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Analysis and classification of stepwise failure of monolithic breakwaters

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ABSTRACT

Analysis, interpretation and classification of stepwise failure of monolithic breakwaters are presented. Through the stepwise failure mechanism, monolithic breakwaters develop incremental residual displacements with each wave load event. This mode of failure is associated with the highly complex processes involved in wave–structure–foundation interaction and may occur even under relatively moderate wave conditions. Based on the results of the analysis of a well validated CFD–CSD model system and large-scale physical model test data, a new concept, named *load eccentricity concept*, is proposed to classify the response of the foundation in *four load eccentricity regimes*. This concept is based on the relative eccentricity e/B, i.e. the ratio of eccentricity e of the vertical force resultant from the mid-point of the foundation–structure interface related to width B of this interface. In fact, the relative eccentricity carries all significant information related to the wave loads (horizontal and uplift forces) and to the properties of the structure (mass and geometry). Further, a new 3-DOF model, which can handle the system nonlinearity (e.g. soil plasticity), is developed for preliminary analysis. For the application of the model, only two parameters are needed: the relative load eccentricity and the relative soil density.

1. Introduction

Geotechnical failure modes of monolithic breakwaters can essentially be classified into two main categories according to the development/mechanism of failure: sudden failures due to extreme events and stepwise failures due to relatively moderate wave loads. Current design procedures basically only consider safety against failures caused by pre-defined extreme events (e.g. against a design wave height). Whereas, in fact, many recorded field failures of monolithic breakwaters are caused in an incremental fashion [1], in which a sufficient number of moderate to moderately severe storms can render the structure nonoperational. Hence, the stepwise failure consists of irreversible small soil deformations and subsequent small residual displacements of the monolithic breakwater which develop incrementally under a series of wave impact loads.

Despite the extensive research efforts at interdisciplinary and multinational level such as EU-Projects PROVERBS (e.g. [2]) and LIMAS (e.g. [3]) and despite recent advances in numerical modelling (e.g. [4,5]), some crucial issues associated with the vulnerability of marine gravity structures to soil foundation failures under wave attack still remain unsolved. This is particularly the case for the *stepwise failure* mode. In fact, no reliable model yet exists to predict this type of failure observed in the laboratory [2] and under field conditions [1], including its role in the often observed seaward tilt of vertical breakwaters [2]. This might explain why no guidelines are yet available in current design codes to account for this failure mode, and its implications for service limit state under moderate to high wave conditions, and for ultimate limit state by increasing vulnerability of the structure to collapse under extreme wave conditions.

In this study, stepwise failure of monolithic breakwaters is studied by means of analysing the results of a well validated semi-coupled CFD–CSD numerical model system as well as large-scale physical model tests of caisson breakwaters. A new parameter is presented to consolidate and describe the severity of wave loads on a marine monolithic structure. Further, a new framework is presented to interpret and classify stepwise failures of monolithic breakwaters. Moreover, a simplified model for practical engineering use is proposed and applied successfully to the large-scale physical model tests. The results of this study can assist systematic development of design guidelines against stepwise failure as well as design recommendations to minimise the long-term effects of stepwise failures.

2. Relevant physical processes

Stepwise failure of monolithic breakwaters is a result of the highly complex physical processes associated with wave–structure–seabed

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Nomenclature

$\boldsymbol{\sigma}$	stress tensor (kN/m^2)
$\boldsymbol{\tau}$	shear stress tensor (kN/m^2)
$\boldsymbol{\varepsilon}$	strain tensor (–)
γ	phase fraction (–)
$\mathbf{E}$	the elasticity 4th order tensor (kN/m^2)
$\mathbf{U}$	fluid mixture (intrinsic) velocity vector (m/s)
$\mathbf{u}$	the displacement vector (m)
∇	the del (Nabla) operator
$\overline{\mathbf{U}}$	fluid mixture averaged (Darcy) velocity vector (m/s)
ϕ	angle of internal friction (deg)
ζ	loading flag; 1 is loading and 0 is unloading (–)
c	the damping coefficient (kN/m^2)
K	the stiffness coefficient (kN/m^2)
p	fluid (pore) pressure (kN/m^2)
CFD	Computational Fluid Dynamics
CSD	Computational Structural Dynamics

interaction (e.g. breaking wave impact and soil response under cyclic loading). Fig. 1 shows how impact wave loads, though of short durations, can jeopardise the stability of the caisson via small cumulative residual horizontal displacements. Additionally, Fig. 2 illustrates typical results from the large-scale physical model tests in the Large Wave Flume GWK [3]. Considering regular (repetitive) breaking waves hitting the structure, for each wave event a small irreversible (residual) vertical displacement ($d_{v,b}$) is recorded together with the associated pore pressure as exemplarily shown in Fig. 2 for a location at the rear side. Each residual vertical displacement is accompanied by an accumulation (build-up) of pore pressure (p) inside the soil foundation. Cumulatively, the structure develops a tilt towards either the seaside or shoreside direction. Many field recorded caisson breakwaters tilts are in the seaward direction [1]. This type of failure is not yet implemented in the current design codes. Nevertheless, general guidelines for design purposes are provided in [6].

