

## 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**

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

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

## 31 **INTRODUCTION**

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

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

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

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

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

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

## 77 **CFD MODELLING OF LIQUEFIED SAND INTERACTING WITH BURIED PIPES**

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

### 80 **Conceptual background**

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

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

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

103 Recent research efforts have been spent to unify the constitutive modelling of  
104 granular materials in their solid, 'transitional' and fluid states (Andrade et al., 2012;  
105 Prime et al., 2014; Vescovi et al., 2019). However, application of such approaches  
106 to boundary value problems is still far from trivial, also due to dearth of numerical  
107 methods and software able to cope with two-phase media and deformations of any  
108 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

111 were formulated in light of relevant experimental evidence:

- 112 1. for practical purposes, it is possible to idealize liquefied sand as a one-phase,  
113 non-Newtonian viscous fluid, and analyse its flow using CFD (see the "CFD  
114 formulation and numerical solution" section, and equations therein);
- 115 2. at the onset of post-liquefaction re-consolidation, even moderate dissipa-  
116 tion of pore pressure can significantly affect the behaviour of liquefied sand.  
117 Although genuinely hydro-mechanical, such mechanism can be phenomeno-  
118 logically captured within the same one-phase fluid framework through suit-  
119 able variations of rheological properties (see Equations (4)–(5));
- 120 3. Post-liquefaction pore pressures needed for the update of liquefied sand's  
121 Bingham rheological properties can be separately estimated through two-  
122 phase, small-deformation analysis of re-consolidation (see Equations (11)–  
123 (12)).

#### 124 **Rheology of liquefied sand**

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

133 2013). In fact, adopting a one-phase approach brings about significant modelling  
 134 advantages while preserving, if properly implemented, features of behaviour rel-  
 135 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.

140 Soil-water mixtures with high solid concentration (i.e., beyond 35 % in volume)  
 141 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 ( $\tau$ ) and shear strain rate ( $\dot{\gamma}$ ):

$$\left\{ \begin{array}{l} \tau = \tau_y + \eta \dot{\gamma} \quad \text{if } \tau > \tau_y \\ \dot{\gamma} = 0 \quad \text{otherwise} \end{array} \right. \quad (1)$$

146 where  $\eta$  and  $\tau_y$  represent viscosity and yield stress of the fluidized soil, respectively.  
 147 In case of 2D/3D flow problems, multi-axial representation of stresses and strain  
 148 rates is necessary:

$$\left\{ \begin{array}{l} \sigma_{ij} = s_{ij} + p \delta_{ij} \\ \dot{\epsilon}_{ij} = \dot{e}_{ij} + \frac{\dot{\epsilon}_{vol}}{3} \delta_{ij} \end{array} \right. \quad (2)$$



149 with the stress ( $\sigma_{ij}$ ) and strain rate ( $\dot{\epsilon}_{ij}$ ) tensors decomposed into their deviatoric  
 150 ( $s_{ij}$  and  $e_{ij}$ ) and isotropic ( $p$  and  $\dot{\epsilon}_{vol}$ ) components –  $\delta_{ij}$  is the second-order identity  
 151 tensor. Accordingly, Equation (1) can be generalized as follows (Cremonesi et al.,  
 152 2011):

$$\left\{ \begin{array}{l} s_{ij} = \tau_y \frac{\dot{\epsilon}_{ij}}{\|\dot{\epsilon}_{ij}\|} + 2\eta\dot{\epsilon}_{ij} \quad \text{if } \|s_{ij}\| > \tau_y \\ \\ \dot{\epsilon}_{ij} = 0 \quad \text{otherwise} \end{array} \right. \quad (3)$$

153 where  $\|s_{ij}\| = \sqrt{(1/2) s_{ij}s_{ij}}$  and  $\|\dot{\epsilon}_{ij}\| = \sqrt{(1/2) \dot{\epsilon}_{ij}\dot{\epsilon}_{ij}}$  are the norms of deviatoric  
 154 stress and strain rate tensors, respectively. Total ( $\dot{\epsilon}_{ij}$ ) and deviatoric ( $e_{ij}$ ) strain  
 155 rate tensors coincide in case of incompressible flow, i.e., when  $\epsilon_{vol} = 0$  at all times.

156 Decades of research have revealed broad variability of rheological parameters  
 157 (Tamate and Towhata, 1999; Parsons et al., 2001; Hwang et al., 2006), particularly  
 158 of viscosity. According to Montassar and de Buhan (2013), “obtained data for the  
 159 equivalent Newtonian viscosity coefficients range between  $10^{-1}$  and  $10^7$  Pa·s”. Not  
 160 only ‘intrinsic’ factors (e.g., soil mineralogy, porosity, and grain size distribution)  
 161 contribute to such variability, but also the lack of standard procedures for the  
 162 interpretation of laboratory tests (Della Vecchia et al., 2019).

