CHAPTER 125

OVERALL SLOPE STABILITY ANALYSIS OF RUBBLE MOUND BREAKWATERS

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ABSTRACT

In this paper a "semi-dynamic approach for the slope stability analysis of rubble mound breakwaters" is presented. The principles of the method are described. Tide and wave action are taken into account in the slope stability analysis of deep slip-surfaces. For these slip-surfaces the moment of maximum wave run-up is most critical.

Pore water pressure variations at the seabottom are also measured below the seabottom but attenuated and approximately up to the so called influence depth $h_{\rm ID}$. The fact that the pore pressure variation is attenuated leads to "overpressures" at the moment of low water and below the wave trough. The attenuation depends mainly on the wave period T and soil characteristics k (permeability) and $E_{\rm oxd}$ (oedometric compression modulus).

INTRODUCTION

The design of a rubble mound breakwater normally starts with the determination of the overall geometry: crest level, crest width and slope angles of both seaward and rear slope. Once the overall geometry is fixed most attention is paid to the design of the armour layer: choice of the type of armour unit (rock or concrete block and in the latter the type of block: cube, tetrapod, dolos, ACCROPOD[®], HARO[®], ...) and the weight of the armour unit.

On behalf of the design of the armour layer, several design rules are available. These rules are based on extensive research: theoretical but mainly scale model tests in hydraulic laboratories. The main loading for the armour layer is the wave attack. Furthermore it is common use to carry out scale model tests on behalf of the design of (important) rubble mound breakwaters.

With regard to the overall slope stability of the slopes of a breakwater, distinction has to be made between the seaward slope and the rear slope. For the rear slope, which is normally not subjected to wave attack, one can apply calculation methods normally used for banks of a river, a canal, ... The seaward slope however is subjected to wave attack. Considering the seaward slope, designers and researchers were used to pay too little attention to the hydrodynamic effect of wave attack.

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Frequently the experience gained on land is applied (Quinn, 1972; Fisher et al., 1975; Toppler, 1982; Chew, 1990; Murray et al, 1990; Koutitas, 1992). Only Barends takes into account the effect of waves. But he focuses on slip surfaces inside the breakwater core (Barends, 1983, 1985). Only in his paper of 1985 he presents some results with regard to the influence of (partial) liquefaction of the foundation layer due to wave attack.

During the design of the Zeebrugge breakwaters, extensive studies, research, testing and investigation into the geotechnical stability of rubble mound breakwaters have been carried out. One of the results is the proposed calculation method which takes into account the dynamic impact of waves and tides. The method is focusing on deep slip-circle stability analysis. As the dynamic effect of waves and tides is taken into account, we call it "a semi-dynamic approach for the slope stability analysis of rubble mound breakwaters".

DESCRIPTION OF THE PROPOSED METHOD

Principles

The slope stability analysis can be carried out by using different methods: Fellenius, Bishop ... for circular slip surfaces and Nonveiller, Janbu, Morgenstern and Price, ... for non circular slip surfaces. In the analysis the friction angle ϕ ' and the cohesion c' based on the effective stresses, are used. For the results shown in this paper, the Bishop formula has been applied. However the principles described further can be used for any other analysis method.

A sensitivity analysis has been carried out in order to learn which parameters are really influencing the overall stability of the seaward slope of a rubble mound breakwater. The following characteristics influencing the geotechnical safety factor F are considered: overall geometry, external forces, geotechnical characteristics (ρ , c', ϕ ') of both the material (mainly the breakwater core) and the subsoil, and pore water pressures. The sensitivity analysis showed the paramount importance of the pore water pressures along a potential slip surface: they are as important or even more important than the shear resistance characteristics.

Figure 1 shows a simplified cross-section of a rubble mound breakwater with a possible slip surface DEFA. All forces acting on the volume ABCDEFA are considered: weight of the volume, external forces acting on the faces AB, BC, DE, EFA. In that way the water pressures influence the overall geotechnical stability:

- water mass in front of the breakwater (AB);
- pressure on the breakwater slope (BC);
- pore pressure within the mound, influencing shear resistance along DE;
- pore pressure within the foundation layers, influencing the shear resistance along EFA.

To find the most critical load situation, two load conditions called A and B are considered: tide action (A) and tide combined with wave action (B).

The first case, only tide action without considering any wave action, occurs at the rear slope of a breakwater. In that case low tide is the most critical situation.



