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Seabed Liquefaction with Reduction of Soil Strength due to Cyclic Wave Excitation

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Abstract

This study introduces the case of pipelines installed in subsea conditions and buried offshore. Such installations generate pore water pressure under the seabed because of cyclic wave excitation, which is an environmental load, and consistent cyclic wave loading that reduce the soil shear strength of the seabed, possibly leading to liquefaction. Therefore, in view of the liquefaction of the seabed, stability of the subsea pipelines should be examined via calculations using a simple method for buried subsea pipelines and floating structures. Particularly, for studying the possible liquefaction of the seabed in regard to subsea pipelines, high waves of a 10- and 100-year period and the number of occurrences that are affected by the environment within a division cycle of 90 s should be applied. However, when applying significant wave heights (HS), the number of occurrences within a division cycle of 3 h are required to be considered. Furthermore, to research whether dynamic vertical load affect the seabed, mostly a linear wave is used; this is particularly necessary to apply for considering the liquefaction of the seabed in the case of pile structure or subsea pipeline installation.

Keywords: Wavelength, Wave Elevation, Hydrodynamic pressure, Soil liquefaction, Set-up factor, Thixotropy, Shear Strength Ratio, Water depth, Wave height, Wave period

1. Introduction

When installing subsea pipelines or Offshore Fixed structures, the pore pressure of the seabed causes the sand layers to liquefy due to the external cyclic wave vertical pressure load. This generates problems in the stability of the structure, and consequently, maintenance and reinforcement procedures are often required. Particularly, there is heavy ship traffic zone those subsea pipelines that are mainly used for transporting crude Oil or gas throughout the sea or from the shore to the SPM(Single Point Mooring), or there are buried subsea pipelines near offshore fixed platforms. Such subsea pipelines must be maintained due to the seabed liquefaction phenomenon caused by ocean waves. However, owing to such floatation and the stability of subsea pipes is significantly affected. The seabed liquefaction phenomenon occurs not only in sandy layers, but also in clay layers. Consequently, in the present, seabed replacement of engineered material or subsea pipeline reinforcement procedures are required. Thus, a method for guaranteeing the subsea pipeline stability for preventing the soil liquefaction caused by ocean waves shall be considered in design stage.

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2. Interpretation Theory of the Seabed Liquefaction

2.1 Amplitude of Ocean Waves and Hydrodynamic Pressure of the Seabed

To determine the pore pressure of the seabed, it is first necessary to calculate the amplitude of the ocean wave. Thus, in this chapter, the amplitude of the ocean waves will be expressed as that of an Airy wave, which is a linear wave, as shown in Eq. (2.1).

$$\text{Wave amplitude of an airy wave, } \eta = \frac{1}{2} H \ cos\left(\frac{2\pi x}{L} - \frac{2\pi t}{T}\right) \tag{2.1}$$

Generally, the hydrodynamic pressure acting at the seabed varies depending on the amplitude, as shown above in Eq. (2.1). According to the linear wave theory, the hydrodynamic pressure is expressed as shown in Eq. (2.2).

$$p = \left(\frac{\rho_w g H}{2}\right)\left(\frac{1}{cosh(kd)}\right) cos(kx + wt) \tag{2.2}$$

Here, $k = 2\pi/L$, H = wave height *(m)*, T = wave period *(s), t = time (s)*, $\omega = 2\pi/T$, and ρ_w = unit mass of water (*ton/m*^{3}). The wavelength L is expressed as shown in Eq. (2.3).

$$L = \text{ Wavelength} = \frac{gT^2}{2\pi} tanh\left(\frac{2\pi d}{L}\right) \tag{2.3}$$

where $d =$ Water depth (m) and $\text{x} =$ wave path length (m).

2.2 Dynamic Pressure Distribution according to the Seabed Depth

Particularly, during typhoons, the surface of the sea level will generate a hydrodynamic pressure to the seabed at each layer, caused by the large amplitude waves acting on the seabed. Consequently, the vertical, horizontal, and shear hydrodynamic pressure on the depth of seabed can be evaluated. Eq. (2.4) represents the vertical pressure at depth of z under seabed Eq. (2.5) represents the horizontal pressure at depth of z under seabed. and Eq. (2.6) expresses the shear pressure at depth of z under seabed.