### 163 *Enhancement of rheological properties during re-consolidation*

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

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

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

188 As for CFD modelling, the data in Fig. 1 suggest that both  $\tau_y$  and  $\eta$  may be

189 split into two components:

$$\tau_y = \tau_y^0 (r_u = 1) + \tau_y^{rec} (r_u, p'_0) \quad (4)$$

$$\eta = \eta^0 (r_u = 1) + \eta^{rec} (r_u, p'_0) \quad (5)$$

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

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

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

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

$$\eta^{rec} = A_{\eta} p' \quad (7)$$

204 in which the material parameter  $A_\eta$  is unfortunately difficult to identify on a  
205 micromechanical basis. Figure 1b indicate  $A_\eta$  values in the range of 5 – 15  
206 Pa·s/Pa.

### 207 **CFD formulation and numerical solution**

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

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

$$\begin{aligned} \rho \frac{Dv_i}{Dt} &= \sigma_{ij,j} + \rho b_i \quad \text{in } \Omega_t \times (0, T) \\ v_{i,i} &= 0 \quad \text{in } \Omega_t \times (0, T) \end{aligned} \quad (8)$$

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

223 local velocity  $v_i$ , while  $\sigma_{ij}$ ,  $\rho$ , and  $b_i$  stand for total (Cauchy) stress tensor, mass  
 224 density, and external body force vector, respectively.

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

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

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

233 where  $w_i$  is the displacement vector of the pipe centroid,  $\rho_p$  and  $A_p$  the mass density  
 234 and cross-section area of the pipe, and  $[g_i] = [0 \ 0 \ -9.81]$  m/s<sup>2</sup> the gravity  
 235 acceleration vector. The force terms on the right-hand side relate to pipe weight  
 236 ( $W_i^p$ ), interaction with the fluidized soil ( $F_{fluid}$ ), and other structural restoring  
 237 forces ( $F_i^{struct}$ ), respectively.  $F_i^{fluid}$  represents the integral of fluid stresses ( $\sigma_{ij}$ )  
 238 along the lateral surface of the pipe ( $\Gamma_p$ , with  $n_j$  its normal unit vector), and  
 239 includes both buoyancy and drag effects. Whenever applicable,  $F_i^{struct}$  reflects the  
 240 considered structural system, and was assumed to linearly depend on  $w$  through  
 241 a (case-specific) elastic stiffness  $K_{struct}$ . The rotational degree of freedom is not

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

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

#### 257 *Pore pressure dissipation during re-consolidation*

258 The numerical solution of system (8) requires a suitable constitutive rela-  
259 tionship between stresses and strain rates in the liquefied sand. To this end,  
260 Bingham modelling with evolving rheological parameters was adopted to capture  
261 re-consolidation effects in the early post-liquefaction phase. According to Equa-  
262 tions (6)–(7), the enhancement of  $\tau_y$  and  $\eta$ , depends on the current effective mean  
263 stress  $p'$ , which is in fact not a variable in the one-phase CFD model. The analyses  
264 of soil-pipe interaction and pore pressure dissipation were therefore decoupled,

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

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

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

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

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

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

281 where  $\gamma_w$  represents the unit weight of pore water. Along with  $u_e$ , the evolution  
 282 of the void ratio  $e$  (ratio of the volume of the voids to the volume of solids, and

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

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

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

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

286 in which  $C_T$  is a constitutive parameter,  $\sigma'_v$  the vertical effective stress (in kPa),  
287 and  $k$  is expressed in  $m/s$ . In agreement with empirical evidence (Haigh et al.,  
288 2012), explicit dependence of  $k$  on  $\sigma'_v$  appears in Equation (13).

289 A number of ‘compression models’ are available in the literature for the 1D  
290 oedometer stiffness, typically implying a power-law dependence on the vertical  
291 effective stress  $\sigma'_v$ . Among all, the well-established relationship proposed by  
292 Janbu (1963) and reappraised by Muir Wood (2009) was adopted:

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

293 where  $\sigma'_{ref}$  is a reference effective stress value, and  $\alpha$  and  $\chi$  two dimensionless  
294 material parameters –  $0 \leq \alpha \leq 1.5$  and  $10^0 \leq \chi \leq 10^6$  (Muir Wood, 2009).

295 Equation (11) was solved in combination with common initial/boundary con-  
296 ditions:

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



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

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

300 where  $\gamma_{sat}$  and  $H$  are the saturated unit weight of the soil and the depth of the  
301 lower boundary, respectively.