Figure 1. Simplified cross-section of a rubble mound breakwater (max. wave run up)

In the second case tide and wave action are acting together. The determining situation is the maximum wave run-up (wave crest) combined with the most critical moment in the tide cycle. A thorough investigation has shown that for a breakwater constructed in shallow water the most critical situation exists at high tide at the moment of maximum wave run-up, in fact immediately after maximum wave run-up. At that moment the wave action is changing from "pushing on the slope" into seepage forces, in fact "pulling on the mass CDEB". For a breakwater in transitional waterdepths the most critical situation will occur at maximum wave run-up at either low water (L.W.), mean water or high water (H.W.). This is in clear contradiction with the widespread idea that the moment of maximum wave run-down, with the wave through in front of the breakwater is the most critical. The latter is only valid for slip surfaces which fully develop in the breakwater core.

Weight of the volume ABCDEFA

The weight of the considered volume (partially of the breakwater and partially of the foundation layer) can easily be calculated based on the geometry and the unit mass of the breakwater material and subsoil.

Watermass in front of the breakwater: forces acting on surface AB

We can easily take into acount the water mass in front of the breakwater by suggesting that the slip surface reaches the water surface, calculating with following water characteristics: $\rho_w = 1 \text{ t/m}^3$; c' = 0; $\phi^c = 0$ (De Beer, 1959). This is only valid for a horizontal water surface in rest, e.g. case A, because the water level is varying with a great period, so very slowly. Combining tide and wave action the sea level is going up and down and the surface isn't horizontal any more, the water pressure won't be hydrostatic: the so called Still Water Level (S.W.L.) is varying between low

and high tide and the waves are characterised by a wave height H, a wave period T and wave length L.

Several wave theories describe the water surface, determined by those parameters: Airy (linear wave theory), non linear wave theory, cnoidal waves ... As breakwaters are normally constructed in shallow water the cnoidal wave theory has to be applied, although the cnoidal theory is very complicated. Grace showed that for both cnoidal and Airy theory correction coefficients have to be used (Grace, 1978). So pressures on the surface AB can be calculated with sufficient accuracy using the Airy theory but taking into account the correction coefficients proposed by Grace.

Water pressure on the slope surface: forces acting on surface BC

In the case of not overtopped breakwaters the pressure on the slope surface of the breakwater is directly influenced by the shape of the incident wave. Based on results published by Brandtzaeg (Brandtzaeg, 1962; Brandtzaeg et al, 1966, 1969) and based on the measurements taken during the scale model tests for the Zeebrugge breakwaters, we recommend to registrate the wave form in front of the slope up to one wave length. Having determined the wave form at the time of maximum wave run-up allows to calculate forces on the slope. Essential in this method is the determination of maximum wave run-up. The run-up height R_u is defined as the vertical distance between S.W.L. and the highest point to which water from an incident wave will run up the slope of a structure. The value of R_u/H depends on the wave characteristics (H, T, L) and slope characteristics (slope angle, roughness, porosity, ...).

Pore pressure within the mound: forces acting on surface DE

On surface DE shear forces are acting. These shear forces depend on the shear strength characteristics of the material and on the effective stress. The latter depends on the pore pressures.

Involving water pressures in the mound, a distinction is made between tide action (case A) and tide and wave action (case B). In case A we can adopt a hydrostatic pressure for the water pressures along the slip surface.

Wave action (case B) on the seaward slope results in a water level set up inside the core. This is due to the fact that the inflow surface during wave run-up is greater than the outflow surface during run-down. In addition the mean flow path for inflow is shorter than that for outflow. This set up leads to higher mean pore pressures. Due to wave action pore pressures are varying within one wave period, depending on the location and on the attenuation through the armour layer, filter layer and core (Troch P. et al, 1996a and b).

Pore pressures within the foundation layers: forces acting on surface EFA

As for surface DE, on surface EFA shear forces are acting which depend on the pore pressures. The sensitivity analysis has shown the predominant importance of the pore pressures along EFA.

If these pressures are hydrostatic with regard to the seawaterlevel, they can be defined easily. But the question has to be put whether they are hydrostatic or not. The



sea waterlevel is continuously changing: slowly (tide with a period = 12h25min) or fast (waves: with periods between 6, ..., 20 s).

Figure 2. Definition sketch of attenuation of tide

Figure 2 shows the principle of wave attenuation in the subsoil. This attenuation results in an "overpressure" in the pores at the moment of L.W. or below the wave through. For tides the overpressure can be written as:

$$\Delta u = \rho_w g \Delta h$$

$$= \rho_w g (1 - \alpha_{Pi}) \frac{H_{sea}}{2}$$
(1)
with Δu : overpressure at low tide (in kPa)
$$\rho_w$$
: unit mass of seawater (t/m³)
$$\Delta h$$
: overpressure at low tide (in m seawater)
$$\alpha_{Pi}$$
: attenuation coefficient of piezometer Pi (-)
$$H_{sea}$$
: tidal range H.W. - L.W. (m)

This overpressure highly influences the overall slope stability.

