

Pipeline-Seabed Interaction

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1 Pipeline-Seabed Interaction (to be printed in ASCE, Waterway, Port,

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7

8 Abstract

9

A review of the existing research on the interaction between a pipeline and an erodible bed exposed to
waves and/or currents is presented. The review covers three topics, i.e., scour, liquefaction and lateral
stability of pipelines.

The basic mechanism that leads to scour in two-dimensional (2D) and 3D case is firstly described. The scour processes are deduced from small-scale laboratory experiments. The onset of scour from piping and the developing tunnel erosion are among the processes described. The lateral expansion of the scour hole along the pipe is described, also based primarily on small-scale laboratory experiments.. The state of the art of the mathematical/numerical modeling of the scour processes is presented. The associated self-burial of the pipe is described and compared to field observations. In addition to scour, liquefaction may also constitute a risk for pipeline stability. The cause of liquefaction
and the resulting consequence for pipeline stability in a natural environment are then discussed.

21 Finally, the lateral stability of pipelines placed on an erodible bed and exposed to waves is briefly described.

22

23 Introduction

24

25 Pipelines placed on an erodible seabed interact with the seabed, when exposed to sufficiently strong 26 waves/currents. Therefore measures are usually taken to secure the structural security of the pipeline. One 27 potential danger is the development of free spans below the pipe, which may cause the pipe to vibrate 28 from vortex shedding. To avoid this phenomenon, pipelines are often buried in the bed, either by self-burial 29 when this is likely to occur before any damage of the pipeline may occur, and otherwise by trenching or 30 jetting. Even when the pipe is only partially self-buried, a benefit is that the forces on the pipeline are 31 significantly reduced. In many areas pipelines are required to be fully buried to avoid damage by fishing 32 gears or anchors. However pipes designed to allow lateral motion to accommodate thermal expansion 33 should not be partially buried, which could prevent the lateral motion. A general introduction to subsea 34 pipeline safety can be found in Palmer and King (2004).

Scour around pipelines differs from the scour around most other marine structures such as platform legs,
wind turbine towers and breakwaters because of the interaction between the pipeline and the seabed. This
interaction arises from the flexibility of the pipeline, which causes it to bend thus changing position during
the developing scour process. The scour process continually adjusts to the changing geometry. This
dynamic evolution makes the topic fascinating to deal with, but also increases the complexity of an
accurate model or description of the phenomena.

When the pipe is laid on or embedded in the seabed, it changes the local flow pattern. The flow contracts and eddies are formed both up- and downstream from the pipe. The current-alone case is shown in Fig. 1. This change in flow pattern causes local increase in the sediment transport capability, which creates scour in that area. The change in flow pattern also causes a pressure difference between the up- and downstream part of the pipe, which causes seepage flow in the bed beneath the pipe, which may result in piping.

Most of the mechanisms for scour outlined below are based on small-scale lab experiments with pipe
diameters varying from 5-20 cm. Many of these tests have later been modelled either by simple heuristic
models or by numerical modeling tools. In recent years field data and large-scale model tests have become
available.

51 Whether small scale experiments and numerical modeling are useful in an engineering context can be 52 discussed: Drago et al. (2015) say "Theoretical modelling allows investigation in detail of the phenomenon 53 occurring close to the interface and provides explanations for causes and consequences of events. But very 54 often, owing to its complexity, it has no practical application in engineering. Usually, simplified semi-55 empirical interpretative models are applied. Semi-empirical models, having a strong feedback from 56 theoretical models, try to quantify and express in formulae the outcome of experimental tests." 57 This statement is certainly true, since most small scale experiments have scale effects, and an idealized 58 setup mainly uses non-cohesive sediment and an initial plane bed, and the incoming flow usually is 2D 59 without changing direction. For this reason, most of the studies reviewed herein must be considered as a 60 background for understanding the physics behind the scour processes.

61

62 **2D Scour**

64 Onset: Piping and Scour

65	Scour below pipelines can be split into a series of mechanisms of 2D as well as 3D in their character. These
66	mechanisms eventually lead to a certain self-burial of a pipeline originally laid on the surface of an erodible
67	seabed. Firstly, the 2D case will be evaluated, where a steady flow meets the pipeline perpendicularly, and
68	uniformity is assumed along the pipe. When not otherwise stated, the pipeline is assumed to have a
69	circular cross section with a diameter <i>D</i> , and the bed sediment is assumed to be non-cohesive.
70	When the pipe is placed <i>above the bed</i> (for instance when the pipe is unable to conform to the natural
71	bathymetry), there will generally be local erosion beneath the pipe caused by the flow contraction around
72	the pipe, causing an increase in the sediment transport capacity just beneath the pipe. This scour only is
73	significant for small initial gaps, typically less than half a diameter.
74	When the pipe is placed on or slightly embedded a certain distance <i>e</i> from the original plane bed due to its
75	submerged weight, piping may occur from the seepage flow in the sea bed below the pipe. This seepage
76	flow is caused by the pressure difference between the upstream stagnation pressure (point A in Fig. 1) and
77	the lower pressure in the leeside wake, point B in Fig. 1. In steady flow, the wake pressure is primarily
78	governed by (slightly lower than) the pressure in the separation point S (Fig. 1) because of the small
79	velocities in the wake as compared to the outer flow. In S, the pressure is low because the outside flow is
80	high (Bernoulli).

Piping failure below the pipe occurs when the hydraulic gradient i in the sediment exceeds a critical value, 82 the floatation gradient i_f , given by

 $i_f = (1-n)(s-1)$ (1)

84 where the right hand side in Eq. (1) represents the submerged weight of a unit volume of water-sediment 85 mixture, n = porosity and s =relative density of sediment to the fluid.

The initiation of the piping begins at the downstream end (point B in Fig. 1). The piping may be facilitated by *erosion* by the up- and downstream wake system around the pipe by digging away some sediment near the pipe, Mao (1986), Chiew (1990), Sumer and Fredsoe (1991). This may contribute to the onset of piping by shortening the streamline beneath the pipe, which increases the hydraulic gradient.

90 The scour process initiated by piping was studied experimentally by Chiew (1990). He created a static 91 pressure gradient using a pipe placed on/in a sandy bed in a box, divided into two equal chambers by 92 placing a watertight partition above the pipe center. In this manner, the water levels in the two chambers 93 could be different, and the pressure gradient below the pipe could be varied. Chiew also measured the 94 pressure gradient around the pipe, and confirmed Eq. (1). Sumer and Fredsoe (2001) performed similar 95 tests using the pipe itself as the partition wall. Both studies concluded that the scour played a minor role 96 for the onset. In the test by Sumer and Fredsoe (2001), the impact of the vortices for an embedment of 97 e/D=0.064 was measured at approximately 20-25%. It is not quite clear whether this is from local scour or 98 from fluctuations in the flow. As noted by Chiew (1990), there is another difference between static 99 experiments and real flow. In the latter, the pressure fluctuations in the downstream vortices result in 100 easier onset of piping.

101

102 Impact of Embedment

The onset of the scour beneath a pipeline strongly depends on the initial embedment *e* into the sea bed. An increase in embedment modifies the pressure gradient in the soil along the pipe periphery from A to B, (Fig. 1) in two ways; i.e. (1): the streamline in the soil becomes longer, thus reducing the gradient, and (2) the wake pressure is decreased, leading to an accompanying decrease in the gradient in the soil. The first mechanism is the most important for small embedment depths, while the latter becomes increasingly important with larger values, where the flow blocking is less, thereby decreasing the contraction flow.

109 Therefore, the pressure at the stagnation point S, and thereby in the wake, becomes less negative. For

these reasons, there is an upper limit for the onset of scour when the pipe is embedded in the bed. In the

111 Chiew (1990) experiments, no onset was observed when *e/D* exceeds 0.5.

Because the pressure gradient from A to B in a fully turbulent flow is proportional to the flow velocity U

squared, the onset criterion can be rewritten as follows (Sumer and Fredsoe, 2001) :

114
$$\left[\frac{U^2}{gD(1-n)(s-1)}\right]_{cr} \ge f(e/D)$$
(2)

where g = the acc. of gravity and f = a function, that depends primarily on the embedment depth. It also depends on the pipe Reynolds number and roughness, because both have impact on the wake pressure and thus influence the hydraulic gradient in the soil below the pipe. This relationship is shown in Fig. 2, where the dashed line represents the current-alone case.

In shallow waters, Chiew (1991a) additionally found the flow to change significantly with decreasing water
 depth at water depth below 6-8 *D*, which may increase the possibility for onset. This small depth-diameter
 ratio is most relevant in the fluvial environment.

122

123 Onset of Piping in Waves

124 In the wave case, the scour picture is similar to that in the current-alone case, with some modifications

125 because the flow attack is bi-directional. The onset is governed by the same piping mechanism as explained

above; however, as described by Sumer et al. (2001), the pressure gradient from A to B, Fig. 1, is a function

127 of the Keulegan-Carpenter number *KC* for waves. The *KC* is defined as follows

 $KC = 2\pi a / D$

where a = the orbital amplitude and D = the pipe diameter. Changes in the KC cause changes in the flow pattern, e.g. the time available for piping decreases as KC decreases, because the flow changes direction

(3)

131	with a higher frequency when KC decreases, whereby the piping process occurs more slow in waves than
132	in current. In addition, the pressure drop from A to B (Fig. 1) decreases as KC increases, because the
133	downstream wake is not fully developed at small KC , and flow separation occurs further downstream on
134	the pipe periphery than in steady current. As a result, the onset in waves for the same embedment occurs
135	at a lower near-bed velocity in waves than in a current. This phenomenon is most pronounced at small KC
136	(Sumer and Fredsoe, 2001).
137	Additionally, the potential loosening of the soil beneath the pipeline when subjected to the recurring
138	motion associated with the orbital flow may play a role for easier onset in waves.
120	The viewes plus surrent case. Zong at al. (2010) considered the enset of nining for the combined motion
139	
140	From experiments they observed that the critical velocity for the onset of piping was slightly larger in the
141	combined flow, and the maximum critical velocity occurs when the current velocity and the orbital motion
142	were of the same size.
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154 The flow picture changes during the tunnel erosion evolution from two different causes: the change in bed 155 morphology and the changes in flow regime.

156

157 Flow Pattern

158 Initially, the flow velocity just beneath the pipe is higher than the undisturbed flow because of flow 159 contraction. In a steady current, the enhancement of the flow at the pipe below the axis is approximately 160 the same as that at the top, namely around 1.5 times the undisturbed incoming flow velocity, as measured 161 by Fredsoe and Hansen (1987), and similarly observed by Chiew (1991a). This flow is governed by the 162 location of the separation points of the lee wake, which are located close to the divergence of the flow, i.e. 163 nearly below and above the axis of the pipe. In this initial part of the scour process, the flow is quasi-steady 164 as scour develops. However, when the gap reaches approximately 5-10% of the pipe diameter, the leeside 165 wake experiences vortex shedding and the flow becomes highly unsteady (Sumer et al., 1988b).

166

167 Bed Morphology

168 The bed development for the current-alone case is shown in Fig. 3. Initially the scour occurs primarily just 169 beneath the pipe. The eroded sediment is deposited as a berm (deposition ridge) downstream. During the 170 scour development, the berm moves further downstream and levels out. The deepest part of the scour 171 hole also moves downstream. The erosion of the downstream berm is significantly facilitated by the vortex 172 shedding behind the pipe because the vortices contains high-velocity peaks (the increase has been 173 measured to be more than a factor of three), which are able to sweep away significant amounts of 174 sediment deposited in the downstream berm. This activity extends the downstream scour profile. This 175 removal of the berm makes the profile more streamlined providing less flow resistance to the flow beneath 176 the pipe, leading also to a slightly increased scour development just beneath the pipe axis. The importance of the vortices has been recognized by Hulsbergen (1984), and elaborated in details by Sumer et al. 177 178 (1988b).