Stepwise failures are more apparent when the structure is frequently subject to breaking wave loads. The highly dynamic and stochastic nature of wave loads on vertical structures makes a reliable prediction very difficult. Wave breaking magnifies the dynamic effect of wave loads, as the loading frequency approaches natural frequencies of the structure whereas non-breaking and totally broken waves can mostly be simplified as equivalent static loads for design purposes without a significant risk of structural resonance. Oumeraci et al. [2] present a classification of wave loads in impact and non-impact (or pulsating) loads (Fig. 3).

As shown in Fig. 4, when breaking waves approach a vertical wall, the water in front of the wall has a high content of entrapped air and can be regarded as a bubbly mixture between the wave front and the wall or as a large air pocket underneath the tongue of the breaking wave which is then entrapped at the wall. This entrapped air results in oscillations of wave-induced pressure just after the wave impact (the 2nd force peak). This is due to the high air compressibility. This process cannot be reproduced numerically unless fluid compressibility is taken into account.

While the horizontal wave load is not significantly affected by the motions of the caisson breakwater, this is not the case for wave-induced uplift force. The interaction between wave, structure and foundation is significant for the uplift loads as they are strongly affected by the rocking motion of the structure. Experimental results, Fig. 5, show that caisson rocking due to breaking wave impact can reduce or increase the uplift force by a magnitude up to 30%, which forms a solid base for the argument of the importance of wave–structure–foundation interaction [2].

As the structure undergoes the rocking motion response to wave impact loads, the foundation underneath is cyclically loaded under both edges of the structure. The strong solid–fluid coupling of the seabed soil and its plasticity significantly affect the soil response to wave-induced loading of the structure. Because of the huge own-weight of monolithic breakwaters, their wave-induced cyclic loads on the soil foundation are either asymmetric (for moderate to high wave loads) or very asymmetric (for small to moderate wave loads) as explained in De Groot et al. [9] and schematically illustrated in Fig. 6.

As stated earlier, according to Oumeraci [1], several vertical breakwater failures resulted from a seaward tilt. Such a failure was also observed in centrifuge tests [10–13]. Oumeraci [1] also reports that most of the failed breakwaters had a low crest (and consequently heavily overtopped) and too high toe berm (however not composite breakwaters). These two observations may seem less relevant due to the configurations of centrifuge tests, e.g. [13] (no rubble base and equivalent mechanical loading to simulate wave loading). The reason for the seaward tilt failure has been attributed to several mechanisms, including seabed scour or soil liquefaction underneath the breakwater seaward side, although stepwise failure with seaward tilt was reproduced in large-scale physical tests with scour prevented [3], and seawards directed impacts caused by excessive wave overtopping, which result in caisson's tilt seaward as shown in Fig. 7. This mode of failure is of special significance to structures of relatively low mass [14]. Nevertheless, a satisfactory explanation is still lacking.

3. The CFD–CSD model system

A semi-coupled CFD–CSD model system is developed in OpenFOAM®[1] to extend the testing conditions from the large-scale physical model tests of Kudella et al. [3] in order to study stepwise failure of monolithic breakwater. In this section, the hydrodynamic (CFD) model, the hydro-geotechnical (CSD) model, the coupling procedure and the validation of the coupled CFD–CSD model system are presented.

3.1. Hydrodynamic (CFD) model

A hydrodynamic solver (*waveVolAvgPorousInterFoam*) is developed as an extension of the Eulerian volume-of-fluid multiphase (single fluid mixture, air and water) *porousInterFoam* solver. The latter (original solver) considers the effect of porous media via introducing a sink term to the momentum balance equation to model their resistance to the flow. The former (new developed solver) uses the volume averaging principle of fluid velocity among other modifications. The governing equations are adopted from volume averaging models (e.g. Hsu et al. [15]). An extra term is introduced to the continuity equation to account for fluid compressibility. The continuity equation reads:

$$\nabla\cdot\overline{\mathbf{U}} + \frac{1}{Q}\frac{\partial p}{\partial t} = 0 \tag{1}$$

where $\overline{\mathbf{U}}$ is the ensemble average fluid velocity vector ($\overline{\mathbf{U}} = n\mathbf{U}$; where $\mathbf{U}$ is the intrinsic velocity and n is the porosity), p is the (intrinsic) fluid pressure, $[\frac{1}{Q} = \frac{n}{K_f}]$ and $[\frac{1}{K_f} = \frac{S_w\gamma}{K_w} + \frac{(1-S_w)\gamma}{K_a}]$ where n equals unity outside porous media, K_f and K_w are the bulk moduli for pore fluid and pure water respectively, γ is the phase fraction (varies between unity for only water and zero for only air) and S_w is the degree of saturation ($S_w = \frac{V_w}{V_v}$) inside porous media (S_w equals unity outside porous media), V_w is the volume of pore water, V_v is the volume of voids and K_a is the air bulk modulus equal to the absolute pore pressure, which under atmospheric

[1] OpenFOAM® is a registered trademark of OpenCFD Ltd.