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Stability of an underwater trench in marine clay under ocean wave impact

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ABSTRACT: A long section of the immersed tunnel of the Busan-Geoje Fixed Link in South Korea is to be constructed in a trench in soft marine clay. The site is exposed to large typhoon waves and the trench is to be left open for approximately one year. The trench profile was chosen as a balance between excavated volume (costs) and risk of slope instability under large waves. To evaluate this risk, far-shore wave data have been transformed into near-shore wave conditions by means of numerical modelling and wave flume tests have been carried. Potentially critical waves have been identified and their impact has been analysed by means of coupled hydro-mechanical numerical simulations. Based on the strength reduction method, safety factors for the slope stability during wave impact have been determined. The results and the consequences for the economic design of such a trench subject to large waves are discussed.

1 INTRODUCTION

The Busan-Geoje Fixed Link between South Korea's second largest city Busan and the island of Geoje consists of a 3.2 km long immersed tunnel (Odgaard et al. 2006), two rock tunnels and two 1.7 and 1.9 km long cable-stayed bridges. About 2.2 km of the immersed tunnel is constructed in a 12 to 15 m deep trench in soft marine clay at water depths between 20 and 50 m. The area is prone to frequent raids of typhoons and the trench is dredged approximately 1 year before construction of the tunnel. When the tunnel is placed in the trench, the lower half of the trench is filled with tunnel protection material (backfill). It was therefore necessary to design the trench profile as a balance between the excavated volume, i.e. construction costs on the one hand and the risk of slope failure under wave impact within a 1 years period on the other hand.

This paper discusses in Section 2, how the wave characteristics at the site have been determined and how the most critical waves with regard to trench slope stability have been identified. Based on the geotechnical characterization of the soft marine clay presented in Section 3, the impact of the critical waves on the trench is investigated by means of coupled hydro-mechanical numerical simulations (Section 4). Strength reduction analyses are made in the simulations in regular intervals of one second to determine the failure modes and factors of safety (Dawson et al. 1999). The last part of the paper contains a discussion of the results and general conclusions.

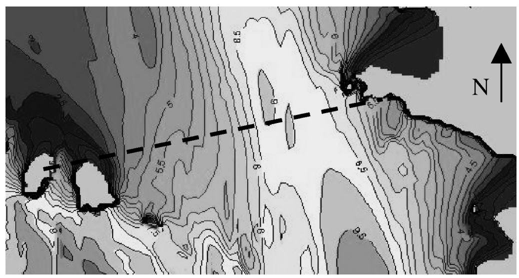

Figure 1. Numerical wave modelling: Calculated significant wave heights for a 10,000 years return period. The tunnel is indicated by the dashed line.

2 DERIVATION OF DESIGN WAVES

2.1 *Numerical wave modelling*

Based on a statistical analysis of wave and wind data in the area, the extreme far-shore boundary waves and extreme wind speeds have been derived. From these input data, the wave conditions at the location of the tunnel have been determined by means of numerical wave modelling with the program MIKE 21 (DHI Water & Environment).

The obtained maximum significant wave heights along the tunnel alignment are 6.2 m for a return period of 10 years, 8.0 m for a return period of 100 years and 9.2 m for the 10,000 years wave event (Figure 1). The large waves come from southerly direction with a wave velocity of approximately 13 m/s and pass the tunnel

Figure 2. Effect of typhoon waves on a breakwater in Busan (typhoon Maemi, 12.9.2003): (a) before and (b) after the storm.

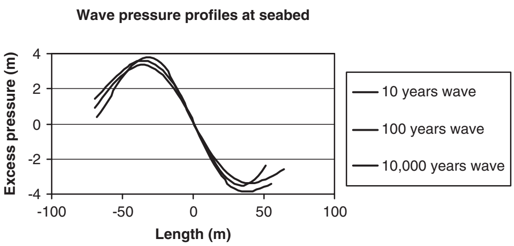

Figure 3. Most critical seabed pressure profiles for the trench stability determined from the wave flume tests.

trench almost perpendicularly. The potential devastating effect of such large waves is illustrated for a breakwater in Figure 2.

2.2 *Wave flume model tests*

Based on the results of the numerical wave modelling, wave flume tests have been carried out for tunnel element TE7, which is subject to the highest waves at a relatively shallow water depth of 20 m. Each test consisted of a series of several hundreds of waves. From the pressure measurements in the tests, the seabed pressure profiles with the largest pressure difference across the 32 m wide trench slopes have been identified. These pressure profiles shown in Figure 3 are considered to be most critical for the slope stability. It should be noted that due to hydrodynamics, the seabed pressures under waves do not simply correspond to the physical wave height above.