### 302 **SIMULATION OF PIPE FLOTATION IN LIQUEFIED SAND**

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

## 318 Calibration of re-consolidation model

319 In the original experimental work (Horsten, 2016), sand re-consolidation tests  
320 were performed prior to flotation experiments. Such tests were performed in a  
321 0.6 m diameter cylindrical container filled with a 1.2 m thick layer of saturated  
322 loose sand, and liquefaction was induced by means of single peak vibrations  
323 brought about by a falling weight. Pore pressures were measured by five bespoke  
324 transducers placed along depth with 0.2 m regular spacing. Specific reference is  
325 made here to Sample #2, reportedly characterized by zero initial relative density  
326 (initial void ratio  $e_0 \sim e_{max}$ ). The considered re-consolidation tests provided  
327 data useful for calibrating the pore pressure dissipation model described above.  
328 Required soil properties and model parameters were directly inferred from Horsten  
329 (2016) whenever possible – see Table 2, set 1.

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

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

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

344 In Figure 4a numerical simulations of  $u_e$  isochrones are compared to ex-  
345 perimental measurements, while Figure 4b shows simulated and measured time  
346 evolution of  $u_e$  at four different depths. Both plots exhibit good agreement be-  
347 tween computed and measured values. Further insight can be gained from Figure  
348 5, showing computed isochrones of permeability (Figure 5a) and 1D oedometer  
349 stiffness (Figure 5b), respectively. In line with Adamidis and Madabhushi (2016),  
350 the overall change in  $k$  during re-consolidation is rather small, whilst  $E_{oed}$  ex-  
351periences large variations. Computed stiffness values appear reasonably close to  
352 expected small-stress values for clean sand (cf. Lauder and Brown (2014), Haigh  
353 et al. (2012)). The performance of the non-linear pore pressure dissipation model  
354 is further discussed in Appendix I with respect to test results provided by Adamidis  
355 and Madabhushi (2016).

### 356 **Pipe flotation tests**

357 The three pipes in Table 1 were subjected to separate flotation tests (Horsten,  
358 2016). In all cases, liquefaction of loose Ittebeck sand was achieved through  
359 the impact of a weight falling on the sidewall of the rigid container. Resulting  
360 displacements of the pipes were measured in time at several locations along their  
361 length. As explained in Appendix II, raw flotation measurements had first to be  
362 post-processed to eliminate the effects of spurious rotations caused by imperfect  
363 clamping (Horsten, 2016).

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

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

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

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

- 387 friction angle of  $30^\circ$ ;
- 388 – initial viscosity  $\eta^0 = \eta(r_u \approx 1) = 2200 \text{ Pa}\cdot\text{s}$  was selected to capture pipe
  - 389 velocity at the onset of flotation;
  - 390 – the last parameter  $A_\eta$  was identified to match general trend and final equi-
  - 391 librium of flotation during re-consolidation.

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

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

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

## 409 SIMULATION OF LATERAL PIPE DRAGGING IN LIQUEFIED SAND

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

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

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

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

- 440 – the initial viscosity  $\eta^0 = \eta(r_u \approx 1) = 300 \text{ Pa}\cdot\text{s}$  was selected to capture drag  
441 force values at the beginning of lateral dragging;
- 442 – the last parameter  $A_\eta$  was identified to reproduce the increase in drag force  
443 during re-consolidation.

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

453 Identified Bingham parameters proved again consistent with existing knowl-

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

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

## 469 **CONCLUDING REMARKS**

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



477 without explicit modelling of the transition from fluid-like to solid-like behaviour.

478 The soundness of the proposed approach and related calibration procedures  
479 were investigated with reference to the experimental literature regarding the in-  
480 teraction of buried pipes with liquefied sand. It was shown that capturing the  
481 regain in yield stress and viscosity induced by re-consolidation impacts positively  
482 the evaluation of interaction forces and/or displacements experienced by pipes  
483 moving through liquefied sand.

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

494 The underlying large deformation approach is also expected to suit other flota-  
495 tion triggering mechanisms, e.g., those associated with underwater backfilling of  
496 pipeline trenches.

#### 497 **DATA AVAILABILITY**

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

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

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702 particle finite element method in opensees.” *Computers and Structures*, 132,  
703 12–21.

704 **Appendix I. FURTHER VALIDATION OF THE PORE PRESSURE DISSIPATION**  
705 **MODEL**

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

714 Although all lying within expected ranges, the two parameter sets in Table  
715 2 exhibit differences due to the sand type and, likely, to the adopted physical  
716 modelling strategy (1g vs centrifuge modelling).

