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Dynamic response and sliding distance of composite breakwaters under breaking and non-breaking wave attack

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ABSTRACT

Over the last 15 years improved awareness of wave impact induced failures has focused attention on the need to account for the dynamic response of maritime structures to wave impact load. In this work a non-linear model is introduced that allows evaluating the effective design load and the potential sliding of caisson breakwater subject to both pulsating and impulsive wave loads. The caisson dynamics is modelled using a time-step numerical method to solve numerically the equations of motion for a rigid body founded on multiple non-linear springs having both horizontal and vertical stiffness. The model is first shown to correctly describe the dynamics of caisson breakwaters subject to wave attack, including nonlinear features of wave–structure–soil interaction. Predictions of sliding distances by the new method are then compared with measurements from physical model tests, showing very good agreement with observations. The model succeeds in describing the physics that stands behind the process and is fast, accurate and flexible enough to be suitable for performance design of caisson breakwaters.

1. Introduction

The capability of wave impact loads to cause the sliding of composite-type breakwaters had been proved in the early sixties by Nagai (1966) who stated "It was proven by 1/20 and 1/10 scale model experiments that, at the instant when the resultant of the maximum simultaneous shock pressures just exceeds the resisting force, the vertical wall slides".

Analyses of failures carried out in Europe and Japan over the past 15 years confirm impact loads induced sliding to be the most important cause of failure for caisson breakwaters. Nevertheless, despite the fact that the importance of impulsive loadings and their effects on the dynamic of caisson breakwater have been widely recognised, a simple and comprehensive methodology for the assessment of cumulative sliding distance is still missing. This paper presents a simple but consistent method for modelling the dynamic response and sliding distance of composite breakwaters subject to wave attack.

In the following, documented cases of sliding-induced failures of caisson breakwaters are briefly summarised (Section 2.1) and findings from previous researches on dynamics of caisson breakwaters reviewed (Section 2.2–2.3). A non-linear dynamic model for the response of caisson breakwaters subject to wave loading is then presented (Section 3), together with a procedure for the generation of wave force time-histories for use in dynamic analysis (Section 4). The effectiveness of the model is then verified using simplified force time-histories (Section 5) and finally compared to measurements from physical mode tests on sliding of caissons subject to both pulsating and breaking wave attack (Section 6) showing very good agreement with both analytical solutions and experimental observations.

2. Literature review

Research on the dynamics of caisson breakwaters subject to wave loading has mainly concentrated on surveying damaged and failed structures, understanding the physics that stands behind the dynamics of caissons and defining wave loads for use in dynamic analysis. Accordingly, in the following we summarise documented failure of caisson breakwaters and most significant efforts towards the understanding of caisson dynamics.

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2.1. Documented failures

Oumeraci (1994) gave a review of analysed failure cases for both vertical and composite breakwaters: 17 failure cases were reported for vertical breakwaters and 5 for composite or armoured vertical breakwaters. The author identified wave breaking and breaking clapotis as the most frequent damage source of the disasters experienced by vertical breakwaters, by means of (in order of importance): sliding, shear failure of the foundation and overturning.

Franco (1991, 1994) and Franco and Passoni (1992) summarised the Italian experience in design and construction of vertical breakwaters giving a historical review of the structural evolution in the last century and critically describing the major documented failures (Catania, 1933; Genova, 1955; Ventotene, 1966; Bari, 1974; Palermo, 1983; Bagnara, 1985; Naples, 1987; Gela, 1991). In all cases the collapse was found to be due to unexpected high wave impact loading, resulting from the underestimation of the design conditions and the wave breaking on the limited depth at the toe of the structure.

Knowledge on failure mode of vertical breakwaters has been widened by the large experience inherited in recent years from observations made all through last decades in Japan. Among the others, Goda (1974) reported and re-analysed a large number of historical sliding-induced failures of vertical caisson breakwaters in Japan, Hitachi (1994) described the damage of Mutsu Ogawara Port (1991), Takahashi et al. (1994) discussed the failures occurred at Sakata (1973–1974) and Hacinohe (1991) Ports. Takahashi et al. (1998) discussed results from an extensive field survey of Japanese breakwaters and summarised caisson wall failures in the period 1977–1997. Among other findings, the authors confirmed impulsive breaking wave pressure to be the main cause of damage for caisson breakwaters, together with the collision of concrete blocks against the caisson walls. More recently, Takahashi et al. (2000) analysed 33 major failures that occurred between 1983 and 1991 and reported typical failures of composite breakwaters; the authors identified sliding of caissons and structural failures due to impulsive wave pressure as the most important failure modes for caisson breakwaters installed on a steep foreshore and subject to breaking wave attack.