The final scour depth S in this 2D case is approximately 0.3-1.0 times the pipe diameter, with a high degree of scatter, as shown in Fig. 4. In the figure, θ is the Shields parameter, which describes the sediment mobility, quantified by the dimensionless bed shear stress as follows:

182

183
$$\theta = \tau_b / (\rho g d (s-1)) \tag{4}$$

184 where τ_b = the bed shear stress, d = the mean grain diameter and ρ = the fluid density. In Fig. 4, Re is 185 the pipe Reynolds number, defined as

186
$$\operatorname{Re} = UD / v \tag{5}$$

187 where ν is the kinematic viscosity of the fluid. Chiew (1991b) examined a large amount of data regarding 188 maximum scour depth beneath pipelines. Chiew' paper provides a good review of earlier experiments and predictive formulae describing scour depth beneath pipelines exposed to a current. The data include pipe 189 190 diameters from 25 to 500-mm. He observed largest depth in the transition region from clear water to live 191 bed scour. Clear water scour defines the case where there is no general mobility of sediment on the seabed 192 from the hydrodynamic loading except very near the pipe, where the flow is enhanced. This transition usually occurs at $\theta = \theta_c \simeq 0.045 - 0.050$ for sand. In the clear water case, there is no influx of sediment to 193 the scour hole. Chiew (1991b) relates the larger scour depth in this specific region to this missing supply, 194 where the measured scour actually is larger than what a number of predictive formulae suggest. 195

196

197 Waves

In the case of waves, the vortex shedding occurs on both sides of the pipe from the bi-directional near-bed oscillatory flow. This makes the profile even flatter and more streamlined on both sides of the pipe as compared to the current-alone case (Sumer and Fredsoe , 1990). The extension of the vortex street L, as shown Fig. 5, primarily depends on the length of the orbital stroke or, in dimensionless form on *KC* , Eq. (3)

202 . The width of the scour hole increases also and becomes more streamlined with increasing KC. This 203 explains the increase in scour depths with increasing KC as shown in Fig. 6. At small values of KC, the 204 scour depth is smaller than in the current-alone case because the extension of the wake is smaller than in 205 the current case. At larger KC (higher than approximately 50), it becomes larger than in the current-alone 206 case because of the flattening of the profile.

207

208 Waves and Current Combined

In the combined wave-current case, there is a transition from the current-alone case, with scour depth just less than one diameter, to the wave dominated case, where the scour depth is *KC* -dependent. The relative strength between current- and wave-induced flow velocities is given by the parameter

212
$$\alpha_r = \frac{U_c}{U_c + U_m}$$
(6)

where U_c = a near-bed current velocity, usually the current velocity half a diameter away from the 213 undisturbed seabed, and $U_{\scriptscriptstyle m}$ is the maximum orbital velocity at the bed. Drago et al. (2015) suggested, that 214 when α_r is less than 0.8, the scour is wave-dominated, and when α_r is larger than 0.8, it is current-215 216 dominated. The value 0.8 corresponds to the limit, where the flow shifts from one-directional to bi-217 directional flow. Sumer and Fredsoe (1996) found a smoother and earlier transition, as shown in Fig.7. This 218 result relates to the wake behavior around the pipe in oscillatory flow. In this flow, the lee wake vortices 219 can be pushed back to the opposite side by the pressure gradient in absence of bi-directional flow because 220 the wake has smaller velocities than the outside flow, and are thus easier to push back than the outer flow 221 (Jacobsen et al. 1984).

222

223 The Time Scale

The time scale T for the tunnel erosion is a measure for how fast equilibrium develops in the scoured profile. This time scale is used to determine of the potential scour depth in a real environment, e.g. in a

single storm event. From laboratory experiments, Fredsoe et al. (1992) developed a non-dimensional time
 T* expressed by

228

229
$$T^* = T\sqrt{g(s-1)d^3} / D^2 = 0.02\theta^{-1.7}$$
 (7)

230

According to Eq. (7), the scour process speeds up (the timescale decreases) with increasing bed shear stress θ , while no notable variation in *KC* is detected. In the estimate of *T*, an exponential variation from the initially large scour rate towards equilibrium is assumed, i.e. the estimate is an integrated approach. Dogan and Arisoy (2015) performed additional experiments, finding a variation with *KC* as well. In their study, the timescale is suggested to increase as *KC* increases. In their analysis, they distinguish between the live-bed scour and the clear-water cases suggesting different expressions for the timescale for the two different cases.

238

239 Backfill

For the current-alone case, tunnel erosion is nearly independent of the current strength, while the scour depth in oscillatory flow increases as *KC* increases. Therefore, an equilibrium scour profile will change its equilibrium depth when the height and/or wave period change. For example, during the end of a storm, the calmer waves may partially backfill the scoured profile because of the drop in *KC*. This backfill occurs at a much slower rate than the erosion because it occurs at a lower Shields number θ .

Fredsoe et al. (1992) experimentally studied the development in the scour profiles by changing the wave climate from one initial regular climate with specific values of *KC* and θ_i to another final one *KC*_f and θ_f . They found that in such transitional situations, the new equilibrium scour depth is determined by the new *KC* -value equal *KC*_f.

250 Time varying flow conditions

251 Zhang et al. (2016) performed experiments with oscillatory flow combined with a time varying current in 252 their new large scale O-tube facility (An et al., 2013). Two types of test were done; i.e. a time-stepping flow 253 condition and a uniformly increasing/decreasing flow condition. The oscillatory part of the motion was kept 254 constant. In the time-stepping approach, a steady flow (constant flow rate) scoured in 0.24 mm sand until 255 equilibrium was reached, where after the flow rate was changed to another either higher (causing erosion) 256 or lower flow rate (causing backfill). Zhang et al. found similarly to Fredsoe et al. (1992) that the final scour 257 profile is determined by the final flow conditions. Also they observed that the time scale for backfilling is 258 much larger than the time scale for erosion. They constructed a simple time-stepping model based on 259 knowledge on the final scour depth combined with a time-scale model. The uniformly increasing flow 260 conditions were performed with a 50 mm pipe placed on 24 mm sand, and simulates the conditions during 261 the ramp-up phase of a storm. Zhang et al. demonstrated that for this case, the developing scour can be 262 predicted by integrating the time-stepping model, and dividing the total time period for the scour development into small time steps and calculating the scour development for each time step by the above 263 264 mentioned model.

265

266 Irregular Waves

Kiziloz et al. (2013) experimentally studied the tunnel scour under irregular waves, and by comparing with
regular wave tests suggested which height-period wave parameters (in the form of Significant or Root
Mean Square waves) should be applied to get the best agreement between regular and irregular waves.
Myrhaug and Rue (2003) performed a similar exercise numerically, based on the scour formulation by
Sumer and Fredsoe (1990).

272

273 Mathematical/Numerical Modeling of the 2D Scour Development

274

In this section, a number of examples on the modeling of onset and evolution in the 2D scour profile aredescribed.

- 277
- 278 Onset

279 Zang et al. (2009) modeled scour onset by applying a k- ω turbulence closure. They obtained the pressure 280 coefficient distribution along the bed, which is needed to study the piping process. The calculations 281 included different embedment depth. They also included a discussion of the changes in the pipe Reynolds 282 number, the impact of the incoming boundary layer thickness and the water depth. They also applied the 283 numerical model to the wave case. The seepage flow from the pressure gradient was investigated while 284 contribution from scour near the pipe still remains to be investigated. Gao and Luo (2010) performed a 285 similar study for the current-alone case using a numerical finite element method (FEM) for the flow above 286 and below the pipe. Similar to Zang et al. they did not calculate the scour in the seabed either. They 287 justified their omission by observations, which indicate that the initial scour was not near the pipe.

288

289 Tunnel erosion

290 Current-alone. Fredsoe and Hansen (1987) demonstrated that potential theory can predict the final gap 291 beneath the pipe quite well. They modified the potential flow description by adding a vortex tube to ensure 292 a more correct description of the flow beneath the pipe because the potential theory predicts much larger 293 flow velocities beneath than above a pipe located near a bed. As explained in the previous section "tunnel 294 erosion", the enhancement of the flow at the bottom of the pipe is similar to that at the top (see also 295 Chiew, 1991a). The model by Hansen and Fredsoe (1987) describes the sediment transport as bed load and 296 followed the evolution to equilibrium, where the near-bed flow velocities beneath the pipe decrease to be 297 equal to the far field flow velocity at the bed. Their model predicts the scour depth beneath the pipe as a 298 function of the initial embedment, in reasonable agreement with measurements. Downstream of the pipe,

299 the evolution in the bed profile becomes too complicated for the simple model to obtain realistic profiles of 300 the scoured bed.

301 Sumer et al. (1988b) applied the discrete vortex method to focus on the impact of the downstream vortices

in conjunction with scour development. They showed that the peaks in the bed shear stresses caused by

the shed vortices were the primary agents in the lee-wake erosion.

Van Beek and Wind (1990) modeled the tunnel scour applying a k- ε turbulence closure. They described the sediment transport applying the van Rijn (1982) sediment transport formulation. They achieved reasonable agreement between predictions and the developing 2D scour profile beneath the pipe, except for the time scale, which was predicted smaller than in their experiments.

Brørs (1999) also applied a k- *ɛ* turbulence closure, and included density effects in the momentum and
turbulence equations. Whether the latter is of importance is not clear from the paper. He described the
suspended sediment using a deposition and erosion formulation, agreeing well with the measured
developing scour profile in the current-alone case.

Li and Cheng (1999) applied a finite element solution of the potential flow to describe the scour evolution
with a detailed bed-load sediment transport description including bed slope effects.

Liang et al. (2005b) calculated scour in the clear-water case and in the live bed case, applying a detailed sediment description divided into bed load and suspended load. They introduced a sand slide model to smooth out numerical irregularities, agreeing well with Mao's (1986) measurement, also regarding the timescale.

Oscillatory flow. Liang and Cheng (2005c) extended their current-alone studies to include the oscillatory case. They investigated the phase-resolved flow and the residual (phase averaged) flow, also known as the streaming. The streaming occurs because the pulsating flow beneath the pipe only is fully symmetric just below the center of the pipe. Away from this location, boundary layer effects in combination with wake provide asymmetry to the near-bed flow: the flow is directed away from the pipe. They demonstrated that the streaming is an important feature for transporting the sediment away from the scoured profile. Their

result agrees reasonable well with the Sumer and Fredsoe (1990) experiments, but no systematic variation
in scour depth with *KC* or sediment mobility was given.

Kazeminezhad et al. (2012) used a two-phase simulation to describe the tunnel erosion in oscillatory flow,
wherein they described the formation of the scour profile and the two end-mounds around the pipe in
oscillatory flow.

Fuhrman et al. (2014) also studied the tunnel erosion in oscillatory flow, using the k- ω model from the

330 open source code OpenFOAM[®]. They succeeded in predicting the increase in maximum depth with 331 increasing KC as observed from laboratory experiments, cf. Fig. 7. They demonstrated that a detailed 332 description of the sediment transport modes is essential to calculate the shape of the scoured hole, i.e. 333 without including the suspended sediment, the calculations predicted a bump just beneath the pipe, 334 because the bed shear stresses opposite to the near-bed streaming pattern are directed towards the center 335 of the scour hole. This bump does not appear when suspended sediment transport is included, because of 336 the time-averaged streaming pattern in the scour hole, which is directed away from the hole. 337 Waves plus current. Larsen et al. (2016) applied the same modeling tool as Fuhrman et al. (2014) to 338 investigate the scour for a wide range of combined KC and relative current strengths. Regarding the 339 equilibrium scour depth, they obtained similar trends in the combined motion as depicted in Fig. 7. Larsen 340 et al. also suggested an expression for the timescale in the combined flow, based on a large number of

341 model results.

