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On-Bottom Stability of Submarine Pipelines

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Introduction

The on-bottom stability (also termed as "geotechnical stability") of submarine pipelines mainly involves vertical, lateral, and axial pipeline-seabed interactions under ocean waves and/or current and engineering operating conditions.

Vertical Stability of the Pipeline on and in Seabed

Submarine pipelines that are intended to be buried in the seabed should be checked for possible sinking or floatation in order to satisfy the vertical stability on and in the soil. For the pipelines to be laid on the seabed with low shear or cohesive strength, the bearing capacity of the soil needs to be evaluated. If the seabed soil is, or is likely to be, liquefied under pure waves or combined waves and current, the liquefaction depth should be predicted accurately. Seabed liquefaction can affect both the vertical stability, i.e., sinking and floatation, and the lateral stability of submarine pipelines (see Det Norske Veritas and Germanischer Lloyd 2017).

Ultimate Bearing Capacity

In geotechnical engineering, the ultimate bearing capacity is defined as the theoretical maximum pressure which can be supported without failure by the soil immediately below and adjacent to a foundation. Submarine pipelines can be regarded as shallow foundations installed on the seabed. With reference to a strip footing, three distinct modes of failure have been identified, i.e., general shear failure, local shear failure, and punching shear failure. The evaluation for the ultimate bearing capacity can be considered in terms of plasticity theory, e.g., the slip-line stress field theory, the lower and upper bound theorems in limit analysis (Chen and Liu 1990).

The seabed soil can be essentially assumed to behave as a Tresca material under undrained conditions, or as a Mohr-Coulomb material under drained conditions. Taking into account the effects of the geometric curvature of the pipe and the adhesion or friction at the pipe-soil interface, the slip-line field solutions for the undrained and fully drained conditions can be

W. Cui et al. (eds.), *Encyclopedia of Ocean Engineering*,
https://doi.org/10.1007/978-981-10-6963-5_201-1

derived, respectively (Gao et al. 2013, 2015). A general slip-line field solution for the ultimate bearing capacity of a pipeline on the soil obeying Mohr-Coulomb yield criterion is expressed as (Gao et al. 2015):

$$\frac{P_u}{D\sin\theta} = cN_c + qN_q + (0.5D\gamma'\sin\theta)N_\gamma \quad (1)$$

where P_u is the collapse load for the soil suffering general shear failure (in kN/m); D is the outer diameter of the pipeline (in m); $\theta =$ arccos $(1 - 2e/D)$ is the embedment angle (in degree), e/D is the embedment-to-diameter ratio; "D sin θ" refers to the efficient width of the pipe-soil interface, which is related to the pipe penetration; c is the soil cohesion (in kPa); q is the surcharge pressure (in kPa/m) (note: for $e/D \leq 0.5$, q is set to zero; for $e/D > 0.5$, the pipeline embedment can be treated as $e/D = 0.5$ with an equivalent uniform surcharge pressure $q = (e - 0.5D)\gamma'$, where γ' is the buoyant unit weight of the soil (in kPa/m^3)); and N_c, N_q, and N_γ are the bearing capacity factors for the cohesion, for the distributed load, and for the buoyant weight of soils, respectively. For the clayey seabed under undrained conditions, if neglecting the effects of the pipe geometric curvature ($e/D \rightarrow 0$), the pipe-soil interface adhesion (i.e., the interfacial friction coefficient $\mu = 0$), and the internal friction of the soil (i.e., the angle of internal friction $\varphi = 0$), the slip-line field solution can be degenerated into Prandtl's solution for conventional strip footings, i.e., $N_c = 2 + \pi$. With increasing pipeline embedment, the value of N_c decreases from $N_c = 2 + \pi$ at $e/D \rightarrow 0$ and finally reaches $N_c = 4.0$ at $e/D = 0.5$. It has been indicated that the geometric curvature effect is unneglectable when evaluating the ultimate bearing capacity of submarine pipelines.

Note that the aforementioned solutions were obtained under the assumption that the seabed soil is described with a perfectly plastic stress-strain relationship. Such approximation is applicable for the soils of low compressibility, which might meanwhile correspond to the general shear mode of failure. However, for a very soft clayey seabed, large settlements of the pipeline may occur under its submerged weight and laying disturbances without general shear failure occurring. In such cases, the limiting criterion for bearing capacity should be the maximum allowable settlement.

Seabed Liquefaction

Seabed liquefaction is the phenomenon that the seabed sediment loses a significant part of or all its shear strength under the action of ocean waves or earthquake. Both residual and oscillatory mechanisms for the wave-induced pore pressure response in the seabed have been identified in flume experiments and field observations. The residual liquefaction of the seabed is due to the pore pressure buildup under undrained or partially drained conditions, which usually takes place in fine sands or silty soils under severe wave loading and always combined with currents in the offshore field.