Vertical pressure at depth of z under seabed;

$$\sigma_v = p\left(1 + \frac{2\pi z}{L}\right) e^{-2\pi z/L} \tag{2.4}$$

Horizontal pressure at depth of z under seabed

$$\sigma_h = p\left(1 - \frac{2\pi z}{L}\right) e^{-2\pi z/L} \tag{2.5}$$

Shear pressure per soil depth;

$$\tau_{vh} = p\left(\frac{2\pi z}{L}\right) e^{-2\pi z/L} \tag{2.6}$$

Where, p represents the vertical pressure on the seabed.

The following equation expresses the maximum shear stress that acts on the depth of z from seabed due to the hydrodynamic force:

$$\tau_{vh,max} = \frac{\sigma_1 - \sigma_3}{2} = \sqrt{\left(\frac{\sigma_v - \sigma_h}{2}\right)^2 + \tau_{vh}^2} = p_z\left(\frac{2\pi z}{L}\right) e^{-2\pi z/L} \tag{2.7}$$

Table 2.1 Soil Set-up Factor (DNV-RP-E303,2005)

PI	< 25%	25–50%	> 50%
St >3	0.58	0.65	0.65
St <3	0.58	0.65	1.95/St ≤1

PI	10 Days	3 Months
PI <30	1.15/St	1.4/St
30 ≤ PI <50	1.15/St	1.4/St
	0.41 - 0.07(Ip-30)/20	0.55
50 ≤ PI <80	(1.15 + 0.25(PI - 50))/St	1.9/St
	0.34 - 0.16(Ip - 50)/30	0.55 - 0.17(Ip - 50)/30
PI ≥ 80	1.9/St	1.9/St

2.3 Properties of Reduction of the Seabed Shear Strength

Owing to the continuous occurrence of ocean waves on clayey grounds, the undrained shear strength of the soil decreases. In addition, the strength of the early backfilled material, having reduced more than that of the base surface, is affected by the external environment. Thus, the reduction factor of the early undrained shear strength of clayey grounds is defined as the set-up factor, $\alpha = C_t(1/S_{[t)})$, whose value is set as 0.55 when the clayey ground is early backfilled. This value is recommended to be set as 0.8 because during operation, only part of the shear strength is restored, but not fully recovered shear strength. Therefore, the decreased strength of backfilled materials is defined as $S_{u0} = S_u\,\alpha$. However, the Ken Been et al.(2011) recommended the undrained shear strength of early backfilled clay soils to be Su/St or 8 kPa. However, in clay soils, the thixotropy ($1.15C_{[t)} = \text{thixotropy factor}$) must be considered. Eq. (2.8) is a revised equation proposed by Lee (2010), and Eq. (2.9) represents the undrained shear strength of backfilled materials for typical clay soils.

$$S_{u0}^{thixotropy} = \frac{S_u}{S_t} \cdot 10^{-0.07125+0.1103\,log(t/t_0)} \tag{2.8}$$

$$S_{u0}^{initial} = \frac{\sigma_v'}{S_t} \text{(Undrained shear strength of initial backfilled materials)} \tag{2.9}$$

Where, $t_0 = 1\,day,\ t = thixotropy\,days,\ S_u =$ Soil undrained shear strength of an original seabed, $S_t = soil\ sensitivity(q_{u/q_{remold}}),\ and\ \sigma_v' =$ effective vertical stress of backfilled materials. Tables 2.1 and Table 2.2 show the set-up factor depending on the properties of the soil.

Boulanger and Idriss (2006) and Andersen (2009) explained the strength reduction caused by the soil pore pressure generated by the continuous load of external ocean waves through Eq. (2.10).

$$\begin{aligned} \tau_{cyc}/S_{u0} &= 1.3N_{cyc}^{0.115},\ When\ OCR\ < 1.0 \\ \tau_{cyc}/S_{u0} &= 1.19N_{cyc}^{0.108},\ When\ OCR\ \geq 3.0 \\ \tau_{cyc}/S_{u0} &= 1.18N_{cyc}^{0.146},\ When\ 1 \leq OCR\ < 3.0 \end{aligned} \tag{2.10}$$

Here, N_{cyc} is the wave acting numbers.

