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Including the Influence of Waves in the Overall Slope Stability Analysis of Rubble Mound Breakwaters

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Abstract. An offshore breakwater is designed for the construction of a LNG-terminal. For the slope stability analysis of the rubble mound breakwater the existing and the extreme wave climate are considered. Pore water pressure variations exist in the breakwater and its permeable foundation. A wave trough combined with the moment of maximum wave run-up results in a decrease and increase of the pore water pressure, respectively. Therefore, the wave actions have on overall effect on the slope stability of the breakwater. To include the wave actions in the slope stability analysis a simplified method is used. For the slope stability analysis, a specific piezometric line is determined. This piezometric line consists of a wave profile and the profile of wave run-up. The slope stability analysis are performed with GEO-SLOPE/W 2007. For the geotechnical design of the breakwater load cases of extreme and normal waves combined with, respectively, extreme and normal water levels are analysed. All the load cases which included the wave actions result in lower stability safety factors than the load cases with only still water levels. Therefore the wave actions are the determining load case for the geotechnical stability of the breakwater and it should be studied in detail.

Keywords: breakwater, geotechnical design, pore water pressure, slope stability, wave profile, wave run-up.

Conference topic: Design experiences and theoretical solutions.

Introduction

The design of a breakwater consists mainly of two parts: the hydraulic design and the geotechnical design. For the hydraulic design of the breakwater normal and extreme wave climate conditions are considered to determine the overall design of the breakwater, i.e. crest height and width, slope of the breakwater, the type of armour layer, the toe structure, etc. The geotechnical design mainly consists of the breakwater foundation design and the slope stability analysis. The slope stability analysis is based on normal and extreme (tidal) water levels. The water levels are considered hydrostatic. In contrast with the hydraulic design, the wave climate (despite its importance) is not always considered for the geotechnical design.

The presence of a wave in front of the breakwater results in a decrease of the pore water pressure in the permeable foundation in front of the breakwater. Due to the uprush of waves on the breakwater structure, the pore water pressure increases in the permeable core of the breakwater and in the permeable foundation under the breakwater. A theoretical model of pore water pressure variations in the soil in front of a breakwater was presented by De Rouck and Van Damme (1996). This theoretical model is based on extensive studies and research performed for the design of the rubble mound breakwater in Zeebrugge, Belgium. For the pore water pressure oscillations in the permeable breakwater core an improved calculation model is more recently presented by Vanneste and Troch (2012). The improved calculation method is based on large scale model tests in wave flumes. Both methods of De Rouck and Van Damme (1996) and Vanneste and Troch (2012) are considered by Shafieefar and Fakher (2015) to study the influence of wave induced pore pressure on slope stability analysis of breakwaters. Shafieefar and Fakher (2015) conclude that for wave heights larger than 1.5 m and water depths smaller than 12 m, the effects of excess pore water pressure should be included in the slope stability analysis.

For the study presented in this paper, a simplified method compared to the one of De Rouck and Van Damme (1996) and Vanneste and Troch (2012) is used. The effect of waves and wave run-up is considered by applying a specific piezometric line. The piezometric line consists of a wave profile and a profile of wave run-up. A similar method was considered by De Rouck *et al.* (2010, 2012) for the geotechnical design of the breakwater in Ostend, Belgium.

This paper presents the slope stability analysis performed for the construction of a LNG-terminal. The piezometric line is given as an input for the slope stability analysis. The slope stability analysis are performed with the geotechnical software GEO-SLOPE/W 2007, which uses limit equilibrium formulations. For the slope stability analysis presented in this paper the the Morgenstern-Price theory is considered.

The paper describes first the design conditions, i.e. water levels, wave climate and soil conditions. Then the definition of the wave profile and run-up profile in front of the breakwater are discussed. This is followed by a discussion of the piezometric line in the breakwater. At the end the conclusion are given.

Design conditions

In the following sections the design conditions are discussed.