3 GEOTECHNICAL PROPERTIES

Extensive ground investigations and laboratory tests have been carried out for the project (Steenfelt et al.

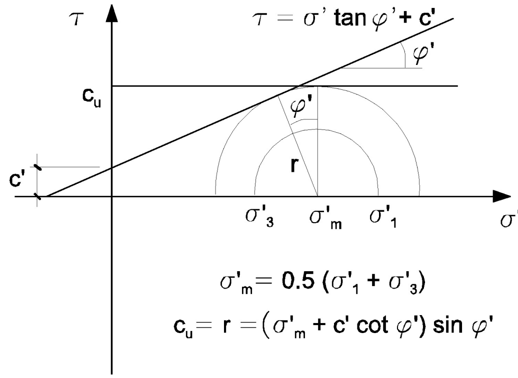

Figure 4. Represenation of undrained strength in the numerical model with the Mohr-Coulomb model based on effective strength parameters.

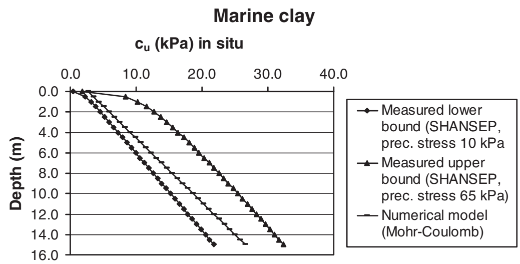

Figure 5. Comparison between measured and modelled undrained strength of the marine clay for in situ conditions.

2008). The undrained strength of the clay has been assessed from 30 cone penetration tests along the alignment and can be described according to the SHANSEP approach (Steenfelt & Foged 1992) as

$$c_u = 0.28\sigma'_{v0}\left(\frac{\sigma'_{v0}+\Delta\sigma}{\sigma'_{v0}}\right)^{0.76} \quad (1)$$

with the preconsolidation stress $\Delta\sigma$ of the marine clay typically ranging between 10 and 30, max. 65 kPa. The effective strength parameters $\varphi' = 25°$, $c' = 3$ kPa have been derived from 19 consolidated undrained triaxial tests. In the numerical model, the material strength is described with the Mohr-Coulomb model. Due to the fact that the clay is only slightly overconsolidated, the Mohr-Coulomb model can also be used without modification to correctly model the undrained strength in case of undrained conditions (Figure 4 and Figure 5).

The stiffness properties and permeabilities have been derived from oedometer tests. The material parameters are summarised in Table 1.

Table 1. Material parameters for the numerical simulations.

	Marine clay	Alluvium
Material model	Mohr-Coulomb	Mohr-Coulomb
γ_{sat} (kN/m^3)	14.7	20
K_0 (-)	0.6	0.426
E (kPa)	8000	50000
ν (-)	0.2	0.25
φ' (°)	25	35
c′(kPa)	3	0
k (m/s)	$1 \cdot 10^{-9}$	$1 \cdot 10^{-5}$

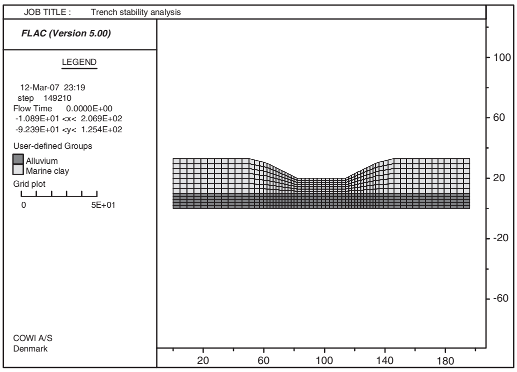

Figure 6. FLAC model of the trench.

4 NUMERICAL SIMULATION OF WAVE IMPACT

4.1 *Model description*

The effect of waves on the trench is modelled by means of numerical simulations using the finite difference program FLAC (Itasca 2005). The two-dimensional, plane-strain model is shown in Figure 6.

The vertical boundaries of the model are fixed in horizontal direction, while the bottom of the model is fixed both in horizontal and vertical direction. The vertical boundaries and the bottom are assumed to be impermeable.