329

Backfilling. The backfill process, from the change in wave impact perpendicular to the pipe, was studied numerically by Fuhrman et al. (2014). They found, in agreement with the experiments, that by switching from one KC to another, the scour profile adapted to the new value of KC. Their numerical analysis also agreed well with the experiments regarding the transitional period (the time scale) for the adaptation. *Refined flow modeling.* Whether even more detailed flow models provide improvements in the description of the scour is an open question. Liang and Cheng (2005a) refined the modeling by applying k- ε , k- ω and Smagorinsky's subgrid scale (SGS) turbulence models to examine the flow around a circular pipe placed a

small distance above a plane wall. They concluded that the SGS model performed best; however, scour
calculations were not included in any of the models. A more detailed description of the turbulent structures
in the wake is yet to be investigated.

352 Refined sediment transport modeling. In the above-mentioned numerical models, different types of 353 common sediment transport models or the Eulerian two-phase model have been applied. Because of the 354 downstream vortex shedding with occasional peaks in the bed shear stress, a time-resolved sediment 355 transport model is needed to describe the time scale and the final scour depths. Additionally, it is important 356 to distinguish between the bed load (with a slope correction term) and the suspended load. In the leeside 357 wake, next to vortex shedding, the turbulence in the wake is fluctuating; therefore, its effect on sediment 358 transport should be incorporated in the model. Mattioli et al. (2012) performed a detailed particle image 359 velocimetry (PIV) study of the turbulent structures in the wake around a pipeline exposed to waves. They 360 also studied the transportation patterns of suspended sediment further away from the pipe.

361

362 Scale Effects

363

364 Small-scale experiments have limited values because the Re is usually much higher in the prototype than in 365 the experiments. Similar to forces on pipelines, the scour patterns are sensitive to Re, pipe roughness and 366 sediment size to pipe diameter ratio. As an example, Sumer et al. (2001) examined the change in flow 367 regime from subcritical to supercritical regime in the flow around a pipe (corresponding to transition from 368 laminar to turbulent boundary layer separation in the wake, Sumer and Fredsoe (2002)) by comparing the 369 onset of scour beneath a smooth pipe and an artificially rough surface (glued .3-mm plastic particles). The 370 limiting value for onset changes by more than 60 %, requiring much higher flow velocities for the onset of 371 scour for a .1D burial depth. This is because, in this case, the pressure gradient from A to B drops because 372 of the flow transition.

Zang et al. (2009) numerically studied changes in Re, but only in the sub-critical regime, where the flow
changes with Re is limited.

Liang et al. (2005d) performed a numerical study of the Re dependence on tunnel scour using a k-ω model
with Re-dependence in the dissipation and production terms. In their prototype model results in the
supercritical range, they reported far less vortex shedding behind the pipe with corresponding smaller
predicted scour depths, approximately 10-15%. Whether the vortex shedding becomes suppressed in the
prototype still awaits experimental confirmation.

380

381 **3D Development of the Scour Profile: Shoulders and Free Spans**

382

Tunnel scour cannot be uniformly distributed along a pipeline, because the pipe needs seabed support.
Instead, in the initial phases, scour occurs occasionally along the pipe, and then the scour holes expand
along the pipe.

386 During this process, the pipe is supported on smaller and smaller stretches of soil, the span shoulders, as 387 shown in Fig 8. When the span shoulders become sufficiently short, the weight of the pipe may cause a 388 geotechnical bearing failure of the shoulder, causing larger embedment of the pipe at the shoulder. Draper 389 et al. (2015) observed this experimentally. In their experiments, the sinking at the span shoulders occurred 390 in small jumps in the vertical displacement, indicating a number of collapses of the span shoulders rather 391 than a continuous bearing failure. During this process, the pipe increases its far-field embedment (defined 392 as the local mean vertical deflection from the undisturbed seabed) as compared to the undisturbed bed by 393 sinking down in the remaining part of the shoulders.

394

395 Sinking and sagging

396	Leckie et al. (2015) and Draper et al. (2015) discuss the impact of the length of the scour hole on the
397	pipeline deformation. When the hole is long, the pipe locally sags down into the hole causing variations
398	along the pipe, as sketched in Fig.9, left column. When, however, the lengths of the holes are shorter from
399	the imperfection points below the pipe, the pipe sinks more uniformly.
400	To accurately describe the span development requires the following subtopics
401	• Will the scour depth beneath the pipe increase when the pipe sags down into the hole?
402	• What happens when the pipe touches the bed in the free span?
403	• How fast do the scour holes expand along the pipe?
404	• What will the final length of the spans and shoulders be, i.e., what are the limiting mechanisms?
405	Even through the scour pattern contain 3D features, much can be learned from 2D experiments and
406	modeling. The first question in the list above can be investigated in 2D as described in the following.
407	
408	Sagging Experiments
409	The first question in the list above was studied experimentally in a 2D setup by Fredsoe et al. (1988), who
410	allowed a 10-cm pipe to sag in a controlled manner vertically into a developing scour hole to mimic the
411	vertical movement of the pipe at the middle of a long span as it moves down into the scoured hole, finally
412	touching the bed. A number of runs with different sagging velocities were performed to simulate different

413 pipeline bending stiffness. In each individual run, the sagging velocity was constant until the pipe touched

the bed. Little difference was observed between the sagging experiments and the fixed pipe tests, i.e. the depth just beneath the pipe increased by 10-20%, while the scour further downstream became slightly smaller in the sagging-pipe case. Draper et al. (2015) performed similar tests with a larger (196-mm) pipeline. Both test series mentioned above were current-alone cases. In each test, Draper et al. (2015) varied the sagging velocity with time to simulate the lateral expansion of the scour hole. These tests indicate that the scour depths increase moderately from the sagging compared to no movement of the 420 pipe. They also get larger scour holes beneath a slowly sagging pipe than a rapidly sagging one, suggesting 421 that a flexible pipe sinks less than a stiffer one. The most flexible pipe studied only reached a depth of 422 0.58D into the bed, while the stiffest reached a depth of 1.28D into the bed. The reason for this is that the 423 stiffer pipe reaches the bed more slowly; therefore, there is more time available to erode below the pipe. A 424 direct comparison between Fredsoe et al. (1988) and Draper et al. (2015) is not possible because the two 425 series of experiments use different sediment mobility parameters θ .

Both afore-mentioned studies included the scour beneath the pipe for different values of far-field
embedment. It has been observed, that the scour depth below the pipe nearly doubles from around .8D for
zero far field embedment, to 1.4D for an embedment of 0.9D, when the pipe sags sufficiently slowly into
the scour hole. The two studies are here in line. Draper et al also confirmed (approximately) the suggested
time scale for the scour development as suggested by Fredsoe et al. (1992).

Numerical Modeling: Cheng and Li (2003) modeled the scour development under a sagging pipe by
calculating the Navier-Stoke (NS) equations with a Smagorinsky subgrid model. Their model predicted that
the scour beneath increased with a decrease in the sagging velocity and could reach more than one pipe
diameter for very small sagging velocities. Zhao and Fernando (2008) modeled the scour around a sagging
pipeline with an Eulerian two-phase model to describe the sediment transport, and a k-ε model for the fluid
flow. Both numerical studies agreed nicely with the measured scour profiles (Fredsoe et al., 1988).

437

438 Sinking Experiments

Draper et al. (2015) performed sinking experiments for the current-alone case and for the combined wavecurrent case. In the latter, the wave impact was increased gradually, to resemble a storm. In the tests, the pipe sank from the scour beneath the pipe. The scour began at the edges of the pipeline placed in a 1-m wide tunnel. When the wave impact was increased rapidly, the pipe began to move laterally because no sinking of the pipe had occurred. When the wave impact increased slowly, the scour initiated at the edges

444	had sufficient time to develop towards the middle of the flume, and sinking occurred from the weight of
445	the pipe. In this case, the pipe remained laterally stable until it was completely buried.

446

447 Backfilling and Self-Burial

448 A scoured hole around a pipeline may later undergo sediment backfilling. One reason for this backfilling is 449 the *touchdown* of the pipe into the scoured hole, causing the gap beneath to disappear. This changes the 450 flow pattern because all the near pipe flow has to pass above the pipe and backfill commences, as observed 451 experimentally (Sumer and Fredsoe, 2002) and sketched in the right column of Fig. 9. Thereby the pipe 452 becomes (partial) self-buried. Draper et al. (2015) observed experimentally, that a flexible a pipe (with a 453 higher sagging velocity) will self-bury less than a stiffer one, varying from 0.58D to 1.28D in their 454 experiments. Liang et al. (2005e) modeled the backfill of the scour hole after the pipe hits the bottom of 455 the scour hole.

456

457 **Development of the Scour Holes along the Pipe**

458

The development of the scour holes along the pipe has an impact on the sinking velocity of the pipe into the bed; i.e. when the holes spread faster, the pipe sinks faster. Several researchers have studied the migration rate without any definitive answer of the problem.

Hansen et al. (1991) performed current-alone tests, where they studied the transverse development of an initial small (artificially generated) scour hole for different initial embedment depths. The pipe was allowed to sink into the bed in the vertical direction only. They observed that the free-span development decreased with increased embedment at the shoulders. This indicates that a mechanism to stop further lateral expansion would be to decrease the length of the span shoulders such that the weight of the pipe causes geotechnical failure of the shoulders and thereby increase the embedment. Hansen et al. performed

experiments with two different pipe weights at the same initial embedment. They observed that the 468 469 heaviest pipe sank, while the lighter one did not. This is a strong indication that further sinking at the 470 shoulders is from geotechnical failure rather than to scour. Hansen et al. (1991) also investigated scour hole 471 development from an obliquely incoming current and found an increase in the propagation rate at the 472 most-exposed downstream corner, and a similar reduction in the less-exposed upstream corner. 473 Sumer and Fredsoe (1994) measured the span development in a 2-m wide current flume, where a small gap 474 between the pipe and the sidewalls ensured that the span development would begin at the sides, and 475 migrate inwards from there. In such an experiment only one shoulder appears. The pipe was allowed to 476 move freely in the vertical direction. The pipe stopped sinking at approximately 0.5-0.8 D below the initial 477 bed level. Such an experiment provides information on how large fraction along the pipe must be 478 constituted by the shoulders, while it does not give any information regarding the absolute value of the 479 shoulder or span length.

480 Cheng et al. (2009) also studied the 3D development subject to a steady current and in larger details. Just as Hansen et al. (1991), they initiated the scour process by introducing a small gap in the middle of the 481 482 pipeline, and, again similar to Hansen et al., the pipe was partially embedded in the sand across the flume. 483 Cheng et al. observed two phases in the scour development, a higher primary rate, and a secondary slower 484 rate occurring later. Their explanation for these two different rates was that initially the pressure gradient 485 across the pipeline was large, causing strongly 3D flows in the developing scour hole, including a large 486 amplification in the near-corner bed shear stress. At later stages, the pressure gradient drops as the scour 487 hole widens, and the flow beneath the pipe becomes more 2D in its structure. For a small embedment, only 488 primary propagations were observed. Similar to Hansen et al. (1991), Cheng et al. (2009) found a decrease 489 in the scour propagation rate as the initial embedment was increased. Finally, Cheng et al. also provided 490 information on the shape of the scour hole profiles near the shoulder, i.e. they observed the slope of the 491 hole at the shoulders to be fairly constant through each test and close to the angle of natural repose.