The shear strength caused by repetitive ocean waves in clayey soils, $\tau_{s,zi}$, is expressed as shown in Eq. (2.11).

$$\tau_{s,zi} = C_{2D}\left(\frac{\tau_{cyc}}{S_{u0}}\right)\left(\frac{S_{u0}}{\sigma_{v,zi}'}\right)\sigma_{v,zi}' \tag{2.11}$$

Table 2.2 Soil Set-up Factor(1/α) (Rausche et al. (1996))

Predominant Soil Type	Range of Soil Set-up Factor (1/α)	Recommended Soil Set-up Factor (1/α)	Number of Sites
Clay	1.2–5.5	2	7 (15%)
Silty Clay	1.0–2.0	1	10 (22%)
Silt	1.5–5.0	1.5	2 (4%)
Sandy Clay	1.0–6.0	1.5	13 (28%)
Sandy Silt	1.2–2.0	1.2	8 (18%)
Fine Sand	1.2–2.0	1.2	2 (4%)
Sand	0.8–2.0	1	3 (7%)
Sand Gravel	1.2–2.0	1	1 (2%)

Here, $C_{2D} = 0.96$ (factor of multi-direction).

Following figure 2.1 shows the experimental value of the cycle shear strength ratio of clayey soils depending on the OCR(Over Consolidation Ratio) value.

In sandy Layer, the set-up factor, α, is 0.83 after the first backfilling, but it reaches 1.0 during operation. The internal friction angle of sandy grounds after the first backfilling is $\varphi_0 = \varphi\alpha$. Therefore, the initial SPT, N_{30} (Standard Penetration Test for Number of counter blow for 30cm penetration)is expressed as shown in Eq. (2.12).

$$SPT,\ N_{30},\ N_0 = (\varphi_0 - 20)^2/12 \text{ (Dunham (1954))} \tag{2.12}$$

Where,.

Furthermore, to obtain the value of SPT, N_{60}, the correction factor $C_N = \sqrt{\frac{100}{\sigma_v'}} \leq 1.7$ is applied. Then, $\sigma_v' = (\rho_s - \rho_w) g\, d,\ when\ d\ =\ trench\ depth$, and $\sigma_{v(zi)}' = \sigma_{0(zi)}' \frac{(1+2K_0)}{3}$.

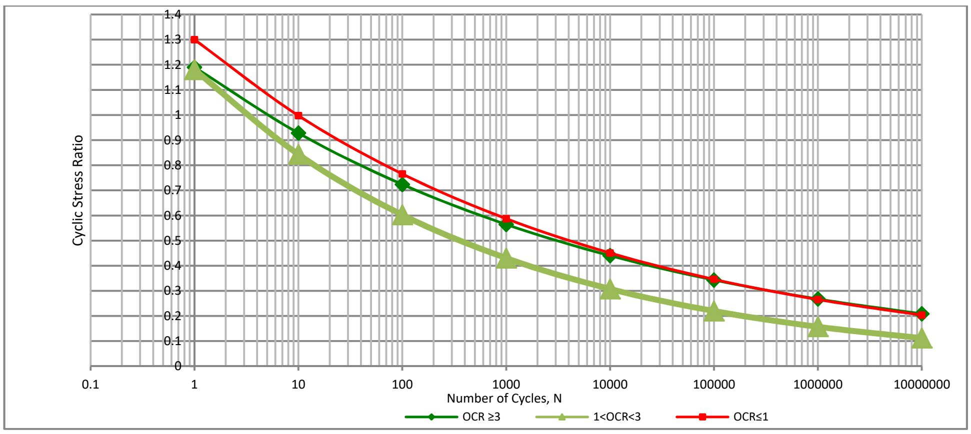

Figure 2.1 Cycle shear strength ratio of clayey soils in relation to the OCR value τ_{cyc}/σ_v'