717 **Appendix II. CORRECTION OF RAW FLOTATION DATA**

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

733 **LIST OF SYMBOLS**

734 **Latin symbols**

735  $A_p$  = pipe cross-section area

736  $A_{\tau_y}$  = constitutive parameter accounting for yield stress enhancement during  
737 re-consolidation

738  $A_\eta$  = constitutive parameter accounting for viscosity enhancement during  
739 re-consolidation

740  $b_i$  = body force vector

741  $C_T$  = hydraulic conductivity parameter

742  $D_p$  = pipe diameter

743  $D_r$  = relative density

744  $D_{50}$  = median soil particle diameter

745  $e$  = void ratio

746  $e_{min}$  = minimum void ratio

747  $e_{max}$  = maximum void ratio

748  $E_{oed}$  = 1D oedometer stiffness

749  $E_p$  = pipe Young modulus

750  $\dot{e}_{ij}$  = deviatoric strain rate tensor

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

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  $\alpha$  = soil stiffness parameter  
778  $\chi$  = soil stiffness parameter  
779  $\delta_{ij}$  = Kronecker identity tensor  
780  $\dot{\epsilon}_{ij}$  = strain rate tensor  
781  $\dot{\epsilon}_{vol}$  = volumetric strain rate  
782  $\dot{\gamma}$  = shear strain rate  
783  $\gamma_w$  = water unit weight  
784  $\Gamma_p$  = pipe perimeter



785  $\eta$  = viscosity

786  $\eta^0$  = viscosity of fully liquefied soil

787  $\eta^{rec}$  = viscosity enhancement during re-consolidation

788  $\phi$  = porosity

789  $\rho$  = soil mass density

790  $\rho_p$  = pipe mass density

791  $\sigma_{ij}$  = Cauchy stress tensor

792  $\sigma'_r$  = radial component of the effective stress

793  $\sigma'_v$  = vertical component of the effective stress

794  $\sigma'_{ref}$  = reference effective stress

795  $\tau$  = shear stress

796  $\tau_y$  = yield stress

797  $\tau_y^0$  = yield stress of fully liquefied soil

798  $\tau_y^{rec}$  = yield stress enhancement during re-consolidation

799  $\Omega_t$  = 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,  $\rho_p$  =  
804 HDPE mass density,  $E_p$  = HDPE Young’s modulus. . . . . 42

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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

	$h_p$ [mm]	$L_p$ [m]	$t_p$ [mm]	$D_p$ [mm]	$A_p$ [m <sup>2</sup> ]	$I_p$ [m <sup>4</sup> ]
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	$2.3 \cdot 10^{-5}$
$\rho_p = 950 \text{ kg/m}^3 \quad E_p = 1100 \text{ MPa}$						

**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,  $\rho_p$  = HDPE mass density,  $E_p$  = HDPE Young's modulus.

	$H$ [m]	$\gamma$ [ kN/m <sup>3</sup> ]	$C_T$ [m/s]	$e_0$ [-]	$\chi$ [-]	$\alpha$ [-]	$\sigma'_{ref}$ [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).

	$\tau_y^0$ [kPa]	$\eta^0$ [ Pa·s ]	$A_{\tau_y}$ [-]	$A_\eta$ [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.

811 **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  $\sigma'_r$  stands for radial effective stress. . . . . 47