492 Wu and Chiew (2012, 2015) performed current-alone span development experiments in the clear-water 493 regime. Like Cheng et al. (2009), they identified a rapid and a slack phase in the development. In their 2012-494 study, they identified two driving environmental forces based on dimensional considerations, namely the 495 Shields parameter θ and the pipe Froude number F defined as follows

496

$$F = U / \sqrt{gD} \tag{8}$$

498

Additionally, two stabilizing forces was identified; i.e. the pipeline embedment and the water depth-to pipeline ratio. The dimensionless spreading velocity during the rapid development was observed to depend stronger with F than with θ , but their experiments are restricted to small values of θ . Just as Cheng et al. (2009), they found spreading velocity to decrease with increasing embedment, but a direct comparison with the data by Cheng et al. (2009) was not possible, because Cheng et al.'s was performed with live-bed conditions. Wu and Chiew (2012) observed the spreading velocity to increase with decreasing water depth at water depth below 6-8.

506 Wu and Chiew (2013) performed a detailed study of the flow velocities in a span. They observed that strong 507 velocities still occurred at the shoulders, while in the middle of the span, the velocities were reduced 508 significantly. This must be associated with the strong 3D flow structure at the edge of the shoulder. 509 Wu and Chiew (2015) complemented their earlier scour measurements with measurements of the pressure 510 differential Δp between the up- and downstream part of the pipe. They measured at 4 different stations 511 spaced 20-cm between each station. They compared the temporal change in Δp with the properties of the free span. They observed Δp drops at the corresponding location, where the span shoulder breaches 512 because expansion of the span. They also observed that the rapid phase is associated with a large Δp . 513 They related the initial rapid phase to the onset of scour development (piping). In the later slack phase, 514 515 they observed similarities to the piping phase very near the span shoulder. They measured Δp near the

516 span shoulders to support their statement. At the span shoulder, they measured a local enhancement of 517 the bed shear stress next to the differential pressure between the up- and downstream side of the pipe. 518 They concluded that both of these factors played a role in increasing the local sediment transport capacity, 519 and are thus responsible for the propagation of the scour hole along the length of the pipe. 520 Waves. Cheng et al. (2014) extended their experimental investigations to include waves next to the current. 521 They found that the scour propagation rate c decreases nearly linearly with an increase in the initial 522 embedment depth at all wave environments studied. They related this to the reduction in the gap flow 523 near the shoulders with increasing embedment. Further, they found c to increase as KC increases, 524 similarly to the increase in 2D scour with KC as seen from Fig. 6. In the wave case with oblique attack, 525 Cheng et al. (2014) found no distinction in the scour propagation rate c at the two shoulders, in 526 contradiction to the current case, where the propagation was largest at the downstream end. This is 527 explained by the bi-directional orbital velocities in the waves.

528

529 **Conceptual Modeling of the Spreading along the Pipe**

Bernetti et al. (1990) were the first to propose a simple conceptual model for the lateral spreading. Their model assumed a constant slope equal to the natural repose of the sediment, and related the scour to the rate of sediment transport. The assumption regarding the steep slope is confirmed by observations of the scour profile near the span shoulders by Cheng et al. (2009). Bernetti et al.'s model did not describe any slowdown of the spreading from pipeline caused by the embedment at the shoulders.

Hansen et al. (1991) elaborated further on a simple spreading model, in which they constructed a sediment balance at the corner of the scour hole adjacent to the span shoulder. They assumed the near-corner bed shear stress is amplified by a factor α , and that this amplification occurs at a distance βD along the pipe at the corner. β is of order O(1), while α was determined from their experiments to fit the spreading rate as best as possible using a bed-load sediment transport formulation. α decreases from approximately 2.5 for no embedment to 1.0 (no amplification) for e/D approaching 0.65. Hansen et al.'s model also included the general case with obliquely incoming flow: by splitting up the incoming sediment transport into a
normal and parallel component, they found as later observed by Cheng et al. (2009), that the downstream
corner moves faster than the upstream corner.

544 Chen and Cheng (2004) solved the 3D NS equations, including a Smagorinsky subgrid-scale model (to model 545 the eddy viscosity), and were able to calculate the complicated near-shoulder flow pattern, but without 546 experimental verification. They found bed shear stress amplifications primarily in two zones near the 547 shoulder, one just beneath the pipe and another upstream of the shoulder near the upper edge of the 548 slope.

In the model suggested by Cheng et al. (2009), they considered the propagation to be primarily caused by tunnel erosion of the more shallow parts of the scoured hole near the corner, where the slope was assumed equal to the angle of repose. The scour rate was estimated using the timescale for tunnel scour suggested by Fredsoe et al. (1991), and given by Eq.6. They predicted the primary and the secondary propagation rates, agreeing well with measurements in terms of the variation with different embedment depths.

555 Cheng et al. (2014) finally extended their predictive model for scour propagation for the current-alone case 556 (2009) to include the wave plus current case, following the same procedure as in their 2009 paper. The 2D 557 equilibrium scour depth used in their model was adapted using a formulation by Sumer and Fredsoe (1996). 558 Integrated approach. Fredsoe et al. (1988) suggested in a more integrated approach that the scour hole 559 stopped developing laterally when the sinking pipe touched the bottom of the scoured hole. This very 560 heuristic model makes sense because as soon as the pipe touches the bed, backfilling might commence, as sketched in Fig. 9, right column. This prevents the hole from further expansion, and the former long span is 561 562 transformed into two smaller free spans on each side of the touchdown point. This model therefore 563 estimates the maximum length of free spans. The model predicted proportionality between the stiffness 564 length of the pipe l_s and the maximum span length of the scoured hole. The stiffness length is the length

required for a pipeline to deflect one diameter from its own (submerged) weight W_s (per unit length). The stiffness length is as follows

567 $l_s \sim (128 DEI / W_s)^{0.25}$ (9)

- 568 where EI is the bending stiffness of the pipe.
- 569

570 Field Measurements

571 Leckie et al. (2015) presented 7 years field measurements along a 22.9-km long distance of a subsea 572 pipeline located on Australia's North West Shelf. They applied sonar profilers and video recordings, and 573 their study demonstrates how difficult it is to transfer knowledge based on experiments/modeling to a real 574 environment. The upper 1 m of the soil consists of drained sand along the full length of the pipeline and the 575 flow is dominated by bidirectional currents. The pipe has an outer diameter of 36-cm and the stiffness 576 length is calculated to be 44-m by applying Eq. 8. Significant self-burial of the pipe was observed. About 577 50% of the pipe was in span just after installation, and nearly all spans were shorter than 10-m. The span 578 value decreased by approximately 60% after 7 years, while the length of the spans was consistently about 579 6-m. This value corresponds to half the distance between the field joints along the pipe. No spans reach 580 more than 40 m. The large number of closely spaced spans indicates regular and frequent initiation created by the joints of the pipe, which are spaced 12.2-m, c.f. Leckie et al. (2015). The long spans reach a 581 582 length equal to the stiffness length (44 m), which was suggested by Fredsoe et al. (1988) to be the 583 maximum value for free spans. During the seven years of observations, Leckie et al. (2015) found differences to occur in the scour and self-burial pattern along different stretches along the pipeline. They 584 585 relate these differences to variations in soil properties and hydrodynamic impact along the pipe. They 586 found mean far-field embedment around 0.3D at certain stretches along of the pipe, and attaining values 587 up to 0.6D at other stretches. No backfilling around the pipe in the scour holes was observed. They 588 attributed this to lack of infill of sediment; i.e. the live bed scour conditions occurred less than 0.6% of the 589 total time. Regarding the along-pipe scour propagation, Leckie et al. (2015) compared the measurements

with the model by Cheng et al. (2009), and found the model to over-predict the propagation rate. They
relate this over-prediction to a wider grain size distribution curve of the soil in the field than the soil used in
the laboratory measurements, on which Cheng et al.'s model is based.

593

594 Simulation of Real Storm Conditions

595 Draper et al. (2015) simulated the environment at the North West Shelf of Australia. This area is exposed 596 to cyclones, and the wave heights can reach 10-15 m. These peak values are reached during a period of 12 597 to 36-hours. They considered the scour development in the initial stages of a storm with acceleration in the 598 near bed flow velocities. Their simulation is based on available prediction tools, and they analyze two issues 599 about the scour impact; i.e. (A) Will a pipeline increase its stability during the initial stages of a storm, and 600 (B): Will sufficient scour happen during the initial stages of a storm to ensure sufficiently stability during the 601 peak of the storm. Their considerations assume no scour is present ahead of the storm. They concluded 602 from their model simulations, that scour in the environment under consideration occurs quickly enough in 603 the initial phases of a developing storm to ensure, that significant embedment occurs during a few hours, 604 or before peak condition in the hydrodynamic forcing arrives.

605

606 Impact of Vibrations of the Pipe on the Scour Profile

When the pipe is located in a free span, it may vibrate due to waves and/or current; see e.g. Sumer et al. (1989). The vibrations cause an additional pulsating flow around the pipe, which instantaneously increases the bed shear stresses, thus resulting in larger ability of bed erosion. This results in an expansion of the scoured bed profile. All investigations mentioned below are for the current-alone case; the wave-case still needs to be studied.

Sumer et al. (1988a) studied experimentally the scour development in the current-alone case, where the
pipe only was allowed to vibrate in the vertical direction. They observed that the scour beneath became
wider and deeper, increasing about 30 %. The changes became especially pronounced, when the gap

615 beneath the pipe became large enough to allow vortex induced vibrations (VIV) to occur. These vibrations

616 initiate larger vibration amplitudes of the pipe with associated larger scour. Gao et al. (2006) performed

617 similar experiments and studied the impact of initial positive and negative embedment values. They

observed a similar deepening of the scoured profile similar to Sumer et al. (1988a), and showed that next to

619 the vibration amplitude also the frequency has an impact on the scour development.

620 Shen et al. (2000) performed scour experiments with a pipe, which was allowed to move vertically and

horizontally. They found an impact from the vibrations to be slightly higher than the studies described

above with a pipe with only one degree of freedom in the motion. Shen et al. found about 50 % increase in

the area of the scoured profile in the dynamic case compared to the static one.

A special 3D case is investigated by Li et al. (2013), who studied scour in the transitional bend region from
the vertical part to the horizontal part of a catenary riser, i.e. a vertical bend with special emphasis on scour
from the oscillations of the vertical riser.

Numerical modeling. Zhao and Cheng (2010) modeled the impact of a vibrating cylinder with two degrees of freedom in motion. They applied the same modeling tool as Liang et al. (2005a,b), described above in the section "tunnel erosion". In their numerical simulations, they provide a detailed description of the vortices developing downstream the pipe, and the changes in the vibration patterns during the scour development are analyzed. For a transverse vibrating pipe, they found numerically that the scour depth increases about 25% compared to a fixed pipe. This value increases to 30% for a vibrating pipe with two degrees of freedom.

634

635 Other Backfilling Mechanisms

Additional to the touchdown of the pipe or changes in the wave height climate, backfilling can be caused by
a turn with time in the wave-current direction: waves running more or less parallel with the pipeline
alignment will backfill by infilling of moving bed load on the scoured slopes. This mechanism does not
depend on any presence of a gap beneath the pipe; see Fredsoe (1978, 1979). This backfill mechanism can

result in less self-burial in the field, than suggested by laboratory experiments with perpendicularapproaching wave/current.