ON SITE MEASUREMENT OF TIDE AND WAVE ATTENUATION IN THE SUBSOIL

Test set-up

In order to investigate the attenuation of varying water pressures (at the seabottom) in the subsoil four measurement campaigns have been carried out on behalf of the design of the Zeebrugge breakwaters. In this paper we will focus on the 4^{th} campaign, taking into account both tides and waves.

In general this 4th campaign has the following characteristics:

- The seabottom is located at ca. Z-5,50.
- From the seabottom the following layers are found:
 - up to -14,00 : coarse sand with shells
 - up to -15,50 : a clayey sand layer
 - a stiff clay layer (Tertiary, Bartonian clay) up to much greater depths.
- In the harbour area M.L.W.S. is situated at the level Z+0,32 and M.H.W.S. at the level Z+4,62, i.e. waterdepths of about 5,80 m to about 10,10 m at the location of the multiple piezometer probe. The design wave height for the main breakwaters amounts 6,20 m, at design waterlevel Z+6,76.

A general view of the measuring system is given in fig. 3. The pore pressures in the seabottom are measured by electrical pressure transducers, mounted at seven levels in a multiple piezometer probe which was jacked into the seabottom. The pressure transducers are connected by a cable bundle to a measuring platform installed on top of a steel tube pile in the vicinity of the probe. The measuring platform contains the conditioning for the piezometer probe, a transmitter and all the related equipment. The measured values are sent by the transmitter to the coast by an UHF link. At the coast, the measured values are caught by a receiver and sent to a computer by telephone link where they are stored on disk and subsequently analysed.



Figure 3. Lay-out of the measuring system for the 4th campaign

The multiple piezometer probe consists of six steel pipes linked together by stainless steel nipples containing the pressure transducers. Below each nipple a decrease in diameter was chosen to prevent direct contact between the different piezometer levels along the mantle of the probe. The diameters of the elements of the probe were kept as small as possible in order to keep the penetration forces within acceptable limits and to minimize soil disturbances. The pressure transducers are of the semi-conductor strain gauge type with very fast response and suitable electric output signal; they have a very small volume displacement, no moving parts and a low energy consumption.

More details about the lay-out of the system and the multiple piezometer probe, even about the measuring platform and equipment, data acquisition system and installation procedure are given in De Wolf et al. (1983) and De Rouck (1991).

Results

The tide is transmitted up to the tip of the probe at 13,00 m below the seabottom: within the sand layer only slightly attenuated but strongly attenuated in the clay layer.

Even wave action is registrated up to the tip of the probe: the amplitude of the waves decreases very quickly from the seabottom to a depth of about 2,00 m underneath the seabottom, with further attenuation for greater depths.

THEORETICAL MODEL OF WATER PRESSURE PROPAGATION IN THE SUBSOIL

Out of the results of the measuring campaign it can be concluded that the water pressure in the subsoil is not hydrostatic due to the continuously changing seawaterlevel. The amplitude of the pressure variation is attenuated.

The differential equation governing the pore water pressure change during one tide (and one wave) has been written based on the Darcy equation and the continuity equation.

Consider an elementary volume dx.dy.l and the effect of tides (fig. 4).



Figure 4. Tide: pore water flow in the subsoil due to seawater lowering

The equation of Darcy:

$$\mathbf{w} = -\frac{\mathbf{k}}{\rho_{\rm w}g}\frac{\partial \mathbf{u}}{\partial z} \tag{2}$$

with: w : flow velocity in z-direction

u : pore water pressure

k : permeability coefficient of the soil

 ρ_w : specific mass of seawater

The continuity equation takes into account following phenomena:

- if the seawaterlevel drops from high tide to low tide, the pore water pressure decreases; the pore water expands, so some water water is pressed out of the considered volume;
- at the same time the total effective stress decreases (both the vertical and horizontal stress decrease): the grain skeleton expands, so some water is sucked.

The continuity principle leads to:

$$\frac{\partial w}{\partial z} = -\left(\frac{1}{E_{\text{red}}} + \frac{n}{K_{yy}}\right)\frac{\partial u}{\partial t}$$
(3)

with E_{oed} : oedometric compression modulus of the soil

n : porosity of the soil

K_w : compression modulus of seawater

As all soillayers are situated below the seabottom it has been assumed that they are fully saturated.