Besides residual liquefaction, the instantaneous liquefaction (also termed as "momentary liquefaction") is particularly prone to occurrence in seabed sediments during severe storms, which is essentially caused by the instantaneous upward seepage within the upper layer of the seabed under wave troughs. In the instantaneously liquefied layer of the seabed, the vertical gradient of excess pore pressure (j_z) should be identical to the buoyant unit weight of the soil (γ'), i.e., the improved criterion for instantaneous liquefaction (Qi and Gao 2018): $j_z \cong \gamma'$. Based on the analytical solution to Biot's consolidation equations for porous seabed response to waves (Yamamoto et al. 1978) and the above criterion for instantaneous liquefaction, the maximum depth of instantaneously liquefied layer (z_L) under wave troughs can be derived (Qi and Gao 2018):

$$z_L = \frac{p_0}{\gamma'} - \frac{1}{\mathrm{Re}\left[\lambda(1-\alpha) + \lambda'\alpha\right]} \quad (2)$$

where p_0 is the amplitude of wave-induced excess pressure at the seabed surface; the operator "Re{}" means to take the real part of the complex variable in the brackets; $\lambda(=2\pi/L)$ is the wave number; L is the wavelength; α and λ' are the complex numbers related to the properties of the seabed and waves (see Yamamoto et al. 1978).

If the pipeline is to be buried in a liquefiable seabed, the designed burial depth should be larger than the maximum depth of instantaneously liquefied layer (see Eq. 2). To keep the buried pipeline vertically stable, the residual liquefaction of the soil around the pipeline needs to be further evaluated. Moreover, under the condition that waves and current coexisting in the field, the seabed liquefaction and the on-bottom stability of the pipeline should be accurately predicted.

Lateral Stability

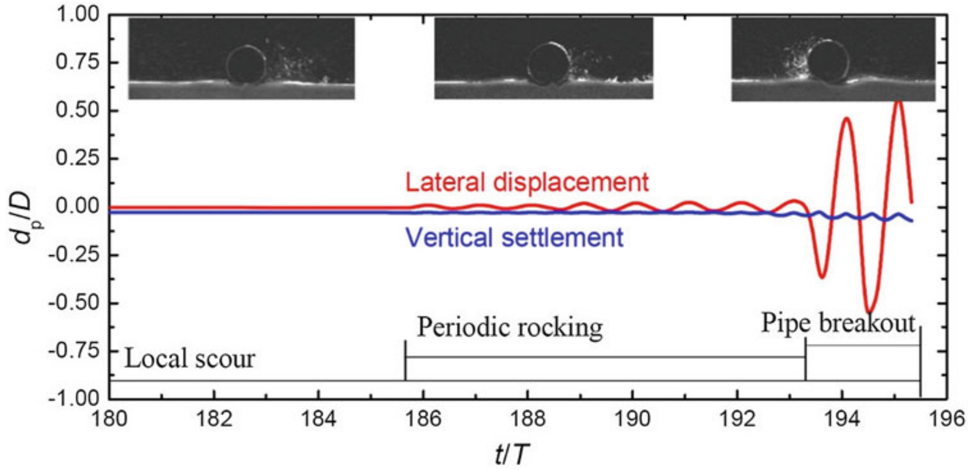
In submarine geological and hydrodynamic environments, the multi-mechanics processes can emerge in the proximity of as-laid pipelines, including shear flow above the seabed, sediment transport along the seabed surface, the excessive pore pressure in the soil, etc. They are generally coupled with each other and have significate influence on the lateral stability of submarine pipelines (Gao 2017).

The triggering mechanisms for the pipeline lateral instability involve not only pipe-soil interactions (Wagner et al. 1989; Zhang et al. 2002; Youssef et al. 2013; Gao et al. 2016) but also flow-pipe-soil coupling process (Gao 2017). Experimental observations (see Fig. 1) with an oscillatory flow tunnel showed that there always exist three characteristic stages during the lateral instability of a pipe shallowly embedded in sands under a storm-like wave loading, i.e., (a) local scour of sands around the pipe, (b) periodic rocking of the pipe with small amplitudes, and (c) pipe breakout from original location (see Gao 2017). In the process of the pipe losing lateral instability, local scour always emerges as an indicator for the flow-pipe-soil coupling effect, which was observed taking place at the rear and front of the pipe. The pipe periodic rocking due to the vortex shedding may increase its penetration into the seabed and further affect the lateral stability. The occurrence of pipeline instability is characterized by a distinct lateral displacement (e.g., $d_p/D>0.5$, where d_p is the pipe lateral displacement, see Fig. 1).