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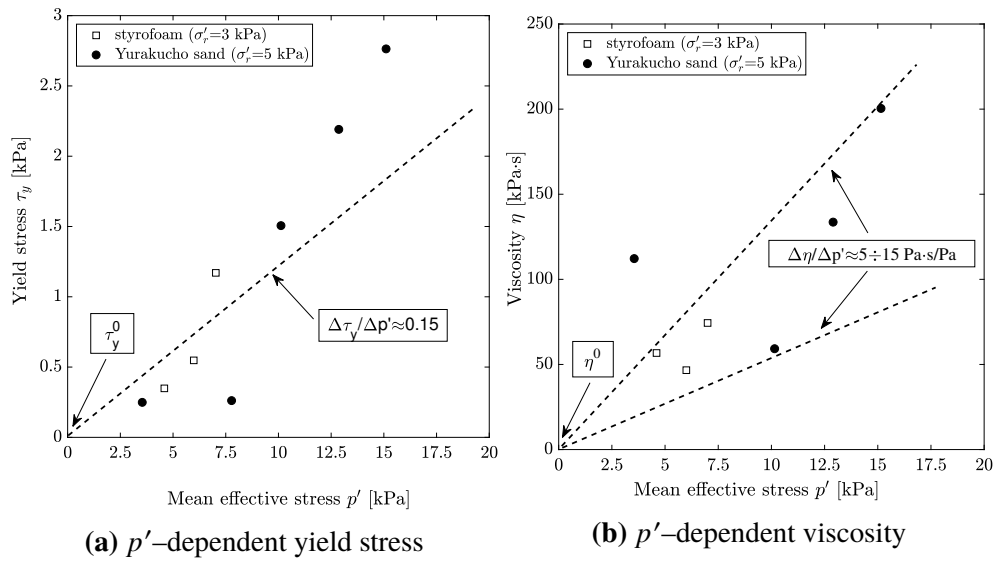
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830 imental data from Horsten (2016). Theoretical ‘no-soil equilib-  
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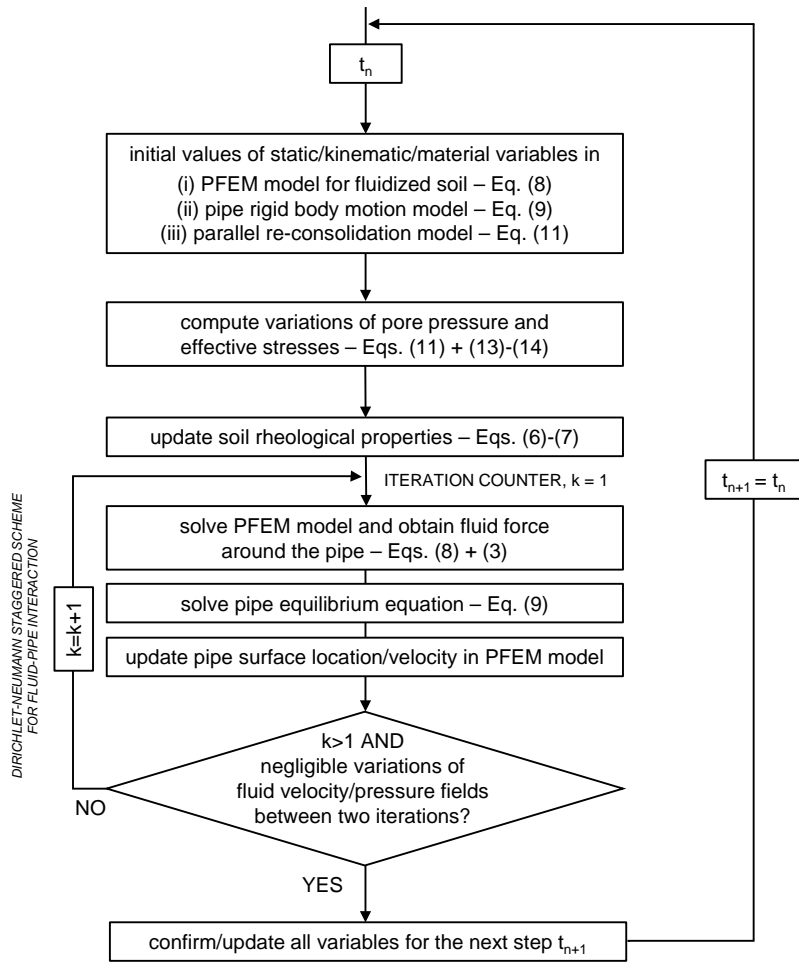
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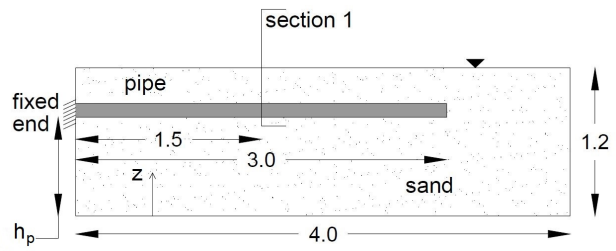


**Figure 1.** Dependence of Bingham parameters on mean effective stress, after Gallage et al. (2005) – pre-liquefaction relative density  $D_r \approx 30\%$ ,  $\sigma'_r$  stands for radial effective stress.