642

643 Summary Regarding 3D Scour

644 The maximum length of the free spans is determined by the stiffness length of the pipe, and the observed 645 lengths are usually shorter. A mechanism to stop the expansion of the scour hole along the pipe is the 646 increased embedment of the pipe in the shoulders, which significantly reduces the scour below the pipe in 647 the free span. Slow sinking rate results in a lengthening of the span, because more time is available to scour 648 underneath. Vibrations of the pipe line will increases the length of the free spans because the scour 649 becomes deeper. A complete model including parameters such as KC, angle of wave approach and 650 strength/direction of a co-existing current is still lacking, but many of the elements in the process are now 651 physically understood.

652

653 Natural Variations in Seabed Level

654

655 The natural variations in seabed level near the pipe are obviously just as important for the pipeline stability 656 as the local scour. Migrating features can create other kinds of free span along the pipe than those 657 described above, when the pipe is not initially buried below the lowest level of active sediment transport. 658 An important parameter is the ratio between the stiffness length of the pipe and the radius of curvature of 659 the bed undulation; i.e. when this ratio is small, the pipe adapts to local bed elevations, and no additional 660 free gaps next to those created by local scour will develop. When the pipe is stiff, large gaps can develop 661 when the sea bed curvature is large. Other important parameters are the migration velocity of the bed 662 undulations, and the vertical variations in seabed level. These variations can be caused by several reasons:

• Bed sediments brought into suspension during storms.

- Migration of small- and medium-scale bed forms.
- Migration of large-scale bed forms.
- In nearshore areas: fluctuations in the nearshore profile.

667 Regarding the first item listed above, changes in the seabed level caused by entrainment or deposition of 668 suspended sediment usually is less than a few centimeters, and therefore constitutes a minor risk. The 669 second bullet above also constitutes a minor risk regarding small-scale bed forms, which usually are wave-670 generated ripples (vortex ripples), which seldom exceeds 5 to 7-cm in height. Mega ripples are the most 671 common medium-scale bed form, which is frequent in environments with tides. The tide does not 672 necessarily need to be very strong, because the sediment anyway can be transported by the current in 673 combination with waves as the agitating force. Mega ripples are typically 0.5 to 2-m in height and 30 to 674 200-m in wave length. Typically, they migrate with a speed equal 10 to 100-m per year. The migration 675 velocity depends strongly on their height, because high sediment waves move more slow than lower 676 sediment waves in an otherwise identical environment. When pipelines are installed in areas with mega 677 ripples, the pipe is usually trenched. The required trenched depth is obtained by performing statistical 678 analysis of the trough level of the mega ripples.

679 Large scale bed forms mentioned in the third bullet above includes tidal banks, which are found in 680 environments with tidal flow in combination with wind waves such as the English Channel, where they 681 easily can reach a height more than 8 to 10-m. The propagation velocity of these banks is slow, and the 682 slope is mild, causing the seabed level changes usually to be insignificant during a lifetime of 25 years. 683 Regarding the forth bullet above, the seabed variability in nearshore areas can be large. An important 684 feature is the breaker bars, which are moving back and forth. In addition, these bars are unstable in the 685 longshore direction because of rip currents. In these areas, the pipes nearly always are trenched below the 686 minimum expected sea bed level.

A discussion of different spans is given in Drago et al. (2015), and includes presence of sand waves and
large scale changes in sea bed bathymetry.

689

690 Other Pipe Cross Sections than the Single Circular

691

692	In this section, the 2D scour around pipelines with a cross section different from a single circular one is
693	shortly described. Examples on such configurations are spoilers, which are installed to increase the scour
694	and thereby facilitating the self-burial process. Also configurations such as side-by-side pipes (two in a
695	tandem or more) or the piggy pipeline, where a smaller pipe is located on top of a larger, are described.

696

697 Spoilers

A spoiler is a small flat plate, a fin, placed vertically at the top of the pipeline. The purpose is to enhance the self-burial on the cost of a higher exposure to hydrodynamic forces. The spoiler increases the strength and extent of the downstream wake, and thereby increases the wake-related contribution to the scour, thereby facilitating the onset of scour and increase the final scour depth. Hulsbergen (1984) reports, that a spoiler can increase the self-burial depth up to 2-3 times the pipe diameter.

Chiew (1992, 1993) studied the effect of spoilers in the current-alone case and in the wave case. Spoilers with a height equal 0.25 and 0.5 times the pipe diameter were used in his experiments. These spoilers increase the scour beneath the pipe by 39 and 46% respectively. Chiew also investigated the impact of a spoiler positioned at different locations on the upstream part of the pipe. He attributed the functioning of the spoiler to two different physical mechanisms; i.e. (A): An increase in the blockage ratio of the flow, more of the incoming flow is diverted to pass beneath the pipe, and (B): An increase in the size and strength of the downstream vortex and thereby increasing the downstream lee scour.

Oner (2009) measured the flow beneath the pipe with a spoiler applying PIV, and observed that this flow

hardly was increased by the implementing the spoiler.

712 *Waves.* Chiew (1993) experimentally studied the impact of a spoiler on a pipeline exposed to waves.

However, the pipe constituted a large fraction of the available water depth, which caused in wave

reflection and wave breaking. The results concerning a spoiler placed on the top therefore did not become

715 decisive for the case of a pipe located at larger water depths.

716 The impact of spoilers was studied numerically by Cheng and Chew (2003), who solved the 2D viscous NS-

equations, with the eddy viscosity kept constant in space. In contradiction to the later measurements by

Oner (2009), they predicted an increase in the flow rate in the gap between the pipe and the seabed, when
a spoiler is implemented.

720

721 Tandem pipeline

722 Two pipelines with the same diameter installed in parallel close to each other modify the scour pattern 723 beneath both pipelines. The modification depends on the spacing between the two pipelines. Zhao et al. 724 (2015) modeled the problem numerically with their (2008)-model and they compared their results with 725 experiments performed by Westerhorstmann et al. (1992). They analyzed gap ratios ranging from 0.5D to 726 5D and predicted for the current-alone case a slightly increased scour depth beneath the upstream pipeline 727 as compared to a single pipeline, and a much greater scour depth beneath the downstream pipe. The scour 728 attained its maximum value for a gap ratio about 2.5D between the two pipelines. In this case, the scour 729 beneath the downstream pipe became 50% larger than for a single pipeline. The gap ratio equal to 2.5D 730 coincides with the minimum distance for the vortex shedding from the upstream pipe to occur. For gap 731 ratios larger than 2.5D, the impact for the upstream vortex shedding on the downstream scour weakens.

732

733 Piggyback pipeline

734 A piggyback pipeline is formed by two pipelines with different diameters arranged in a bundle, with the 735 smaller pipe located above the larger pipe, and either in direct contact with each other or with a small gap. 736 This problem has been studied numerically by Zhao and Cheng (2008) for the current-alone case. They 737 calculated an increase of about 40% in the scour depth for a surface mounted smaller pipeline with a 738 diameter equal 0.2D, D being the diameter of the larger pipe. In this case, the smaller pipe functions similar 739 to a spoiler. For a gap equal to the diameter of the smaller pipe, the scour just beneath the larger cylinder 740 only increased by 10% as compared to no pipeline above. In the latter case, there are two vortex streets, 741 one behind each cylinder. For smaller gaps, the two pipelines act as a single body with only one vortex 742 street. Yang et al. (2013) also studied the flow around the piggyback exposed to waves, however without 743 doing any scour calculations.

744

745 IMPACT OF SOIL PROPERTIES ON SCOUR

746

Frodibility of sandy soil depends next to the Shields parameter on the content of clay and/or silt and on thegradation of the sediment.

749Sediment gradation. Armoring of the bed may occur for soil with a wide grain distribution curve, because750the coarsest fraction shields the bed. This implies, that a coarser grain diameter about d_{85} shall be applied751in the definition of the Shields parameter defined in Eq. (4). Dey and Singh (2008) studied the scour depth752beneath a pipe placed on non-uniform sediment, and observed a large impact of the gradation. Only clear-753water scour was studied, and the impact is probably also most significant in this case, where the mixing of754moving sediment is minor.755Clay/Silt/sand. The content of clay or silt in combination with sand in the seabed changes the beds erosion

resistance significantly by introducing cohesive properties: adding increasingly amounts of mud to sand

increases the erosion resistance, with a transition occurring in the region 3-15% according to Mitchener et

al. (1996). In their study, they found the maximum resistance to erosion occurs at 30-50% content of sand.
At even higher content of mud, the bed's resistance to erosion will again decrease.

Only a few studies have been made on the pipeline scour in different types of soil. Postacchini and

761 Brocchini (2015) reported experiments with wave induced scour below a pipe in sandy soil with different 762 concentrations of clay content. They found, that for clay content lesser than about 5%, the scour still 763 depends on KC, and the scour decreased with increased clay content. For higher clay content, the 764 outcome of their scour experiments became more uncertain from difficulties in establishing exactly the 765 same initial test conditions related to compaction during reshaping of the bed. Based on their experiments, 766 they proposed a formula for the scour depth, which next to KC also includes the concentration of clay. 767 Additionally, Postacchini and Brocchini (2015) suggested another formula at higher clay concentrations 768 based on data from Kumar et al. (2003) and Zhou et al. (2011).

Mohr et al. (2016) studied the time scale for erosion for a wide range of grain sizes and grain distributions,
including marine sand with a clay content of 3 and 8 %. For coarser sediment, they confirmed the
prediction by Eq. (7), while the time scale becomes increasingly higher for fine silt and for sand with clay
content (9% finer than .2- µm). They proposed an expression for the timescale for fine sediment based on
the erosion rate of the soil.

Supplementary to pipeline scour experiments, additional information on soil properties impact on scour can be transferred from the related problem of scour around a vertical cylinder. This type of structure has undergone more extensive investigations than the pipeline case, especially for the current-alone case. Dey et al. (2011) studied scour caused by waves in a sand-clay mixture around a vertical pile and found that when the proportion of clay exceeds 30%, the scour depth became the same as in clay alone. For a clay content less than 30%, the equilibrium scour depth decreases as the clay content increases. The timescale for the scour increases significantly with the clay content and with *KC*.

Sumer et al. (2007) considered pile scour for sand and silt with different relative density S_r of the seabed.

This density is a measure for the compaction of the soil, and is given as follows

33

783
$$S_r = \frac{e_{v,\max} - e_v}{e_{v,\max} - e_{v,\min}}$$
(10)

in which e_v is the void ratio. Loosely dumped sand and silt may attain a relative density as low as about 784 785 0.35, while compacted soil reaches values up to about 0.85. The experiments by Sumer et al. (2007) 786 demonstrates that the scour increases as S_r increases, and was explained by the increased angle of friction 787 of the denser packed sediment, which makes the hole deeper for the same horizontal extent of the scour 788 hole. This is not important for the tunnel scour beneath the pipe, but may play a role at the span 789 shoulders, where the slope often approaches the angle of repose. 790 The investigation by Sumer et al. (2007) observed the timescale to be largest for the dense-silt case and the 791 smallest for the sand case. 792 **Protection measures against scour** 793 794 795 Scour can be reduced or totally avoided by installing flexible mattresses around it, either placed above or 796 beneath the pipe. The horizontal extent away from the pipe must be sufficient large, so the edge scour at 797 the outer periphery is sufficiently reduced to ensure the mats stability. Chiew (1990) experimentally 798 investigated the impact of installing an impermeable plate at the upstream side of a pipe placed on the bed 799 and exposed to a current. This plate prevents the onset of scour, because the plate lengthens the 800 streamlines of the seepage flow beneath the pipe, and thereby reduces the pressure gradient in the soil. 801 Yang et al. (2014) similarly investigated the impact of a rubber plate placed symmetrical below the pipe, 802 but only exposed to steady current. 803 On a live bed, the stability of mattresses may not be secure because of external generated morphological 804 changes. Migrating sand waves can scour around even flexible mats. In case of large degradation in the

surrounding bed area and when the mats, as is most common, are placed *above* the pipe, the impact of the
protection may become negative. This negative impact happens when the mats finally hang over the pipe,
and thereby enhances the scour by sweeping away sediment, when the mats move back and forth by
waves and current. Problems similar to these are also encountered near the pipeline inlet to an offshore
platform. In this case, the platform itself may create a depression in the bed area from local scour around
the platform.