Water level

The normal and extreme design water levels considered for the geotechnical design are presented in Table 1 and Table 2, respectively. The normal water levels are considered as permanent load cases for the geotechnical stability calculations. The extreme design water levels are considered as accidental water levels for the geotechnical stability calculations. Both low and high tidal water levels are considered. For the extreme water levels a return period of 100 year is considered. This is in agreement with the considered return period for the hydraulic design of the breakwater.

The average level of the seabed is located at –6 m CW. This results in an average normal and extreme water depth at the project area varying between 5 m and 10.3 m.

De Rouck (1991) investigated the most critical moment for the breakwater slope stability when both wave actions and tidal cycles are present. This most critical moment combines the most unfavourable point in time of the tidal cycle with the most unfavourable moment of the wave period. De Rouck and Van Damme (1996) indicated that, based on thorough investigations, for breakwaters constructed in shallow water the most critical situation exists at high water at the moment of maximum wave run-up. For transitional water depths the most critical situation will be at the moment of maximum wave run-up with either low water, mean water or high water (De Rouck, Van Damme 1996). Shallow water is defined as a ratio between the water depth and the wave length smaller than 1/20 (Demirbilek, Vincent 2002). For transitional water depths the ratio between the water depth and the wave length is situated between 1/2 and 1/20 (Demirbilek, Vincent 2002).

The studied breakwater is situated in transitional water depths at the project area. Therefore low, mean and high water levels are considered for the normal water levels.

Table 1. Design normal water levels

Level	[m CW]
Highest Astronomical Tide (HAT)	+1.44
Meas Sea level (MSL)	+1.00
Lowest Astronomical Tide (LAT)	+0.46

Table 2. Design extreme water levels (100 year return period)

Level	[m CW]
High water level	+4.30
Low water level	–1.00

Wave conditions

The considered wave heights are presented in Table 3. For the extreme wave conditions the wave height and period varies along the entire breakwater longitudinal axes. Therefore a range of considered wave heights and periods is given.

Table 3. Wave conditions

Wave condition	Wave height [m]	Wave period [s]
Normal wave conditions	1.6	10.0
Extreme wave conditions – high water level	3.6–3.9	9.0
Extreme wave conditions – low water level	3.1–3.7	7.0–7.2

Soil conditions

At the location of the breakwater construction, the first 16 m to 22 m depth of soil consists of very soft to medium soft soil. This soft to medium soft soil is clayey material. For the geotechnical stability of the breakwater it is necessary to improve the foundation of the breakwater by replacing part of the soft to medium soft soil with sand (Tavallali, Mollaert 2016). The sand used for the soil replacement is silica sand with shells.

Definition piezometric line

In order to take into account the wave actions for the geotechnical calculations a wave trough is considered in front of the breakwater. The wave trough is combined with the moment of maximum wave run-up. In the permeable core of the breakwater, the water level is first considered constant till the centre of the breakwater and then decreases linearly towards the hydrostatic level at the rear side of the breakwater. A typical profile of the considered piezometric line is presented in Figure 1.

For the definition of the piezometric line the following items should be considered:

- The wave profile which determines the wave trough in front of the breakwater;
- The wave run-up which determines the highest water level in the breakwater. This point of maximum wave run-up has to be connected to the wave trough profile. A linear connection is assumed;
- In the breakwater a constant water level till the centre of the breakwater is assumed. Next the water level in the breakwater has to decrease towards the hydrostatic water level at the rear side of the breakwater. A linear decrease is assumed.

Above items are discussed in the next sections.

Wave profile

For the wave profile the linear Airy wave theory is considered (Demirbilek, Vincent 2002):

$$\eta(x,t) = \frac{H_s}{2}\cos\left(\frac{2\pi x}{L} - \frac{2\pi t}{T_p}\right), \quad (1)$$

where: η – surface elevation of the wave profile [m]; x – distance variable of the wave profile [m]; t – time variable of the wave profile [s]; H_s – significant wave height [m]; L – wave length [m]; T_p – peak period [s].