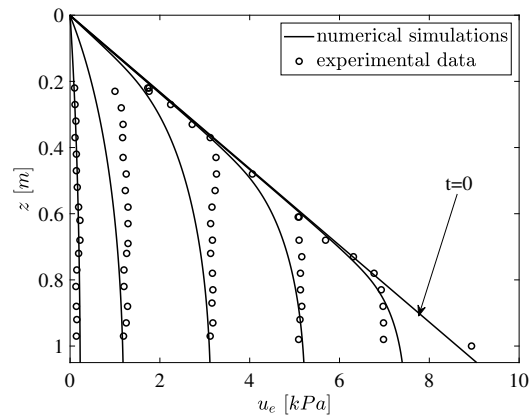




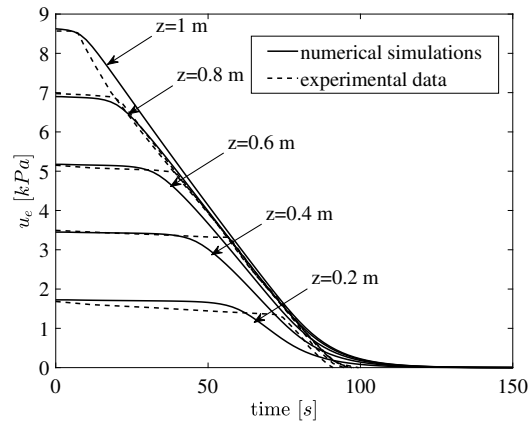
**Figure 2.** Solution of a single step in the proposed pipe-soil interaction algorithm.



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

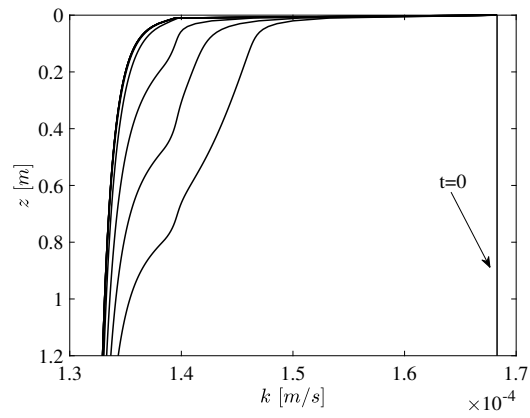


(a)  $u_e$  isochrones (plotted every 20 s)