Combining the equation of Darcy and the continuity equation leads to the differential equation for pressure response in the soil underneath the seabottom:

$$\frac{\partial^2 \mathbf{u}}{\partial z^2} = \frac{\rho_{w} \mathbf{g}}{\mathbf{k}} \left(\frac{1}{\mathbf{E}_{\text{red}}} + \frac{\mathbf{n}}{\mathbf{K}_{w}} \right) \frac{\partial \mathbf{u}}{\partial \mathbf{t}}$$
(4)

 $- \text{ via } \frac{1}{E_{\text{ord}}} \quad \text{the compressibility of the grain skeleton is taken into account}$ $- \text{ via } \frac{n}{K} \quad \text{the compressibility of seawater is taken into account}$

Comparing the order of magnitude of both terms for clay and sand learns that $\frac{n}{K_w}$ is

clearly smaller than $\frac{1}{E_{oed}}$, so $\frac{n}{K_w}$ can be neglected. Equation (4) can be simplified to

$$\frac{\partial^2 \mathbf{u}}{\partial z^2} = \frac{\rho_{\rm w} g}{\mathbf{k} \mathbf{E}_{\rm ord}} \frac{\partial \mathbf{u}}{\partial t}$$
(5)

In a similar way the general equation for water flow in the subsoil due to wave action can be written as:

$$\frac{\partial^2 \mathbf{u}}{\partial x^2} + \frac{\partial^2 \mathbf{u}}{\partial z^2} = \frac{\rho_w g}{k} \left(\frac{1}{E_{oed}} + \frac{n}{K_w} \right) \frac{\partial \mathbf{u}}{\partial t}$$
(6)

As for tides the term $\frac{n}{K_w}$ can be neglected, leading to:

$$\frac{\partial^2 \mathbf{u}}{\partial \mathbf{x}^2} + \frac{\partial^2 \mathbf{u}}{\partial \mathbf{z}^2} = \frac{\rho_{\mathbf{w}} \mathbf{g}}{\mathbf{k} \mathbf{E}_{\text{oed}}} \frac{\partial \mathbf{u}}{\partial \mathbf{t}}$$
(7)

Comparison of the order of magnitude of $\frac{\partial^2 u}{\partial x^2}$ and $\frac{\rho_w g}{kE_{oed}} \frac{\partial u}{\partial t}$ for normal waves (e.g.

as measured during the 4th campaign) and for both clay and sand shows that $\frac{\partial^2 u}{\partial x^2}$ is

clearly smaller than $\frac{\rho_w g}{kE_{oed}} \frac{\partial u}{\partial t}$. So $\frac{\partial^2 u}{\partial x^2}$ can be neglected.

Equation (7) can be simplified to:

$$\frac{\partial^2 \mathbf{u}}{\partial z^2} = \frac{\rho_{w} \mathbf{g}}{\mathbf{k} \mathbf{E}_{oed}} \frac{\partial \mathbf{u}}{\partial \mathbf{t}}$$
(8)

Equation (5) and (8) are identical.

Solving this differential equation is done taking into account the following boundary conditions:

a) at the sea bottom (z = 0)

$$\mathbf{u}(\mathbf{o},\mathbf{t}) = \mathbf{u}_{\mathbf{o}} \cos\left(\frac{2\pi x}{L} - \frac{2\pi t}{T}\right)$$
(9)

Or, for a fixed spot, take x = 0

$$\mathbf{u}(\mathbf{o},\mathbf{t}) = \mathbf{u}_{\mathbf{o}} \cos \frac{2\pi t}{T} \tag{10}$$

with u_o - for tides $u_o = \rho_w g H_{sea}$

with H_{sea} : tidal range H.W. - L.W.

- for waves
$$u_o = \frac{\rho_w g}{n} \frac{H}{2\cosh\frac{2\pi d}{L}}$$

with H: wave height
L: wave length
d: water depth
n: correction coefficient of Grace

b) at great depth $(z = \infty)$

$$\lim_{z \to \infty} u(z,t) = 0 \tag{11}$$

This leads to the following solution:

$$\mathbf{u}(\mathbf{z},\mathbf{t}) = \mathbf{u}_{o} \ \mathbf{e}^{-\mathbf{A}\mathbf{z}} \cos\left(\frac{2\pi\mathbf{x}}{\mathbf{L}} + \frac{2\pi\mathbf{t}}{\mathbf{T}} - \mathbf{A}\mathbf{z}\right)$$
(12)

with:

$$A = \sqrt{\frac{\rho_{w}g\pi}{k E_{oed}T}} \left[\frac{1}{m}\right]$$
(13)

with	L	: wave length
	Т	: wave period
	k	: permeability coefficient of the soil
	Eoed	: oedometric compression modulus of the soil

Figure 5 shows the pore water pressure variation in a soillayer due to tide or wave action. Curve 1 is valid for x = 0, t = T/2 and A = 0,3, while curve 2 is the envelope. Approximatively curve 1 can be replaced by the straight line AB. The distance OB is the so called "influence depth". This influence depth has proven to be a very handsome tool to introduce the variation of the pore water pressure in the subsoil into the slope stability analysis.



