p	D_p	A_p	I_p	
	[mm]	[m]	[mm]	[mm]	$[m^{2}]$	$[m^4]$	
pipe 1	790	3	17	110	0.005	$3.5 \cdot 10^{-6}$	
pipe 2	640	3	33	160	0.013	$1.6 \cdot 10^{-5}$	
pipe 3	500	3	33	200	0.017	$2.3 \cdot 10^{-5}$	
$\rho_p = 950 \text{ kg/m}^3 E_p = 1100 \text{ MPa}$							

Table 1. Pipe geometrical/mechanical properties $-h_p$ = pipe elevation, L_p = length, t_p = cross-section thickness, D_p = outer diameter, A_p = cross-section area, I_p = cross-section moment of inertia, ρ_p = HDPE mass density, E_p = HDPE Young's modulus.

	Н	γ	C_T	e_0	Χ	α	σ'_{ref}
	[m]	[kN/m ³]	[m/s]	[–]	[–]	[–]	[kPa]
set 1	1.2	18.4	$4 \cdot 10^{-4}$	0.88	$7.3 \cdot 10^{2}$	1.15	100
set 2	12	18.7	$1.94 \cdot 10^{-3}$	0.84	$2.8\cdot 10^2$	0.45	100
set 3	0.4	17.7	$4 \cdot 10^{-4}$	1.04	$0.2 \cdot 10^{2}$	0.5	100

Table 2. Re-consolidation model parameters used to reproduce experimental measurements from Horsten (2016) (set 1), Adamidis and Madabhushi (2016) (set 2) and Towhata et al. (1999) (set 3).

	$ au_{y}^{0}$	η^0	A_{τ_y}	A_{η}
	[kPa]	[Pa·s]	[-]	[Pa·s/Pa]
pipe flotation	0	2200	0.6928	20
pipe dragging	0	300	0.6928	13

Table 3. Enhanced Bingham parameters used to reproduce measurements from pipe flotation (Horsten, 2016) and pipe dragging (Towhata et al., 1999) tests.

List of Figures

812	1	Dependence of Bingham parameters on mean effective stress, after	
813		Gallage et al. (2005) – pre-liquefaction relative density $D_r \approx 30\%$,	
814		σ'_r stands for radial effective stress	47
815	2	Solution of a single step in the proposed pipe-soil interaction	
816		algorithm.	48
817	3	Skectch of Deltares' experimental set-up (Horsten, 2016) – dimen-	
818		sions in metres.	49
819	4	Simulation of u_e dissipation – data from Horsten (2016), Sample #2.	50
820	5	Isochrones of sand permeability and oedometer stiffness from the	
821		simulation of Horsten (2016)'s re-consolidation test on Sample #2.	51
822	6	PFEM mesh for the simulation of pipe 1's flotation (Table 1)	52
823	7	Pipe 3's flotation: comparison between CFD results and exper-	
824		imental data from Horsten (2016). Theoretical 'no-soil equilib-	
825		rium' displacement: 21.7 mm	52
826	8	Pipe 1's flotation: comparison between CFD results and exper-	
827		imental data from Horsten (2016). Theoretical 'no-soil equilib-	
828		rium' displacement: 75.5 mm	53
829	9	Pipe 2's flotation: comparison between CFD results and exper-	
830		imental data from Horsten (2016). Theoretical 'no-soil equilib-	
831		rium' displacement: 28.4 mm	54
832	10	Simulation of u_e dissipation during pipe lateral dragging – data	
833		from Towhata et al. (1999)	55

45

834	11	Lateral pipe dragging: comparison between results from exper-	
835		iments and enhanced Bingham simulations at constant dragging	
836		velocity (8 mm/s) and $e_0 = 1.04$ – data from Towhata et al. (1999).	56
837	12	Lateral pipe dragging: influence of pipe velocity on drag force	
838		prior to reconsolidation ($r_u \approx 1$) – data from Towhata et al. (1999).	57
839	13	Simulation of u_e dissipation – data from Adamidis and Madab-	
840		hushi (2016), test OA2-EQ2	58
841	14	Raw flotation curves for pipes 1, 2, 3 – data from Horsten (2016).	59



Figure 1. Dependence of Bingham parameters on mean effective stress, after Gallage et al. (2005) – pre-liquefaction relative density $D_r \approx 30\%$, σ'_r stands for radial effective stress.



Figure 2. Solution of a single step in the proposed pipe-soil interaction algorithm.



Figure 3. Skectch of Deltares' experimental set-up (Horsten, 2016) – dimensions in metres.



Figure 4. Simulation of u_e dissipation – data from Horsten (2016), Sample #2.



every 20 s)

Figure 5. Isochrones of sand permeability and oedometer stiffness from the simulation of Horsten (2016)'s re-consolidation test on Sample #2.



Figure 6. PFEM mesh for the simulation of pipe 1's flotation (Table 1).



Figure 7. Pipe 3's flotation: comparison between CFD results and experimental data from Horsten (2016). Theoretical 'no-soil equilibrium' displacement: 21.7 mm.



Figure 8. Pipe 1's flotation: comparison between CFD results and experimental data from Horsten (2016). Theoretical 'no-soil equilibrium' displacement: 75.5 mm.



Figure 9. Pipe 2's flotation: comparison between CFD results and experimental data from Horsten (2016). Theoretical 'no-soil equilibrium' displacement: 28.4 mm.



Figure 10. Simulation of u_e dissipation during pipe lateral dragging – data from Towhata et al. (1999).



(a) calibration of the enhanced Bingham model



(**b**) influence of the A_{η} parameter

Figure 11. Lateral pipe dragging: comparison between results from experiments and enhanced Bingham simulations at constant dragging velocity (8 mm/s) and $e_0 = 1.04$ – data from Towhata et al. (1999).



Figure 12. Lateral pipe dragging: influence of pipe velocity on drag force prior to reconsolidation ($r_u \approx 1$) – data from Towhata et al. (1999).



Figure 13. Simulation of u_e dissipation – data from Adamidis and Madabhushi (2016), test OA2-EQ2.



Figure 14. Raw flotation curves for pipes 1, 2, 3 – data from Horsten (2016).