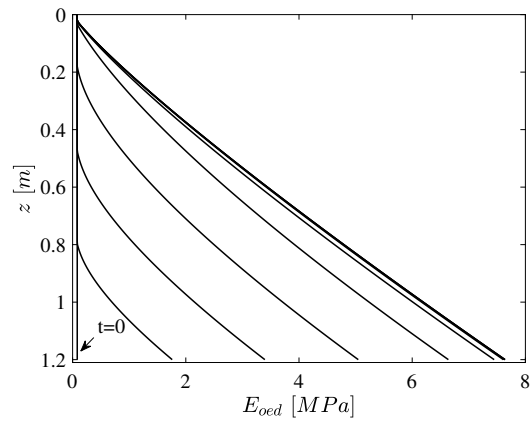


(b)  $u_e$  time profiles

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

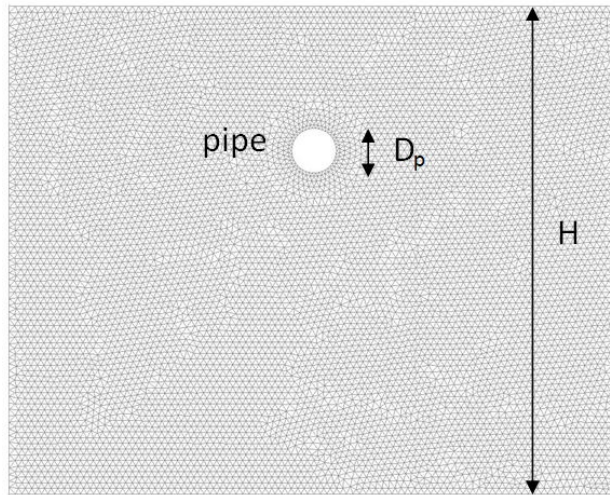


**(a)** permeability isochrones (plotted every 20 s)

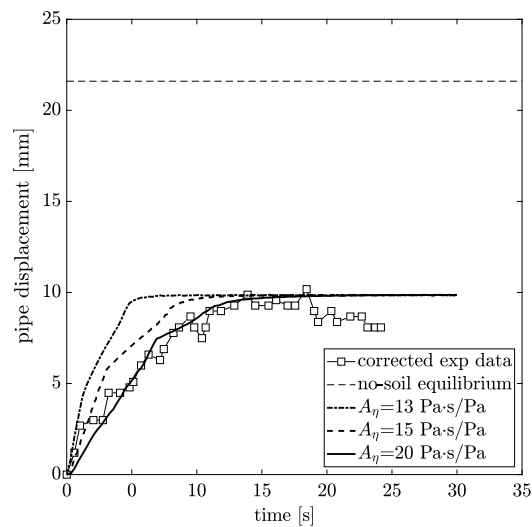


**(b)** oedometer stiffness isochrones (plotted every 20 s)

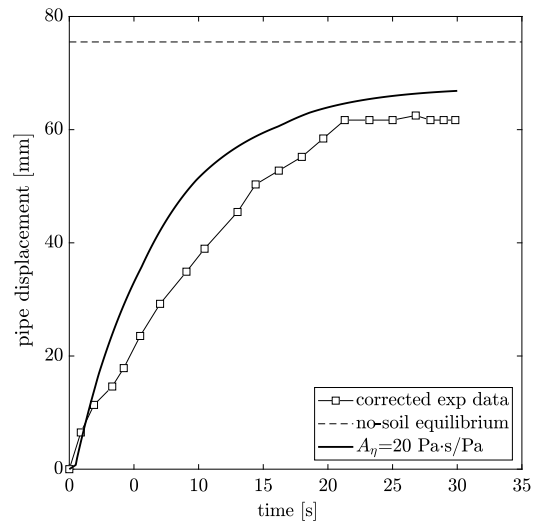
**Figure 5.** Isochrones of sand permeability and oedometer stiffness from the simulation of Horsten (2016)'s re-consolidation test on Sample #2.



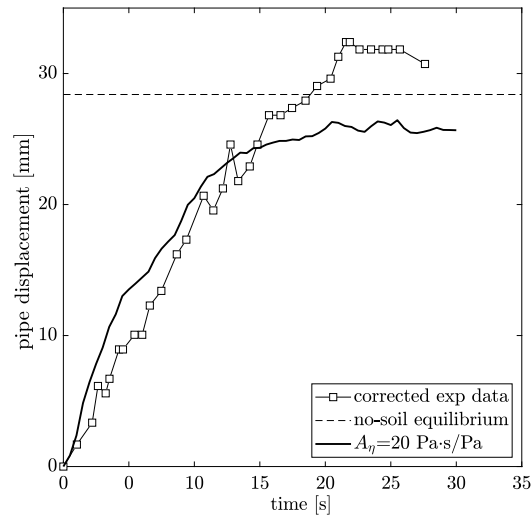
**Figure 6.** PFEM mesh for the simulation of pipe 1's flotation (Table 1).



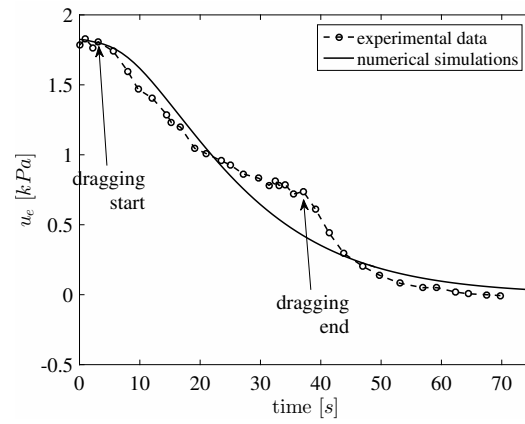
**Figure 7.** Pipe 3's flotation: comparison between CFD results and experimental data from Horsten (2016). Theoretical 'no-soil equilibrium' displacement: 21.7 mm.



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

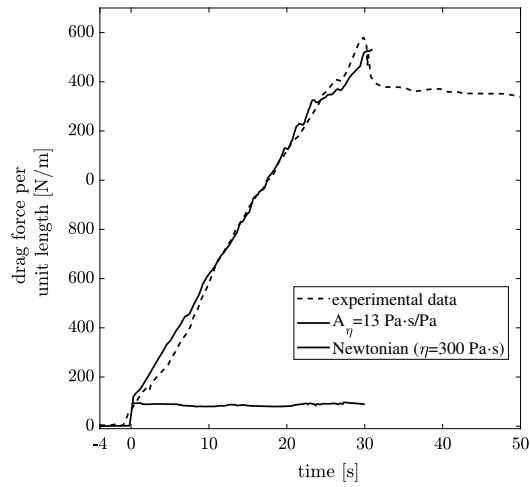


**Figure 9.** Pipe 2’s flotation: comparison between CFD results and experimental data from Horsten (2016). Theoretical ‘no-soil equilibrium’ displacement: 28.4 mm.