Soil stresses and pore water pressures are initialised according to the self-weight of the soils, $K_0 = 1 - \sin\varphi'$ and a still water table at + 63 m model coordinate (still water pressures p_{still}).

In a first step, the trench excavation is modelled either as a drained or undrained process. This is done to allow for modelling of wave impact shortly after trench excavation (undrained excavation modelling) and wave impact 1 year after trench excavation, where the excess pore pressures in the clay in the relevant area next to the trench slopes will almost completely be dissipated (drained excavation modelling).

Afterwards, wave impact is modelled by means of coupled hydro-mechanical analyses with transient

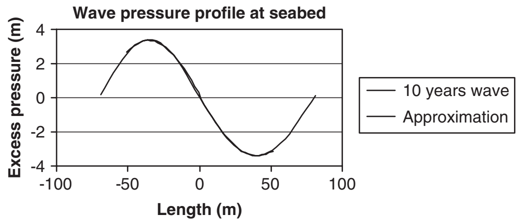

Figure 7. Approximation of the 10 years wave pressure profile in the FLAC model ($A_1 = 3.4$ m, $L_1 = 70$ m, $A_2 = 3.4$ m, $L_2 = 80$ m).

hydraulic and mechanical boundary conditions along the seabed and the trench surfaces. The wave pressure profile is described by means of two sine curves, one for the wave crest and one for the wave trough. This allows to consider different shapes of the wave crest and wave trough according to

$$p = A_1 \sin\left(\frac{\pi}{L_1} x\right) + p_{still} \quad \text{(wave crest)} \tag{2}$$

$$p = -A_2 \sin\left(\frac{\pi}{L_2} x\right) + p_{still} \quad \text{(wave trough)} \tag{3}$$

The described approach is able to model the wave pressure profiles quite accurately, as illustrated for the 10 years wave in Figure 7.

Based on Eqs. (2) and (3), the wave pressures are evaluated for each point in time at each gridpoint along the seabed and the trench surfaces and the corresponding total normal stress and pore water pressure boundary conditions

$$\sigma_n = -p \quad \text{and} \quad p_w = p \tag{4}$$

are applied. It should be noted that water pressures are defined positive, while compressive stresses are defined negative. According to the principle of effective stresses, these boundary conditions imply that the effective normal stresses at the seabed and the trench surfaces remain 0. Based on a linear variation of the boundary conditions along each zone edge, the whole wave pressure profile is approximated as a piecewise linear function with the correct values at each gridpoint. The model is defined such that a wave train with an arbitrary number of waves can be simulated. As a compromise between computational effort and concern about a possible change in safeties from wave to wave, a wave train with 2 waves is considered in the basic simulations. Due to the fact that each of the investigated large waves passes the trench within a period of approximately 6 seconds, the simplified assumption of quasi-static conditions in the simulation appears to be adequate. The compressibility of the pore water is

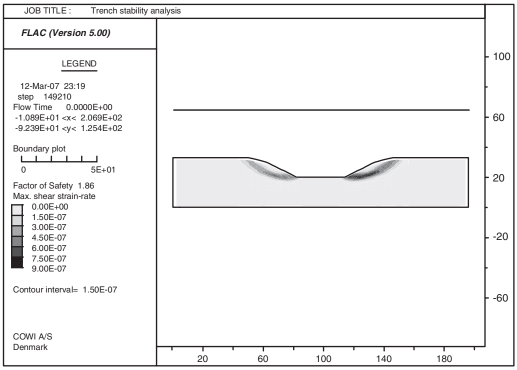

Figure 8. Predicted failure mechanism without wave impact after full consolidation of the trench excavation (drained conditions): Safety factor 1.86.

taken into account by a bulk modulus $K_w = 2 \cdot 10^9$ Pa (pure water), while the compressibility of the soil grains is neglected. Failure mechanisms and safety factors are determined in time intervals of 1 second during wave impact by means of strength reduction analyses (Dawson et al. 1999).

4.2 *Results*

Figure 8 shows the result of a safety analysis without wave impact after full consolidation of the trench excavation. The failure mechanism is illustrated by means of maximum shear strain rates at the end of the strength reduction analysis.