Considering the linear Airy wave theory is a simplification as in shallow waters and close to structures more complex wave models are more accurate.

For the subsurface pressure under a wave normally a pressure response factor should be considered (Demirbilek, Vincent 2002). The pressure under a wave consists of two components: (1) dynamic pressure (due to acceleration) and (2) a static pressure. With the pressure response factor the dynamic component of the pressure under a wave is taken into account. This pressure response factor varies along the wave and with water depths resulting in a varying water pressure along the wave and with the water depth. For a pressure at the sea bottom under the wave trough, the difference between including and not including the pressure factor is maximum 5%. This is based on checks performed for all the load cases considered for this project. Therefore, in order to simplify the input in GEO-SLOPE/W 2007 the pressure response factor is not included in the calculations.

The profile for the wave trough is determined with Equation (1) for time t equal to 0 s and for x varying along the cross section of the breakwater.

For the input in GEO-SLOPE/W 2007 the wave trough is defined based on 4 discrete points, see Fig. 1: two points to indicate the start and end of the wave trough, and two points to indicate the trough itself.

Wave run-up and connection with the wave profile

The maximum wave run-up is calculated according to the deterministic method of TAW (2002):

$$\frac{R_{u,2\%}}{H_{m0}} = 1.75\,\gamma_b\,\gamma_f\,\gamma_\beta\,\xi_0, \quad (2)$$

with a maximum of:

$$\frac{R_{u,2\%}}{H_{m0}} = \gamma_f\,\gamma_\beta\left(4.3 - \frac{1.6}{\sqrt{\xi_0}}\right), \quad (3)$$

where: $R_{u,2\%}$ – 2% wave run-up level above the still water line (only exceeded by 2% of the waves) [m]; H_{m0} – significant wave height [m]; γ_f – reduction factor for the effect of slope roughness [–]; γ_b – reduction factor for the effect of a berm [–]; γ_β – reduction factor for the effect of angular wave attack [–]; ξ_0 – breaker parameter [–].

The reduction factor γ_b is set equal to 1.0 as the designed rubble mound breakwater does not have a berm.

The reduction factor γ_β is set equal to 1.0 as the angle between the wave approaching the structure and the perpendicular line of the structure is not taken into account for the calculations of the wave run-up. This means that the wave attack is considered to be perpendicular to the structure. This is a conservative approach as waves attacking under a certain angle will result in a lower wave run-up level.

The reduction factor for the effect of slope roughness takes into account the wave energy dissipation on a rough slope compared to a smooth slope. The wave energy dissipation on a rougher slope (e.g. covered with stones) results in less wave run-up and less overtopping. In this project, the sea side slope of the breakwater is covered with Accropode II units (CLI 2014). For Accropode II elements a reduction factor γ_f equal to 0.46 is considered and corrected using a linear interpolation between ($\gamma_b\,\xi_0$ = 1.8; $\gamma_{f,corrected} = \gamma_f$) and ($\gamma_b\,\xi_0$ = 10.0; $\gamma_{f,corrected}$ = 1.0) (TAW 2002).