The most flexible scour protection is a falling apron consisting of wide graded material. This protection can withstand moderate bed level changes without serious damage. Flexible mats may also work as protection against scour, if they are placed properly *below* the pipe.

814

815 Liquefaction

816

817 Non-cohesive soil in the seabed exposed to waves may undergo liquefaction, where the soil-water mixture 818 is transformed into a liquid. When liquefaction occurs, pipelines originally placed on the seabed may sink, 819 when their submerged density is higher than the liquefied soil density. Buried pipelines may similarly float 820 to the bed surface, when their submerged density is smaller than that of the surrounding liquefied soil. 821 Cohesive soil may liquefy in the way that it can lose its shear strength during cyclic loading (strain softening, 822 cyclic degradation as described by e.g. Vucetic (1991)). In the following description, focus is primarily 823 directed to non-cohesive material. 824 Usually liquefaction caused by earthquake is the major cause for liquefaction of soil because its large 825 amplitude in the oscillation and higher frequency (earthquake: O(1sec), waves: O(10 sec)). However, also 826 waves can cause liquefaction as detailed in two recent books by Jeng (2013) and Sumer (2014). In the wave 827 case, the cyclic variation in shear stresses in the soil is caused by the pressure variation along the seabed
from the migrating surface waves (positive pressure under the crest where the water depth is large and negative pressure under the trough). These pressure variations induce shear stresses in the soil, when the waves propagates as shown in Yamamoto (1981) and Madsen (1978), who both provide analytical solutions for the induced shear stresses in the soil.

Sawaragi and Deguchi (1992) showed that in the soil, the wave-induced bed shear stress has negligible
impact on soil shear stresses as compared to the stresses induced by the bed pressure variations from the
waves.

Liquefaction of the seabed caused by waves can occur in two different ways: beneath the wave trough, the seepage flow is upward directed, because the small seabed pressure. This loosens the soil and eventually causes the soil to be liquefied. Since this kind of liquefaction only has a short duration during passage of the wave trough, this is called *instantaneous liquefaction*.

The other mechanism is related to loosely deposited sediment, which can be compacted by the waves, and thus create of an excess pore pressure in the bed soil. During the compaction phase, this pressure may be sufficiently large for a period of time, causing the effective grain stresses to disappear, and liquefying the soil. This type of liquefaction is called *residual liquefaction*.

843

844 Instantaneous Liquefaction

845

Instantaneous liquefaction is the process, where a column of soil can be lifted by a pressure lift force. This
force is caused by the difference between the wave induced excess pressure in the soil at a depth z below
the sea bed surface and the pressure at the seabed at the same location. When this difference becomes

849 larger than the overburden pressure of the soil-fluid mixture, the soil will move vertically as a block. The 850 overburden weight W above a level z below the sea bed is as follows

851
$$W = \rho(s-1)g_z(1-n)$$
 (11)

In Eq. (11), *n* is the initial porosity of the soil. The upward directed hydraulic gradient *i* in the soil below the wave trough must exceed $W/(\rho g)$ to obtain instantaneous liquefactions. In a discussion of the paper by Moshagen and Tørum (1975), Prevost et al. (1975) demonstrated, that hydraulic gradient usually is too small to create liquefaction, even under large waves.

856 Impact of gas. When the soil contains gas, the vertical pressure gradient just beneath the seabed surface 857 increases, because wave-induced pressure dampens faster from the sea bed. This dampening is caused by 858 the higher compressibility of gas compared to fluid (Tørum, 2007). The gas (most common Hydrogen Sulphid) can stem from deterioration of organic material, and there is nearly always a small fraction of gas 859 860 as a part of the fluid in the seabed, but it is difficult to measure, see e.g. Sandven (2007). Tørum (2007) 861 suggests that 3% gas content may be enough to introduce liquefaction at the seabed. However, when the 862 gradient is steep, the liquefied layer will correspondingly be thin, and therefore the impact on vertical 863 motion of a pipeline may be negligible.

Impact on pipe movement. The pipe can adjust its position vertically as well as horizontally, when the surrounding soil is liquefied and the pipeline specific gravity is different from that of the liquefied soil. However, in a real environment, the instantaneous liquefaction must occur simultaneous a minimum distance along the pipe to allow movement. This distance must constitute at least a considerable fraction of the stiffness length of the pipe to obtain movement of the pipe. This condition requires the waves to approach the pipe nearly perpendicularly and have a strong 2D-feauture; i.e. long wave fronts.

870

871 Residual Liquefaction

872

873 Residual liquefaction is the process, where loosely deposited fine sediment is moved internally back and 874 forth by shear stresses and obtains a denser, compacted grain skeleton. When this sediment is loaded for 875 the first time (named "virgin loading" by e.g.de Groot et al (2006)), the individual grains can easily rock or 876 even pass over the underlying grains. In average they obtain a denser packing, sometimes after a transient 877 even looser packing (dilatation) during the passage of one grain above the other. This process is well-878 described by De Grooth et al. (2006). The internal displacement of the grains is primarily caused by shear 879 stresses, while an overall increase or decrease in normal stresses (pressure) only causes compression. 880 Sawicki (2014) argues that the gradient in normal stresses also contributes to internal displacement. 881 The compaction process leaves empty space, and implies a reduction in the effective grain stresses. 882 Therefore, redistribution between soil and fluid stresses occurs; i.e. at this stage the fluid pressure carries a

higher fraction of the overburden pressure W originating from the sediment-water mixture above, while
the effective stresses between the grains reduce correspondingly, and thereby increases the excess
pressure p⁺ in the fluid. This pressure acts on the fluid, and squeezes the excess water from the soil-fluid
mixture into the fluid above the seabed. Thereby the excess pressure is relieved. When this relief does not
occur fast enough, the excess pressure p⁺ at a depth z might build up to a level that exceeds W and the soilfluid mixture becomes liquefied above this depth.

The conditions for the soil to liquefy therefore depend on the strength of the cyclic shear stress, the number of waves, the permeability of the soil and the initial compaction ability of the soil skeleton before heavy wave impact. For cohesion-less soil the latter is most often characterized by its relative density.

892 Therefore, the requirement for liquefaction is

- Sufficiently large waves to ensure that a high value of the excess pressure p⁺ is obtained.
- Low permeability soils, causing p⁺ not to be relieved sufficiently quickly; i.e. the sediment must not be
 too coarse because coarse sediment has a large permeability.
- Loosely deposited sediment, where the grain skeleton can be consolidated in the virgin load phase.

Fig. 10 shows a typical recording of the variation in p⁺ at four different vertical levels in a flume test performed by Sumer et al. (2006c), where loosely silt liquefies by propagating waves. The recording shows the typical features of the cycle; i.e. initially, the pressure build-up begins quite similarly at all measured depths in the soil and this build-up occurs during the first minute after the waves are switched on. This time corresponds to the time the grains need to obtain a denser configuration. The increase in p⁺ ceases, as liquefaction evolves at the different z-values.

The liquefaction begins initially at the surface of the bed, where least overpressure is required to carry the overburden weight W, which increases as the distance z from the bed increases, c.f. Eq. (11). From the bed surface the liquefaction penetrates downward until the depth is reached, where p^+ becomes smaller than W.

907 The second stage is a decrease in p^+ because of an outflow of the excess water in the bed. This outflow is 908 created by the upward hydraulic gradient in p^{+} , and begins at the transition from the solid to the liquefied 909 bed (the solidification line.) When the fluid escapes, the grains in the liquefied mixture begins to settle on 910 the solidification line, and now in a more compact manner than in the former loose phase. The compaction 911 is ensured by the oscillatory shear stress variation by the waves, and as a result the solidification moves 912 upward, causing a denser compacted bed at the end, as observed in laboratory experiments, Sumer et al. 913 (2006a). Sawicki (2014) discusses the re-solidification, and states that earlier liquefaction is erased, 914 because the soil once more will be loosely deposited, causing the virgin state to be reestablished. This is 915 not in line with laboratory investigations like Sumer et al. (2006a), however it may occur under

916 circumstances like a sudden decrease in the environmental impact. If pre-consolidation does not exist from
917 earlier storm events, the potential liquefaction risk increases considerably.

918 Impact of the pipe on liquefaction. The presence of the pipe itself in the soil will change the liquefaction

patterns compared to those developing in homogeneous soil: Sumer et al. (2006b) observed a faster

920 buildup of the excess pressure, when the pipe is present. Initially the liquefaction was observed to occur

just beneath the pipe, while the impact of the presence of the pipe at the top of this pipe was negligible.

922 This is in accordance with the numerical modelling results by Dunn et al. (2006).

Experiments. Laboratory experiments can be performed in a wave flume with real waves or by centrifuge
tests. The records shown in Fig. 10 are based on small scale laboratory wave flume experiments with 10 to
20-cm high waves. Because the stress-strain relationship of the soil depends on the stress level, many
liquefaction experiments are performed by centrifuge tests to obtain prototype stress levels in the soil; see
e.g. Sassa and Sekuchi (1999) and Miyamoto et al. (2004). Introducing dimensionless parameters, Sumer
(2014) obtained a good agreement between the two types of tests regarding the pressure built up.

929

930 Behavior of a Pipeline in a Liquefied Bed

931

A change in the pipe position primarily occurs in residual liquefied soil, which in contrast to instantaneousliquefied soil includes a larger area of the seabed simultaneously.

Sumer et al. (1999) considered the 2D case in the laboratory; i.e. a 4 cm pipe was placed with its center 5-

935 cm below a seabed, which consisted of 0.045 mm silt. The seabed was exposed to waves and became

936 liquefied. The pipe was found to adjust its vertical position in the liquefied flow depending on the ratio of

pipe specific gravity ρ_p compared to the specific gravity of the sediment-fluid mixture of liquefied soil ρ_l :

938 when ρ_p is higher than ρ_l , the pipe obviously will sink, and adjust its vertical position to that level, where 939 the two densities become the same. ρ_l usually increases as the distance z from the bed increases, because 940 the grains in the liquefied soil settle, see e.g. Sumer et al. (2006c), Typical average values of ρ_l is 1.8-1.9 ρ , 941 which provides an estimate for whether the pipe will sink or float.

942 Teh et al. (2003) similarly tested a 7.5 cm pipe in 0.033 mm soil. The specific gravity of the pipe varied from 943 1.1 to 2.1. They observed that for specific gravities more than 1.8, the pipe sank in the liquefied soil. Teh et 944 al. (2003) divided the pipeline behavior into three different modes; i.e. (A): A light slow sinking pipe, where 945 the excess pore pressure in combination with the liquefied soil acts as a buoyancy force on the pipe. (B): A 946 light pipe in combination with more permeable soil; i.e. when the excess pore pressure dissipates quickly, 947 the pipe stops sinking before the pipe has attained its equilibrium position. This is because the solidification 948 line in the meantime has moved upward, and prevents further sinking. (C): The pipe is heavy (fast sinking), 949 causing it to sink before the excess pressure disappears. In this case the pipe simply reaches the lowest 950 level of the solidification curve. Teh et al. (2006) developed a simple analytical model for the sinking 951 velocity of the pipe and for the rising velocity of the solidification curve.