The FLAC result can be verified by comparison with other solutions (Table 2 and Figure 9). The drained results can be compared with a traditional limit equilibrium solution and a finite element stress based solution obtained with the program SLOPE/W (GEO-SLOPE) as well as with a strength reduction analysis with the finite element program PLAXIS (Brinkgreve & Bakker 1991). In the case of undrained conditions, a FE calculation is needed with all programs in order to correctly predict the effective stresses and excess pore pressures. The shapes of the slip surfaces and the safety factors show satisfying agreement in all cases. It is obvious that the safety factors decrease with consolidation. The reason is that the excavation represents an unloading process leading to a time-dependent reduction in effective vertical stresses, which are crucial for the soil strength. According to the Mohr-Coulomb failure criterion the shear strength is linearly dependent on the effective normal stress. It is estimated that after one year, the conditions in the clay in the critical area close to the slope surfaces may be nearly drained.

Figure 10 shows the evolution of the computed safety factors during wave impact after full consolidation of the trench excavation. It can be observed that the safety factors show a large variation in the range between 0.91 and 4.12 during passage of the waves.

The safety factors exceed 1.86 obtained for still water when a wave crest is above the trench. Then, increased pressures act along the bottom of the trench and decrease along the trench slopes towards the crown points of the trench (Figure 11). This pressure distribution in the trench counteracts the destabilizing gravity forces, thus leading to increased safety factors of up to 4.12.

On the other hand, a wave trough above the trench causes high pressures on the seabed beside the trench which decrease down along the trench slopes (Figure 12). This pressure distribution adds to the gravity forces, thus leading to decreased safety factors of down to 0.91. The prediction of safety factors smaller than 1 for the 100 and 10,000 years wave has been facilitated by scaling the clay strength (tan φ' and c$'$) up and scaling the obtained safety factors down.

According to Table 3, the difference in minimum safety factors between the 10, 100 and 10,000 years

Table 2. Comparison of safety factors without wave impact.

	FLAC	GEO-SLOPE	PLAXIS
Undrained excavation	2.67	2.62[2]	2.64
Drained excavation	1.86	1.83[1], 1.83[2]	1.87

[1] General limit equilibrium solution
[2] Solution based on finite element stresses

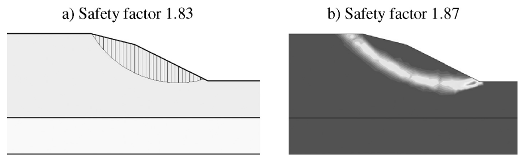

Figure 9. Illustration of results for drained conditions: (a) Traditional limit equilibrium solution (SLOPE/W), (b) Finite element strength reduction solution (PLAXIS).

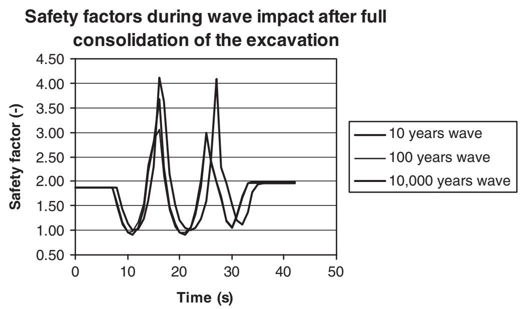

Figure 10. Computed safety factors during wave impact after full consolidation of the trench excavation.

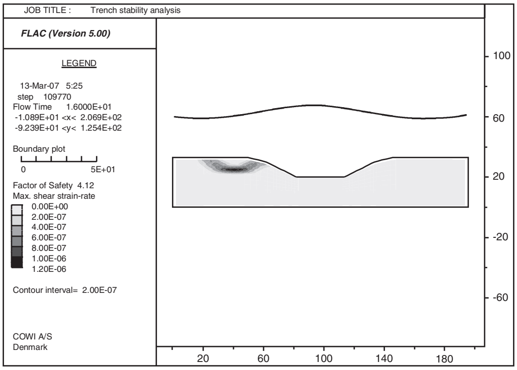

Figure 11. 10 years wave, t = 16 s: Illustration of wave pressure profile and predicted failure mechanism with the largest safety factor of 4.12 for wave impact after full consolidation of the trench excavation.

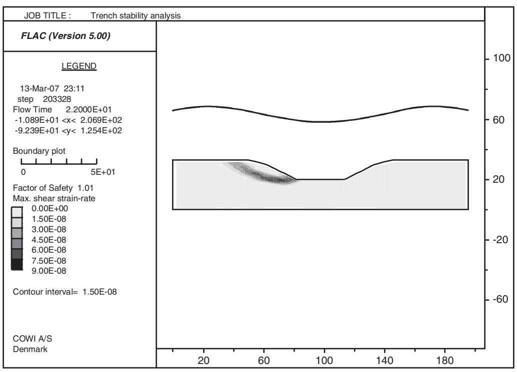

Figure 12. 10 years wave, t = 22 s: Illustration of wave pressure profile and predicted failure mechanism with the smallest safety factor of 1.01 for wave impact after full consolidation of the trench excavation.