After modifying SPT, N_{30} to SPT, N_{60}, the following equation is obtained:

$$SPT,\ N_{1(60)} = 15 + 0.5(N_0 C_N - 15) \tag{2.13}$$

In the work by Seed (1979) and Skempton (1986), the relative density of sandy soils is $Dr = 14\sqrt{N_{1(60)}}$ %. Thus, according to Meyerhof (1948), the internal friction angle of initial sandy soil is $\varphi_{60} = 15Dr + 28$. Here Dr is a general decimal. Thus, depending on the wave period, the soil cyclic shear strength ratio generated by a repetitive ocean wave load, in accordance with the experiments of Seed (1979), Mulilis et al. (1975) and Castro (1977), is as follows:

$$\tau_{cyc}/\sigma'_{v(zi)} = \sigma'_{v(zi)}\tan(\varphi_{cyc}) = 1.63238(0.0009Dr(\%) + 0.0255)$$
$$(3 - 2\sin(\varphi_{60})\,N_{cyc}^{-0.0709}\ \text{When T} \le 2.5\ \text{s} \tag{2.14}$$

$$\tau_{cyc}/\sigma'_{v(zi)} = \sigma'_{v(zi)}\tan(\varphi_{cyc}) = 1.63238(0.0012Dr(\%) + 0.0400)$$
$$(3 - 2\sin(\varphi_{60})\,N_{cyc}^{-0.0709}\ When\ 7.5 \le T > 2.5\ s$$
$$\tau_{cyc}/\sigma'_{v(zi)} = \sigma'_{v(zi)}\tan(\varphi_{cyc}) = 1.63238(0.0013Dr(\%) + 0.0439)$$
$$(3 - 2\sin(\varphi_{60})\,N_{cyc}^{-0.0709}\ When\ T > 7.5\ s$$

Thus, in sandy Layers that are influenced by repetitive ocean wave loads, the internal friction angle caused by the pore pressure decreases, and it can be expressed as shown in Eq. (2.15).

$$\varphi_{cyc} = Atan\left[tan\left(\tau_{cyc}/\sigma'_{v,zi}\right)\right] \tag{2.15}$$

Thus, the shear strength resulting from repetitive ocean wave loads in sandy Layers can be expressed as shown in Eq. (2.16).

$$\tau_{s,zi} = \sigma'_{v,zi}\left[tan\left(\tau_{cyc}/\sigma'_{v,zi}\right)\right] \tag{2.16}$$

The above liquefaction cyclic soil shear strength, $\tau_{s,zi}$, is the experimental shear strength considering the pore water pressure that acts underground.

Therefore, the repetitive shear strength ratio resulting from the relative density and effective surface load of sandy soils and in relation to the external ocean wave period (sec.) is shown in Figure 2.2 and Figure 2.3.

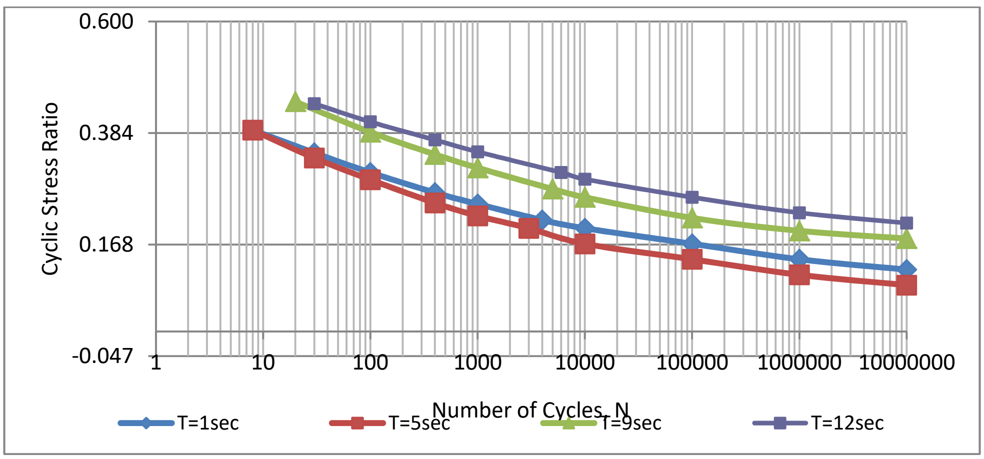

Figure 2.2 Cyclic soil shear strength ratio when D_R= 55% of relative density and σ'_v = *50 kPa* of effective soil overburden stress