The criteria for pipeline lateral instability are crucial to the on-bottom stability design. Based on similarity analyses for physical modeling experiments, it was found that the controlling non-dimensional parameter of hydrodynamics for pipeline lateral instability is the Froude number ($Fr = U_m/\sqrt{gD}$) and another parameter is the non-dimensional submerged weight of pipelines ($G = W_s/\gamma' D^2$, where U_m is the maximum velocity of wave-induced water particle movement and g is the gravitational acceleration. Both the scaling of the Froude number and that of the Keulegan-Carpenter number ($KC = U_m T/D$, where and T is the wave period) can be concurrently satisfied in the physical modeling with wave flumes. KC number essentially controls the generation and development of vortex around the pipeline under oscillatory flow in waves. A unified formulation of criteria for pipeline lateral instability in waves and currents can be expressed as (Gao et al. 2003)

$$Fr_{cr} = a + b\frac{W_s}{\gamma' D^2} \tag{3}$$

where Fr_{cr} is the critical values of Froude number for pipeline lateral instability. In Eq. (3), the two parameters a and b are relative to the hydrodynamic loads (periodic waves or steady current), the end constraint conditions (anti-rolling or freely laid) of a shallowly embedded pipe, and the soil properties of the seabed. On the basis of the results of a series of experiments, the values of these two parameters were determined as (a, b) = (0.07, 0.62) for the anti-rolling pipes in waves and (a, b) = (0.042, 0.38) for the freely laid pipes in waves ($5 < KC < 20$). But for the freely laid pipes in a steady current ($KC \to \infty$), (a, b) = (0.102, 0.423), indicating the pipes are more stable in currents than in waves due to the inertia effect of wave movements. It should be noted that the above recommended values for the two parameters a and b were based on the results of model tests on a uniform medium sand-bed. The particle size is closely related to sediment transport and further affects the lateral stability of the pipeline.



On-Bottom Stability of Submarine Pipelines, Fig. 1 Characteristic stages during the pipeline losing lateral stability under a storm-like wave loading (Adapted from Gao 2017)

The instability criteria in the unified formulation by Eq. (3) provided alternative expressions to the pipe-soil interaction model by Wagner et al. (1989) for the on-bottom stability of a shallowly embedded pipeline, as addressed by Fredsøe (2016).

Besides the aforementioned lateral instability, submarine pipelines could also be under the threat from tunnel erosion due to the seepage failure of its underneath soil, especially under the action of shear flow near the seabed, which may further bring the spanned pipeline undergo vibrations, i.e., vortex-induced vibrations (Gao 2017). It has been revealed that the two typical scenarios, i.e., the lateral instability of the pipe and the tunnel erosion of the soil, are always competitive between each other in the submarine geological and hydrodynamic environments (Shi and Gao 2018).

Axial Pipe-Soil Interactions

As offshore developments extend into deep waters, the relatively high pressure and high temperature (HPHT) becomes a dominant factor for the pipeline safety. The HPHT pipeline would undergo expansion and contraction during start-up and shutdown cycles in the operating life, which may induce pipeline walking on the seabed. It should be noted that the pipeline walking is not a limit state; nevertheless, the excessive compressive force can lead to severe global buckling of the pipeline and even the failure of associated infrastructures (Randolph and Gourvenec 2011).

The ultimate axial soil resistance (F_{Ru}) to a conventional strip footing with flat bottom is generally linked with the normal pipe-soil contact force (F_N) and the interface friction coefficient (μ), i.e., $F_{Ru} = \mu F_N$. But for submarine pipelines, the assessment of ultimate axial soil resistance becomes much more difficult than that for conventional strip footings. Due to the effect of pipeline curvature, the integrated normal pipe-soil contact force (F_N) would exceed the submerged pipeline weight (W_S), i.e., $F_N = \zeta W_S$, by incorporating a wedging factor (ζ):

$$\zeta = \frac{2\sin\theta_0}{\theta_0 + \sin\theta_0\cos\theta_0} \tag{4a}$$

for clayey soils (White and Randolph 2007) and

$$\zeta = \frac{1 - (2\theta_0/\pi)^2}{\cos\theta_0} \tag{4b}$$

for sandy soils (Shi et al. 2019), respectively. In Eqs. (4a) and (4b), θ_0 is the semi-angle subtended at the pipe-soil contact chord.

The axial soil resistance expressed with an equivalent friction coefficient could span an order of magnitude, which is related to many influential factors, including the soil types, drainage condition, and the pipe roughness. A theoretical framework was developed within a critical-state context using effective stresses, applicable to any degree of drainage in the soil, quantifying the magnitude and duration of excess pore pressures generated near the pipe-soil interface (Randolph et al. 2012). It has been recognized that the axial soil resistance is crucial for assessing the effective axial force along the pipeline and the corresponding global buckling predictions.

Cross-References

► Design of Pipelines and Risers
► Pipeline Soil Interactions

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