Table 3. Summary of minimum safety factors.

	Wave impact shortly after trench excavation	Wave impact after full consolidation of the trench excavation
10 years wave	1.38	1.01
100 years wave	1.29	0.95
10,000 years wave	1.24	0.91

waves is relatively small and the safety factors for wave impact after full consolidation of the trench excavation are approximately 36% lower than the corresponding safety factors for wave impact shortly after trench excavation.

Figure 10 shows the safety factors for wave impact after full consolidation of the excavation (drained modelling of the excavation). Graphs with the same general pattern, but with larger safety factors are obtained for wave impact shortly after excavation of the trench (undrained modelling of the excavation).

It may be concluded from the calculated safety factors that failure of the slopes may occur if these extreme waves would pass the trench after 1 year consolidation. However, it can be argued that sliding of slopes is a dynamic process which takes place in the range of at least several seconds. Figure 11 and Figure 12 illustrate that the maximum and minimum safety factors are only a few seconds apart. Even if the exceptional 100 and 10,000 years waves would pass the open trench after 1 year consolidation, larger sliding movements may be prevented by the fact that critical safety factors occur only within about 1 or 2 seconds. Although water wave impact is a slower phenomenon, the situation may be compared with earthquake impact where the exceedance of yield accelerations in shorter periods induces some irreversible plastic deformations which do not necessarily lead to complete failure. An estimate of expected sliding movements during wave impact could be obtained e.g. by means of dynamic coupled hydro-mechanical simulations with the dynamic version of FLAC.

A simulation of the 10 years wave event with $K_w = 2 \cdot 10^7$ Pa (representing pore water with a high content of dissolved/entrapped air) yields a minimum safety factor of 0.99 for wave impact after full consolidation of the excavation and 1.35 for wave impact shortly after excavation of the trench. These safeties for highly compressible pore water are slightly smaller than the corresponding safeties of 1.01 and 1.38 for the basic case. The influence of pore water compressibility on the behaviour of different soil structures under draw down and wave impact has been investigated in detail e.g. by Köhler (2000).

No perceptible change in safety factors can be observed between the first and second wave in the simulations (Figure 10). This has further been confirmed by a simulation with a wave train of 6 waves.

5 DISCUSSION AND CONCLUSIONS

Due to frequent raids of typhoons in the area, it was necessary to consider the possible effect of large waves in the design of the 2.2 km long and 12 to 15 m deep trench for the immersed tunnel of the Busan – Geoje Fixed Link in South Korea. Safety factors for the trench stability under extreme waves with return periods of 10, 100 and 10,000 years have been determined by means of numerical simulations. The minimum values have been found to be between 1.24 (10,000 years return period) and 1.38 (10 years return period) for wave impact shortly after trench excavation and between 0.91 (10,000 years return period) and 1.01

(10 years return period) for wave impact at the end of the construction period (after consolidation of the trench excavation). These safety factors have been considered to be acceptable for the following reasons:

The investigated situation is a temporary construction phase, where the trench is open for about 1 year. Under these circumstances, the 10 years wave event is considered to be the relevant design case.

Although the minimum safety factors for the 100 and 10,000 years waves are slightly below 1 (Table 3), failure is unlikely even under these extreme waves due to the fact that the minimum safety factors occur within only 1 or 2 seconds (Figure 10). This period is considered to be too short for complete failure to occur. High strain rates would be required for failure to develop and it is a well-established fact that the strength of clay is actually higher for high strain rates. Minimum and maximum safety factors are only a few seconds apart (Figure 10).

Failure of the trench slopes represents a financial risk and would require a clean-up afterwards, but would neither affect lifes nor the technical success of the project. The costs for excavation of a trench in offshore conditions at water depths between 20 and 50 m are high and should be limited to a minimum.

6 PROJECT PROGRESS

The excavation of the trench began in July 2006 and was finished in September 2006. Busan only experienced a minor tropical storm in summer 2006. The production of the soil improvement (cement deep mixing) of the marine clay in the trench for the foundation of the tunnel began in November 2006 and has been finished in May 2007. The offshore construction of the tunnel is scheduled to start at the end of 2007 and finish in 2010.

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