952

953 Residual Liquefaction in a Natural Environment

Similar to scour, liquefaction also needs time to develop, and similar to scour, the question occurs
regarding liquefaction about what happens during the rise and fall of a storm. Can the initial phases of the
storm create liquefaction followed by solidification, before the peak conditions occur; i.e. what is the
timescale for the whole liquefaction process compared to the acceleration of the storm? When the bed
becomes liquefied during a storm, the next question is whether in the final part of the process, the
compaction will be completed before the hydrodynamic impact has ceased. The timescale for compaction

strongly depends on the settling velocity of the fine sediment, which can be small because of the largeconcentrations of sediment in the liquefied soil; i.e. "hindered settling".

When the compaction process is fully completed, liquefaction should not be possible to occur a second time, when the bed once more is exposed to a storm with same strength and duration, because the soil is no longer in its virgin stage, but pre-consolidated. Whether an even heavier storm in this case will give rise to liquefaction is a little bit open; i.e. will the upper compacted layer act as a shield for a lower less compacted layer?

When the compaction process has not been completed fully up to the surface of the seabed before the
storm has ceased, the sediment in the upper layer will settle loosely in the bed, and can easily be liquefied
again. However, the lower layers can be compacted, because the compaction process begins from below.
This compacted layer may act as a shield for additional penetration of the liquefaction into the bed, but
experimental evidence for this situation has not been studied experimentally yet.

972

973 Impact of Fines in the Soil

974 Loose sand often has a content of finer material of silt or clay in the pores. Kirca et al. (2014) studied the 975 influence of clay content on the liquefaction risk. The presence of clay in sand/silt reduces the permeability 976 of the soil, and thereby allows the excess pressure to rise to a higher value, before the drain of excess 977 water begins to relieve the excess pressure. Even coarser fractions of sediment such as fine and medium 978 sand increase their risk of being liquefied, when the clay content is sufficiently high. In their experimental 979 study, Kirca et al. (2014) observed, that at least 10% clay content was necessary to obtain liquefaction for 980 0.4 mm sand. In addition, they observed in their experiments with silt, that when the clay content exceeds 981 about 30 %, the clay-silt matrix changes its physical properties; i.e. the matrix behaves as high plasticity

clayey silt, which will not allow the coarser grains to rearrange under the cyclic shear strains. Liu and Jeng
(2015) performed similar experiments and found the same trends.

984

985 Measures against Liquefaction

A risky area for being liquefied is the backfilled cover of a trenched pipe, where the soil can be loosely deposited. This cover is one of the most vulnerable locations for possible liquefaction until it has been compacted by waves, most likely through a liquefaction process. The risk of liquefaction of a backfilled trench can be reduced by adding layers of permeable material (coarse sand, pebbles, and stones) above the trench. These additional layers imply that the excess pressure must overcome a larger weight W to liquefy the soil, and additionally the coarse layers remove the risk of liquefaction in these upper layers. The impact of adding a cover layer of armor stones on top of liquefiable sediment was studied by Sumer et al. (2010).

993

994 Mathematical/Numerical Modeling

- 996 The cyclic variation in shear stresses and pressure and the resulting build-up of the excess pressure caused997 by compaction can be modeled using the following four sub-elements:
- 998 A shear-strain relation for the soil.
- Knowledge of the compressibility of the fluid and the soil skeleton.
- A relationship for the consolidation describing the pore volume reduction by the cyclic imposed shear
 and pressure.

A flow resistance law (such as the Darcy law) to describe the seepage flow as a function of the hydraulic
 gradient.

1004 The wave-induced cyclic soil stresses have been calculated by numerous researchers, and a comprehensive

1005 review is given by Jeng (2003). This review describes next to the classical solution of seepage flow in a

1006 porous incompressible and non-deformable bed (see e.g. Sleath (1970)) also a number of

1007 numerical/analytical studies, where the fluid and/or the grain skeleton are compressible. Most studies are

1008 based on Biot's (1941) consolidation model.

1009 Instantaneous liquefaction. Moshagen and Tørum (1975) treated the wave induced seepage flow by

1010 including the compressibility of the fluid phase, while Yamamoto (1977) and Madsen (1978) also included

1011 the compressibility of the soil in their analysis to calculate pressure and shear stresses.

1012 *Residual liquefaction.* The consolidation is usually considered as a plastic deformation. Examples of such

solutions are given by Rahman et al. (1977) and McDougal et al. (1978). They introduce a pore water

1014 pressure generation term, which depends on the magnitude of shear stress, number of waves, soil type and

1015 relative density of the soil.

1016 A large number of later studies on anisotropy, limited soil depths etc. are described by Jeng (2013) and1017 Sumer (2014).

1018

1019 Does Loosely Deposited Sediment Exist in the Seabed

As discussed above, soil exposed to waves usually consolidates, however with some reservations regarding the timescales for the liquefaction process. In addition to a recent backfilled cover of a trenched pipe, other risky locations can be areas where compacted seabed soil is loosened. This occurs when the seabed becomes stirred up by natural variations in the seabed as mentioned earlier. As an example, the sand
passing the crest of migrating sand waves and deposited on the sand wave front is loosely deposited.

1025

1026 Lateral Stability of a Pipeline on an Erodible Bed

1027

1028 Scour around a pipeline changes its stability against lateral motion. However, nearly no investigations have 1029 been performed which include the impact of scour on the lateral movement of a pipe. The most common 1030 case studied by geotechnical scientists is the impact of passive soil resistance on a pipe from the slight 1031 embedment. A pipe placed on a soil surface usually will experience a slight embedment because the 1032 submerged weight and also because the installation procedure. When embedded and exposed to 1033 hydrodynamic forces, the pipe experiences resistance against lateral motion from two contributions; i.e. a 1034 frictional component and a passive resistance component from the soil. This problem can be investigated 1035 experimentally by moving the pipe back and forth on a saturated soil bed in a controlled manner.

1036 Based on experiments with a sandy bed, Gao et al. (2007) describes the behavior of a slightly embedded 1037 pipe, which loses its lateral stability as a result of continuously increasing action by current or waves. In the 1038 current-alone case, the pipe initially pushes the nearby soil ahead with an associated slight lateral pipe 1039 displacement, until it at a certain time loses its stability and breaks out with a resulting large displacement 1040 in the current direction. In the wave case, the pipe initially rocks back and forth with the wave period and a 1041 very small amplitude. Very soon, the amplitude of the pipe-motion increases to significantly larger 1042 horizontal displacements, and this motion creates a berm on each side of the pipe by pushing the sediment. 1043 Regarding the current-alone case, Gao et al. (2011) found experimentally (using a mechanical-actuator 1044 simulation) that the distance by which the pipe moved before breakout decreased with increasing initial 1045 embedment.

1046 Wagner et al. (1989) performed a number of tests with a rig, which moved pipes with diameters equal 0.1-1047 m and 0.5-m back and forth over a sandy and clayey bed. Because only air and no fluid was present above 1048 the sediment bed, the scour and seepage flow are not present in the experiment. The properties of the 1049 sand range from loose silty sand to loose and dense medium to coarse sand. The pipes were moved in a 1050 controlled manner vertically and horizontally. Wagner et al. found that in the sandy-bed case, the largest 1051 increase in passive soil resistance occurred in the case of loose medium sand. This result is linked to the 1052 larger penetration of the pipe in loose than in dense sand. They suggested a formula for the total lateral soil resistance F_H in a sandy bed given by the sum of a sliding resistance component and a passive soil 1053 1054 resistance component

1055
$$F_H = \mu (W_S - F_L) + \beta \rho g(s-1)A$$
 (12)

Here, W_S is the submerged weight of the pipe per unit length, F_L is the hydrodynamic lift force, μ is the sliding resistance coefficient (which they put equal 0.60), A is half the soil cross-sectional area displaced by the pipe caused by penetration and oscillations, and β is an empirical dimensionless coefficient, which was measured to vary from 38 for loose sand to 79 for dense sand. With this model, they obtained nice agreement with measured and calculated lateral pipe motion.

1061 Wagner et al. (1989) also tested a pipe placed on a *clayey* bed, and in this case the contribution from the 1062 lateral soil pressure relative to the total soil resistance was measured to be larger compared to the sandy-1063 bed case. This increase is because the pipe sinks more in clay than in sand. Wagner et al. finally studied the 1064 impact of consolidation of the clay: in the first number of cycles, the consolidation increases the soil 1065 resistance, but this effect disappears after a few cycles even at a few small amplitude oscillations. For the 1066 clay-case, they proposed a formula similar to Eq. (12), but the last right side term was changed to $\beta cA / D$, where c is the remolded undrained shear strength for clay. In the clay-case, β depends on pipe 1067 1068 displacement and lateral load history.

Hale et al. (1991) concluded that Wagner et al.'s model is conservative, when compared to the pipe/soiltests on which they are based.

1071 Gao et al. (2002) studied the soil resistance in the more realistic case, where fluid is present above the bed. 1072 The experiments were performed in a U-tube, and the pipe was moved back and forth by the fluid forces in 1073 two modes; i.e. with or without rolling (anti-rolling). To obtain sufficient information on the interaction 1074 between the developing scour and ploughing from the pipe's lateral motion, they performed a large 1075 number of experiments with different Shields parameters and different amplitudes and frequencies in the 1076 lateral pipeline motion. Gao et al. (2002) also performed a number of combinations of lateral motion and 1077 initial embedment of the pipe, while the variation in the Shields parameter was limited. Therefore the 1078 importance of scour on the initial lateral motion was not fully clarified. However, by increasing the 1079 oscillatory fluid action gradually, they observed that onset of scour developed before the pipe began to 1080 rock. The breakout from its original position occurred as the last stage of the pipeline instability. For certain 1081 near-bed flow strength, they observed the pipe to be more unstable in waves than in a steady current. 1082 Anti-rolling pipes were found to be much more stable than freely laid rolling pipelines. Gao et al. (2007) 1083 provides alternative expressions to those by Wagner et al. (1989) for the lateral stability in waves and in 1084 current for the sandy-bed case. They suggested the stability to be as follows

 $1085 F = \alpha + \beta G (13)$

1086 in which F is the pipe Froude number and G is the dimensionless submerged weight of the pipe

1087

$$G = W_s / (\rho(s-1)D^2)$$

The two constants in Eq. (13) depend on the soil and whether the pipe is exposed to current or waves: α is 0.1 and β is 0.42 for medium sand combined with a current. For fine sand, β decreases to 0.2. In the case of waves in combination with medium sand, α decreases to 0.05, while β maintains the value 0.4. The impact of *pipe roughness* was investigated by Gao et al. (2011). Initially a rough pipe has a smaller embedment than a similar smooth pipe, but the final maximum settlement in the breakout process is larger for the rough pipe case. The impact of the initial sand bed *slope* for the pipes lateral stability in waves was

(14)

studied by Gao et al. (2012). They found the soil resistance increases most regarding the downslope
instability, and the lateral-soil-resistance coefficient is larger for the up- and down-slope case compared to
the corresponding horizontal seabed case.