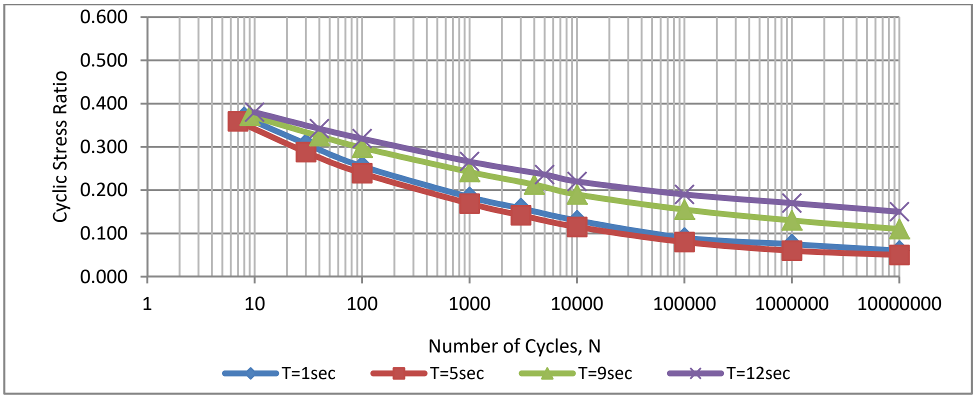

Figure 2.3 Cyclic soil Strength ratio when D_R= 35% of relative density and σ'_v = *50 kPa* of effective soil overburden stress

3. Liquefaction Stress Ratio of Seabed Grounds

The soil liquefaction is determined by the liquefaction stress ratio of seabed grounds. The corresponding shear strength ratio can be expressed as shown in Eq. (3.1) as the ratio of the above-mentioned Eq. (2.7), the shear strength of the clayey soil (Eq. 2.11) and of sandy Layers (Eq. 2.16).

$$SR = \frac{\tau_{s,zi}}{\tau_{vh(max),zi}} > 1.0 \tag{3.1}$$

4. Conclusion

The object of this study was to evaluate the soil liquefaction level caused by external ocean wave loads in the NSRP(Nghi Son Refinery Project) subsea pipeline project in Vietnam. In this project, the seabed liquefaction phenomenon did not occur as the soil Layer was mainly composed of clay.

Thus, the seabed in this site is composed of clayey silt or clay layer, the seabed liquefaction phenomenon cannot occur due to external ocean waves. However, it was concluded that, if seabed soil layer is composition of the silt or fine sand, the seabed must be always examined in advance.

References

Abbas Moustafa, 2012, "Advances in Geotechnical Earthquake Engineering- Soil Liquefaction and Deismic Safety of Dan and Monuments", ISBN 978-953-0025-6, pp. 65–80.

Chih-Hsin Vhang, Leen-Kwei Chien, et al., 2004, "3-D Liquefaction Potential analysis of Seabed at near-shore Area", Journal of Marine Science and Technology, Vol. 12, No. 3 pp. 141–151.

DNV-RP-E303, "Geotechnical Design and Installation of Suction Anchor in Clay", pp. 12–13.

FHWA NHI-05-042, 2006, “Design and Construction of Driven Pile Foundations" pp. 9–179.

Lien-kwei Chien, C.H.Chang, et al., 2003, "The Mechanism Analysis of Wave-induced Seabed Liquefaction", Proceedings of the 13th International Offshore and Polar Engineering Conference, pp. 445–452

Shuh-Gi Chen and Tain-Shyang Chang, 1995, "Simplified Procedure for Evaluating Soil Liquefaction Characteristics, Journal of Marine Science and Technology, Vol. 3, No. 1 pp. 35–42.

T. L. Youd and I. M. Idriss, 2001, "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshop on Evaluation of Liquefaction Resistance of Soils", Journal of Geotechnical and Geo-environmental Engineering, Vol 6. pp. 297–313.

Wonje Lee, Daehyeon Kim, et al., 2010, "Set up of driven piles in layered soil", Japanese Geotechnical Society, Vol. 50, No. 5, pp. 585–598.