2.2. Existing models for dynamics of caisson breakwaters

Marinski and Oumeraci (1992) gave a review of the CIS (formerly Soviet Union) design experience on dynamic response of vertical structures subject to breaking wave forces. Most of the methods developed in the CIS assumed the dynamics of vertical breakwater to be well described by that of a rigid body on a homogenous, elastic and isotropic half space with the soil parameters adopted in the model driving the overall response of the system. Reviewing the available literature (almost always in Russian), the authors identified three schools of thoughts, based respectively on theoretical works by Petrashen (1956), Smirnov and Moroz (1983) and Loginov (1962, 1969). The method suggested by Loginov is the only one to have been included in the Russian guidelines for the evaluation of the loadings and their effects on maritime structures; the model combines the swaying and rotating motions of the caisson in two rocking motions around two separate centres (located respectively above and below the centre of gravity of the caisson) and neglects the effect of damping.

De Groot et al. (1996) extensively reviewed (at time) state of the art methods for design of caisson breakwater foundation, including existing approaches to dynamics. On this ground, simplified models for the dynamic behaviour of caisson breakwaters have been developed within the framework of the PROVERBS (PRObabilistic design tools for VERtical BreakwaterS) research project (see, among others, Oumeraci and Kortenhaus, 1994; Oumeraci et al., 1992; Klammer et al., 1994).

Despite its relative simplicity, the model proposed by Oumeraci and Kortenhaus (1994) represents an efficient tool for the exploration of the dynamic response of caisson breakwaters to wave impact loads and a remarkable attempt to quantify the relative importance of the applied dynamic load and the dynamics (mass, stiffness and damping) of the breakwater (including the superstructure, its foundation soil and the surrounding water) on the overall dynamic response of the system as a whole. For these reasons, this model is briefly described in the following.

The rigid body in the idealised 2D lumped system sketched in Fig. 1 has two degrees of freedom, respectively the horizontal translation and the rotation around A. For such a system, the equation of motion can be re-written in matrix form as follows:

$$\mathbf{M}\cdot\ddot{\mathrm{u}}(t) + \mathbf{C}\cdot\dot{\mathrm{u}}(t) + \mathbf{K}\cdot\mathbf{u}(t) = \mathbf{F}(t) \tag{1}$$

where:

$$\mathbf{M} = \begin{bmatrix} m_x & 0 \\ 0 & m_\theta \end{bmatrix} \tag{2}$$

$$\mathbf{C} = \begin{bmatrix} c_x & c_x\cdot(H_c - y_A) \\ c_x\cdot(H_c - y_A) & c_\theta + c_x\cdot(H_c - y_A)^2 \end{bmatrix} \tag{3}$$

$$\mathbf{K} = \begin{bmatrix} k_x & k_x\cdot(H_c - y_A) \\ k_x\cdot(H_c - y_A) & k_\theta + k_x\cdot(H_c - y_A)^2 \end{bmatrix} \tag{4}$$

$$\mathbf{F} = \begin{bmatrix} F_x(t) \\ F_x(t)\cdot(y_L - y_A) + F_y(t)\cdot(x_A - x_L) \end{bmatrix} \tag{5}$$

$$\mathbf{u} = \begin{bmatrix} u_x(t) \\ u_\theta(t) \end{bmatrix}. \tag{6}$$

The terms m_x, m_θ, k_x, k_θ, c_x, and c_θ in Eqs. (2)–(4) represent the total mass (m), the stiffness (k) and the damping (c) of the system against sliding (x) and rocking (θ) and x_L and y_L are respectively the lever arm of the vertical (F_y) and horizontal (F_x) forces, x_A and y_A are respectively the coordinates of the centre of rotation of the caisson. According to the authors, the stiffness terms can be determined according to Marinski and Oumeraci (1992) while the total mass of the system is given by the summation of the mass of the caisson, the hydrodynamic mass and the geodynamic mass. The damping coefficients were obtained experimentally by means of pendulum tests on the caisson breakwater model itself and for different degrees of immersion (Oumeraci et al., 1992).

Moving from earlier observations during small-scale model tests (Klammer et al., 1994) a simple model for the evaluation of the permanent displacement of caisson breakwaters under impact loads has been suggested by Oumeraci et al. (1995), de Groot et al. (1996) and Kortenhaus et al. (1996). According to the authors, the interaction of the super-structure with the foundation soil is driven by adhesion while the horizontal force does not exceed the critical value $F_{x,c}$:

$$F_{x,c}(t) = \mu_s\cdot\left[W - F_y(t)\right] \tag{7}$$

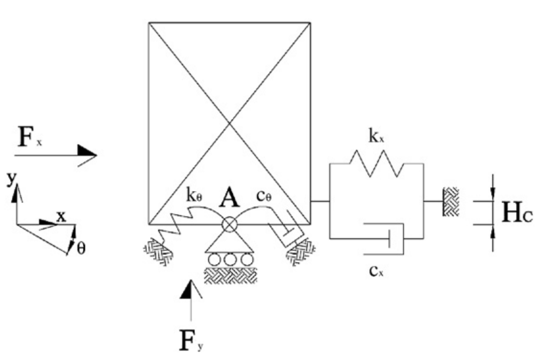

Fig. 1. Dynamic model of caisson breakwater. After Oumeraci and Kortenhaus (1994).