Figure 5. Pore water pressure variation in a soil layer due to tide or wave action

The attenuation decreases when the influence depth $h_{\rm ID}$ increases, i.e. when A decreases. A decreases when the product k.E_{oed}.T increases: a more permeable and less compressible soil combined with longer wave period (and subsequent longer wave length).

Comparison of on site measurement results with the theoretical model

It is investigated how the results of the on site measurements fit with the theoretical model. On the one hand the results for tides for respectively the stiff Bartonian clay and all sand layers (in which pore water pressure measurements have been carried out) and on the other hand the results of the 4th campaign for waves for the sand layer are considered.

For both tides and waves the attenuation coefficient is "measured on site" (i.e. determined based on the measurements on site). The attenuation coefficient is also "calculated" (i.e. by introducing soil characteristics in equation (13)). It is found that "measured" and "calculated" values coincide quite well (De Rouck, (1991).

APPLICATION OF THE PROPOSED METHOD FOR SLOPE STABILITY ANALYSIS

The proposed method for analysing the overall slope stability has been applied at numerous occasions. Within this paper an example for Zeebrugge (Belgium) and Antifer (France) will be given.

Zeebrugge inner harbour breakwater

Figure 6 shows the most critical slip circle for a Zeebrugge inner harbour breakwater when considering the design water level Z+6,76. The crest of the breakwater is situated at Z+10,30. Maximum run-up will not fully occur as the considered extreme wave will pass over the crest (overtopping). The safety factor $F_{min} = 1,42$, which is rather high.



Figure 6. Zeebrugge breakwater slope stability analysis

Antifer main breakwater

Based on the geometry as given in Feuillet et al. (1987) the slope stability analysis is carried out. As no exact geometric data and hydraulic data (tide, waves) and no exact characteristics of the materials were available, approximative geometry and wave loading and approximative but realistic values for the characteristics of all layers (breakwater and subsoil) have been introduced in the slope stability analysis. Regarding these circumstances the result has to be considered as approximative as well. The analysis has been carried out for high tide and low tide. Low tide is clearly more critical. So fig. 7 shows the result for max. wave run-up at low tide: $F_{min} = 1, 13$. For the design of the Zeebrugge breakwaters for the case B (tide combined with wave attack) an $F_{min} \ge 1, 10$ has been required.



Figure 7. Antifer main breakwater slope stability analysis

FURTHER USE OF RESULTS

Knowledge of the water pressures in the soillayers below the seabottom, resp. below the bottom of a river subjected to tides, allows to explain unexpected behaviour of structures. Two examples are given.

An intake of water, constructed on a sandlayer on top of a 12 m thick claylayer subsides at high tide and rises at low tide. This is completely unexpected regarding the higher submerged part at high tide. The phenomenon can be explained by taking into account the water pressures in the claylayer (De Rouck, 1991). The crest of a quaywall subjected to tides, constructed on a claylayer or on a sandlayer on top of a claylayer may move towards the river at high tide and return at low tide. This will be the case if the effect of compression of the claylayer at high tide is dominant compared with the effect of the higher stabilising forces at high tide.

CONCLUSIONS

Within this paper a method for slope stability analysis of the seaward slope of a rubble mound breakwater has been described. The dynamic effect of both tides and waves has been taken into account.

The main conclusions are:

- Slope stability analysis of the slopes of the breakwater, especially the seaward slope, deserves as much attention as the design of the armour layer.
- The slope stability can be carried out by considering a volume ABCDEFA (fig. 1) which reaches into the soil layer underneath the seabottom.
- Pore water pressure variation (due to waves and tides) at the seabottom are also measured below the seabottom but attenuated and approximatively up to the so called influence depth $h_{\rm ID}$. The fact that the pore pressure variation is attenuated leads to "overpressures" at the moment of low water and below the wave trough (for both L.W. and H.W.)
- The attenuation decreases with increasing influence depth h_{ID} , i.e. with decreasing factor A. The factor A on his turn decreases when the product k.E_{oed}.T increases.

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