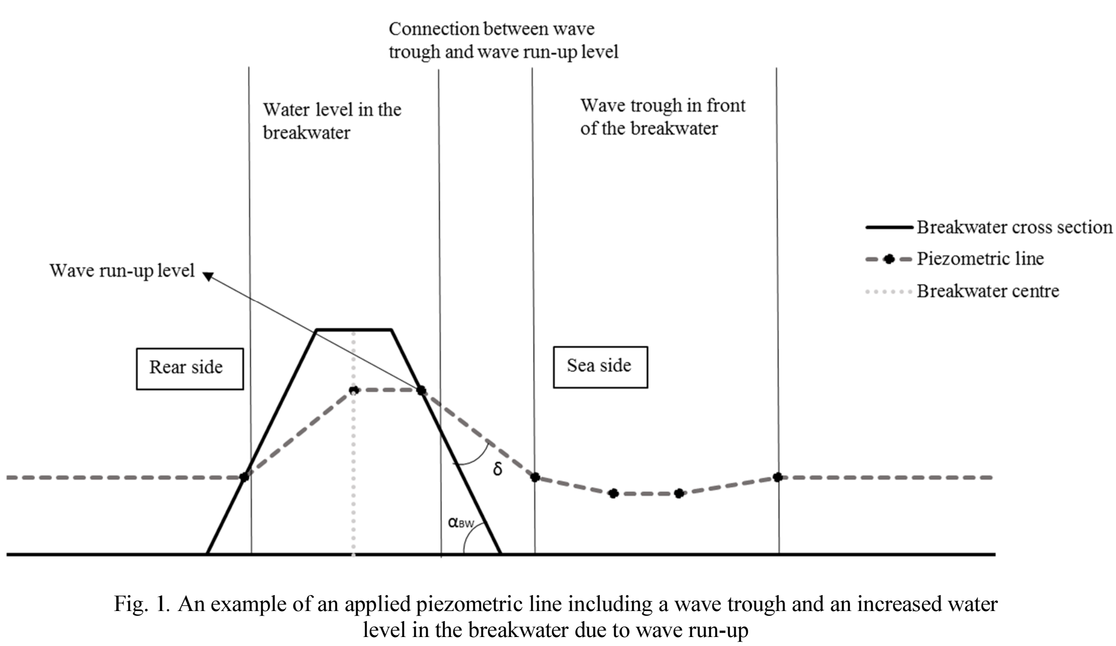

Fig. 1. An example of an applied piezometric line including a wave trough and an increased water level in the breakwater due to wave run-up

If the wave run-up level is higher than the level of the breakwater, the wave run-up level is limited to 1 m above the crest of the breakwater.

The connection between the run-up level and the wave trough is assumed linearly and is determined by the angle δ between the run-up line and the breakwater slope α_{BW}, see Fig. 1. In De Rouck (1991) an empirical formula is suggested for the angle δ:

$$\delta = \alpha_{BW} - \frac{H_s}{L}, \tag{4}$$

where: δ – angle between the run-up line and the breakwater slope [rad]; α_{BW} – slope of the breakwater [rad]; H_s – significant wave height [m]; L – wave length [m].

The angle δ is calculated for the extreme wave heights of 3.6 m and 3.9 m combined with the water level of +4.3 m CW. This condition is the most similar one to the conditions for which Equation (4) is defined. For smaller waves the formula may deviate. For the other wave conditions the same value for the angle δ is used.

Water level in the breakwater

The designed rubble mound breakwater is a permeable breakwater. The water level in the core of the breakwater is assumed equal to the calculated wave run-up level till the centre of the breakwater. From the centre of the breakwater the water level is assumed to decrease linearly till the hydrostatic water level.

This is similar to the approach followed by De Rouck *et al.* (2012) for the slope stability analysis of the breakwater in Ostend. However De Rouck *et al*. (2012) considered a constant water level equal to the wave run-up level in the entire breakwater (from the sea side till the rear side) with a linear decrease towards the hydrostatic water level along the rear side slope of the breakwater. Vanneste and Troch (2012) indicated that the pore water pressure decreases exponentially from the interface between armour layer and core towards the centre of the breakwater. To be more in line with this exponential decrease of the pore water pressure, a linear decrease of the water level towards the hydrostatic water level is considered from the centre of the breakwater towards the rear side.