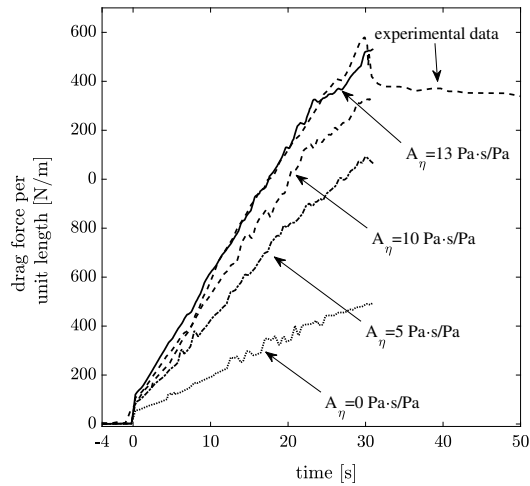


**Figure 10.** Simulation of  $u_e$  dissipation during pipe lateral dragging – data from Towhata et al. (1999).



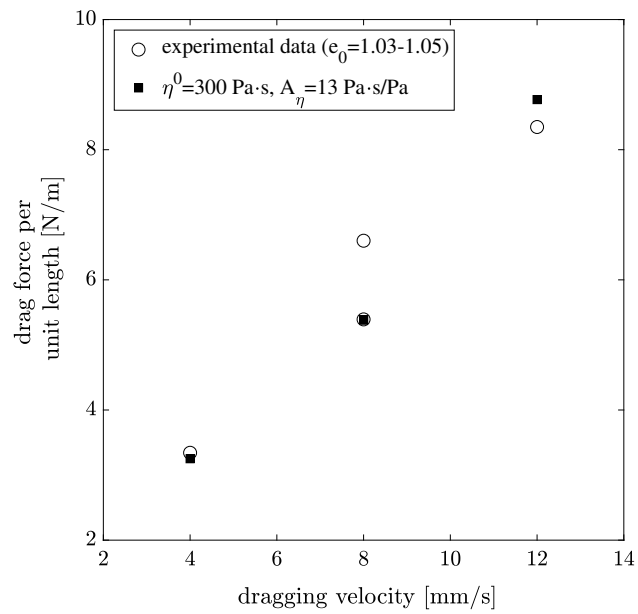


(a) calibration of the enhanced Bingham model

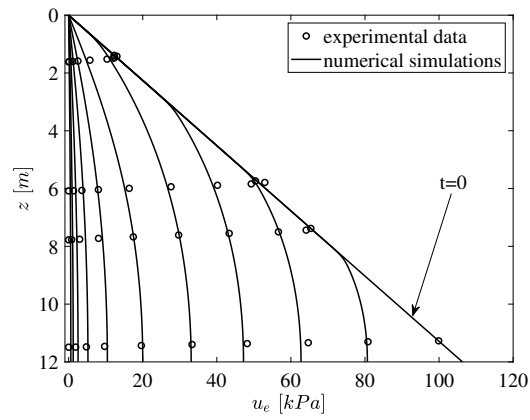


(b) influence of the  $A_\eta$  parameter

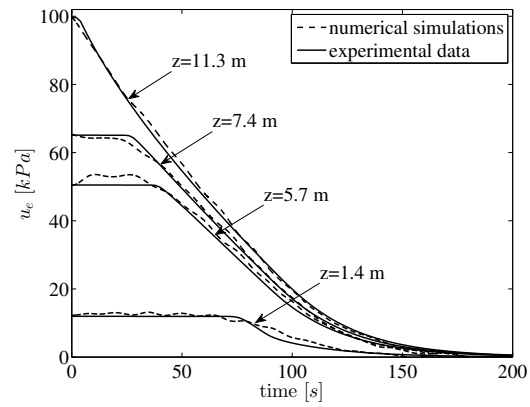
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**Figure 12.** Lateral pipe dragging: influence of pipe velocity on drag force prior to reconsolidation ( $r_u \approx 1$ ) – data from Towhata et al. (1999).

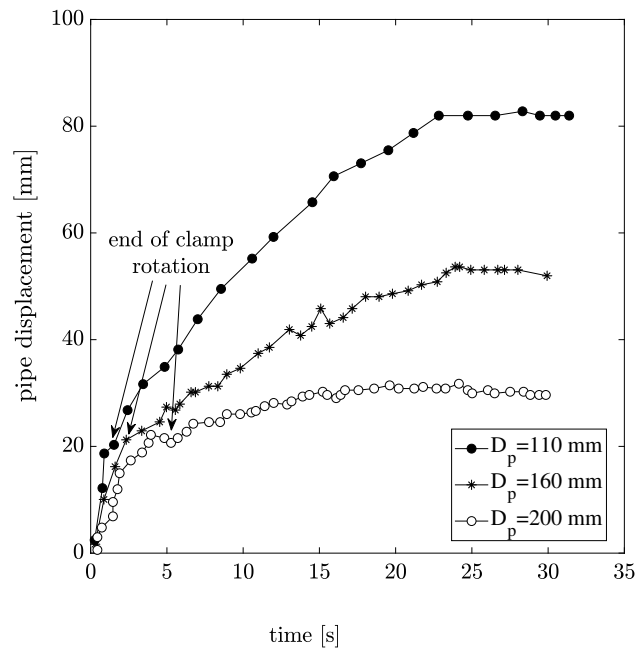


(a)  $u_e$  isochrones (plotted every 20 s)



(b)  $u_e$  time profiles

**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).