1097 Large lateral motion. White and Cheuk (2008) studied the soil-pipe interaction over soft cohesive soil 1098 (kaolin clay), where the lateral motion was about 10-20 times the pipe diameter, to allow the pipe freely to 1099 buckle laterally in a controlled manner caused by thermal expansion. In this case, self-burial is unwanted, and a detailed description is given of the berm development around the pipe, when the pipe is moving back 1100 1101 and forth and ploughing the bed. The importance of these berms is that when they are created by 1102 moderate wave action, they can increase the passive soil resistance at even higher waves, than those which 1103 formed the berms. This increase will restrict the possibility for larger lateral motion. White and Cheuk 1104 (2008) provided a heuristic model for the large lateral motion of the pipe based on a simple kinematic 1105 hardening model for the soil.

1106

1107 Modeling

The modeling of the pipeline soil resistance is primary performed applying a geotechnical perspective without including possible scour from the flow. The models are heuristic in their character. The clayey soil behavior is complex and only behaves elastic for very small deformations. At larger deflections, the constitutive equations include plastic deformation within the yield surface. The changes in strength can decrease the soil resistance with time from softening; i.e. cyclic degradation of clay as described e.g.by Vucetic and Droby (1991). Clay as well as non-cohesive soil can also undergo hardening from consolidation and compaction as described e.g. by Elgamal et al (2003).

21115 Zhang et al. (2002) modeled the stability of a pipe placed on an elastic-plastic medium, where the bed 2116 deformation is based on a kinematic hardening model, such that the pre-loading history is memorized. 2117 They did not split the flow resistance into two terms as was done by Wagner et al. (1989), but introduced 2118 an integrated model, where induced incremental displacement tis linked to the incremental load. The

hardening was linked to the vertical plastic displacement and was calibrated with existing centrifuge test to
obtain a valid full scale model. It has a smooth transition from the elastic state at small loads to the plastic
state at the yield. Tian et al. (2010) observed that the hardening law deviates from the centrifuge test for
relatively small pipeline lateral displacements less than about half the pipe diameter. Tian et al. attribute
this discrepancy to the missing presence of the berm in previous models, and they included this in the
model. However, no scour was included.

1125

1126 Liquefaction Caused by Pipe Motion

1127 The movement of the pipe on the seafloor can create instantaneous liquefaction as described by Foray et 1128 al. (2006). They observed a strong increase in the excess pore pressure at the pipe-soil interface around a 1129 cyclically loaded pipeline. The increase is sufficiently large to create instantaneous liquefaction in a soil 1130 band close to the pipe wall. Because a strong negative pore pressure occurred when inverting the loading 1131 direction, there was no general residual liquefaction observed in the experiments by Foray et al. (2006). 1132 The instantaneous liquefaction caused an increased penetration of the pipe into the seabed. This increase 1133 resulted in a larger lateral resistance. The increased densification of the soil and the negative pore pressure 1134 on the downstream part of the pipe also contributed to a larger soil resistance.

1135

1136 Pipe Behavior on a Liquefied Bed

The special case, in which the bed becomes liquefied but without any scour around the pipe has been studied by Teh et al. (2003) in a wave flume with silt in the bed. They performed their experiments with low Shields numbers to avoid the scour. They studied the behavior of two pipes, both of which were heavier than the surrounding fluid-soil mixture, but one was significant lighter than the other. They identified two different types of behavior; i.e. the light pipeline became unstable before the bed became liquefied and formed a small trench by rocking before the soil became liquefied. On the contrary the heavy pipe caused the bed to liquefy before the pipe began to rock. Similar to Sumer et al. (1999), Teh et al. observed that the

1144	heavy pipeline sank to a specific level in the liquefied soil, depending on soil parameters and the specific
1145	gravity of the pipe. The final embedment of the heavy pipeline was influenced by the initial embedment
1146	depth of the pipe, while it was not the case for the light pipeline.
1147	Impact of Scour
1148	The impact of scour has only been included in very few experimental tests; i.e. those by Gao et al. (2002).
1149	Because the deposition berms at the downstream side (current alone) or both sides (waves) are formed by
1150	soil pushed by the pipe and therefore recently deposited, the berms are easy to erode. Therefore reshaping
1151	of these berms caused by flow around the pipe is most likely to occur.
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Pipeline scour and liquefaction have received an increased attention during the last decades, and the modeling and experimental part is well covered especially with respect to 2D scour in non-cohesive soil.
Regarding cohesive soil, the experimental part is sparsely investigated, and the modeling part is nearly nonexisting. A number of topics are listed below, which need investigations to improve the prediction tool for the pipeline-seabed evaluation.

Conclusion and Future Research Needs

Flow modeling: Regarding the flow modeling, one improvement needed is to describe the flow with a 1160 more accurate Re - variation, thereby reducing scale effects. The combined case of waves and current 1161 1162 can still be investigated even more detailed than today, but such investigations most likely do not lead 1163 to any deeper fundamental understanding. Obliquely incoming waves and current, potentially with 1164 different approach directions, still lacks a throughout analysis. The modeling of the timescale for the 1165 scour process can be improved by including the impact of excess turbulence in the wake. This modeling 1166 requires an improved turbulence description of the wake, and an improved description of the sediment 1167 transport in areas dominated by excess turbulence as demonstrated by Sumer et al. (2003).

3D scour development. This research area is only sparsely covered and restricted to co-directional
 wave-current motion. Additional investigations are needed in the laboratory and on the modeling side.

1170 The numerical modeling tool has been improved during recent years, and work such as Jacobsen (2015)

1171 to solve the 3D scour continuity equation is tailor-made for such investigations. Regarding laboratory

1172 experiments, the combination of waves and current with different angles of attack needs to be

1173 investigated to get an operative realistic model for the span development.

1174 Nearly all laboratory experiments and models assume the pipe to be straight. One exception is Li et al.

1175 (2013), who studied scour near a vertical bend. A special 3D effect also occurs near a horizontal

1176 pipeline bend. Such a shift in the horizontal direction is frequently near the inlet to platforms, and

sometimes bending more than 90[°]. The scour around such a bend may concentrate/diverge the flow,

and thus creating 3D scour features, which still awaits investigation.

Sediment properties and scour. The soil properties play a major role in the seabed behavior. Most
 important is the presence of cohesive sediment, which is difficult to handle experimentally and to
 model numerically. Regarding non-cohesive sediment, the gradation is very important, because the
 coarser fractions can armor the bed. None systematic investigations on the impact of the soil gradation
 on pipeline scour in non-cohesive sediment have been performed for the live bed case.

Real environment. Transforming theoretical and experimental findings to a real environment, where
 the environmental impact is highly unsteady and 3D with shifting directions of waves and current is a
 major remaining problem. Draper et al. (2015) made such an attempt regarding the scour behavior
 during a storm-event, and such investigations should be extended to cover 3D scour effects and
 liquefaction. The latter shall incorporate history effects.

Lateral movement. Impact of scour on the lateral stability of pipelines needs to be incorporated in
 existing heuristic models.

Natural bed level variations. While large and small scale bed level variations partly are qualitatively
 understood, a quantitative description of bank and bar migration is still lacking. Also the interaction

- 1193 between a pipe and migrating sand waves needs to be investigated with special attention to impact of
- 1194 local seabed curvature and pipeline stiffness.

1196 Notation

- *a* = orbital amplitude;
- A = half the soil cross-sectional area displaced by the pipe;
- d = mean grain diameter;
- D = outer diameter of pipeline;
- e = embedment depth of pipe below original seabed;
- e_v = void ratio, appearing in Eq. (10);
- EI = bending stiffness, appearing in Eq. (9);
- $F = U / \sqrt{gD}$ = pipe Froude number;
- F_L = hydrodynamic lift force;
- $g = \operatorname{acc.} \operatorname{of} \operatorname{gravity};$
- G = the dimensionless submerged weight of the pipe, defined in Eq. (14);
- *i* = hydraulic gradient;
- KC = Keulegan-Carpenter number, defined in Eq. (3);
- $l_s =$ stiffness length, Eq. (8);
- *n* = porosity of soil;
- 1213 p= pressure;
- p^+ = excess fluid pressure in the sea bed;
- 1215 Re = Reynolds number, defined in Eq. (5);

- *s* = relative density of sediment;
- S_r = relative density of the seabed, defined in Eq. (10);
- T = time scale for tunnel erosion;
- T^* = non-dimensional time scale, defined in Eq. (7);
- U = near-bed flow velocity;
- U_c = near bed current velocity;
- U_m = maximum orbital velocity at the bed;
- W = Submerged weight of fluid-sediment mixture;
- W_s = submerged weight of the pipe per unit length;
- α_r =relative strength between waves and current defined in Eq. (6);
- β = an empirical dimensionless coefficient;
- μ = sliding resistance coefficient;
- V = kinematic viscosity of the fluid;
- ρ = fluid density;
- ρ_p = pipe specific gravity;
- ρ_l = specific gravity of the sediment-fluid mixture of liquefied soil;
- au_b = bed shear stress;
- θ = The Shields parameter, defined in Eq. (4);

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1464 Figure Caption List

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- 1466 **Fig. 1**. Vortex system around an embedded pipeline exposed to a current.
- 1467 Fig. 2. Onset of scour beneath a pipe in current and with waves. Data from Sumer and Fredsoe (1991) and
- 1468 Sumer et al. (2001). Squares: $2 \le KC \le 7$, Circles: $7 \le KC \le 15$, Triangles: $15 \le KC \le 30$. The half-filled
- symbols include tidal flow next to the waves. Modified from Sumer and Fredsoe (2002)
- 1470 Fig. 3. Development of the 2D scour profiles with time. The number along the profiles corresponds to
- 1471 evolution in time. Modified from Mao (1986).
- 1472 Fig. 4. Measured equilibrium depth. Data compiled by Sumer and Fredsoe (1990). Modified from Sumer1473 and Fredsoe (2002).
- 1474 Fig. 5. Wake system with (a): current and (b): waves. Modified from Sumer and Fredsoe (2002).

1475 Fig. 6. 2D scour in waves. Data by Lucassen (1984) and Sumer and Fredsoe (2002). Modified from Sumer1476 and Fredsoe (2002).

1477 Fig. 7. Experimental data on wave-current tunnel scour. Open symbols from Lucassen (1984): without

1478 slash: $3 \le KC \le 7.5$. With left-oriented slash: KC =15, right-orientated: KC = 10. Filled symbols from

- 1479 Sumer and Fredsoe (1996): the numbers in the figure corresponds to their experimental values. Modified
- 1480 from Sumer and Fredsoe (2002).
- 1481 **Fig. 8.** Sketch of span shoulders. Modified from Sumer and Fredsoe (2002).
- 1482 Fig. 9. (A-A): The cycle of embedment at the shoulders. (B-B): Sagging and backfilling in the scour hole.
- 1483 Modified from Sumer and Fredsoe (2002).

- 1484 Fig. 10. Time series of excess pressure at different distances z from the original bed. (Reprinted from
- 1485 Sumer et al. 2006c, © ASCE).



Fig. 1. Vortex system around an embedded pipeline exposed to a current.



Fig. 2. Onset of scour beneath a pipe in current and with waves. Data from Sumer and Fredsoe (1991) and Sumer et al. (2001). Squares: $2 \le KC \le 7$, Circles: $7 \le KC \le 15$, Triangles: $15 \le KC \le 30$. The half-filled symbols include tidal flow next to the waves. Modified from Sumer and Fredsoe (2002)



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Fig. 8. Sketch of span shoulders. Modified from Sumer and Fredsoe (2002).



Fig 9. The cycle of embedment at the shoulders (A-A) and sagging and further backfilling in the scour hole (B-B). Modified from Sumer and Fredsoe (2002).



Fig. 10. Time series of excess pressure at different distances z from the original bed. (Reprinted from Sumer et al. 2006c, © ASCE).