The measurements of the pore pressure fluctuations under the breakwater performed by Cantelmo *et al.* (2010) also show that pore pressure fluctuations at the centre of the breakwater are relative small compared to the fluctuations more towards the sea side of the breakwater. This indicates that at the centre of the breakwater, the influence of the water level variations due to wave run-up and run-down is marginal. Vanneste and Troch (2015) determined free water surfaces of different test cases (for different wave heights and wave periods). The enveloppes of free water surfaces are showing a significant decrease of the free-surface in the armour layer and underlayer of the breakwater. At the centre of the breakwater, the free-surface envelopes are more or less equal to the still water level indicating that water level variations due to wave run-up and run-down are marginal in that area. Therefore, the assumed approach of a linear decrease of the water level from the centre of the breakwater is conservative.

Final piezometric line

In Figure 1 the three sections of the piezometric line as discussed above are indicated. In front of the breakwater, at sea side, a wave trough defined by 4 discrete points is presented. This wave trough profile is connected to the point of wave run-up level. In the breakwater the water level is set equal to the wave run-up level till the centre of the breakwater. From the centre of the breakwater a linear decrease is considered.

The piezometric line as shown in Fig. 1 is only considered for the permeable sand used for the soil replacement (improved foundation) and for the permeable breakwater materials. For the present clay layers the piezometric line is equal to the hydrostatic water level. The pore water pressure induced by the waves varies too quickly (with each wave cycle passing at a certain position) to have an influence on the pore water pressure of the clay layers. The influence depth of the wave induced pore-pressure variation, as calculated by De Rouck and Van Damme (1996), is about 3 to 4 m for the sand layers and less than 1 m for the clay layers. Close to the breakwater only sandy material is present (as improved foundation) as the clayey material is dredged and replaced with silica sand. Therefore, it is not necessary to consider a variation of pore pressures due to the waves and wave run-up in the clayey material.

Calculations

For several combinations of wave heights (normal and extreme) and water levels (normal and extreme) the piezometric line is defined.

Table 4 presents some results of calculated factors of safety for different load cases. Load case 1, for which only a hydrostatic water level is considered, results in the largest safety factor. This is in line with the findings of Shafieefar and Fakher (2015) who indicate that the effects of excess pore water pressure should be considered for wave heights larger than 1.5 m and water depths smaller than 12 m. Load case 2 is equal to load case 1 plus the presence of a wave. By considering the presence of a wave in load case 2, the safety factor decreases with about 18% compared to load case 1.

For the considered case study, the combination of the extreme low water level with the extreme wave height (i.e. load case 4) resulted in the most critical load case. Load case 3 combines the extreme high water level with the extreme wave height. The calculated safety factor is higher than the one of load case 4. This indicates that the low water level is determining the most cirtical moment (De Rouck, Van Damme 1996).

Table 4. Calculated factors of safety for different load cases

Load case	Load case description	Factor of safety
1	Hydrostatic water level +0.46 m CW	1.69
2	Water level +0.46 m CW and wave with H_s = 1.6 m	1.39
3	Water level +4.30 m CW and wave with H_s = 3.9 m	1.42
4	Water level –1.00 m CW and wave with H_s = 3.1 m	1.29

Conclusions

A simplified method is applied to include the effect of waves in front of a rubble mound breakwater in the geotechnical stability calculations. The simplified method consists of a specified piezometric line including a wave trough in front of the breakwater and a level of wave run-up in the core of the breakwater. This piezometric line is applied to the permeable soil (improved foundation) and breakwater materials. For the clayey soil a hydrostatic water level is considered.

Applying this specified piezometric line results in a lower safety factor compared to the general case of applying a hydrostatic water level. Therefore, it is recommended to consider the specified piezometric line (or a more optimised alternative) due to the wave actions in the slope stability analysis. However, it is observed that in many project the load cases related to the wave actions are not considered (and not analysed) in the slope stability analysis.

It is always a question to the authors if there is a strong justification to apply or not apply the wave actions in the slope stability analysis. It is known that the load cases related to the wave actions give a lower safety factor in comparison to load cases with constant water level.

Another question would be if different load cases (with and without wave actions) should be compared by a similar safety factor. The authors try to highlight the mentioned issues in order to get some discussions, arguments and feedback from the researchers, experts and engineers.

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