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ANALYSIS OF STEPWISE FAILURE OF MARINE GRAVITY STRUCTURES AND IMPLICATIONS FOR DESIGN PRACTICE

By

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MOTS CLÉS: défaillance par étapes, interaction sol-structure-houle, structures poids maritimes, digues monolithiques, digues constituées de caissons

1. INTRODUCTION

Caisson-type gravity structures have an unprecedented potential as multi-purpose structures for diverse applications (e.g. breakwaters and quay walls for harbours, sea walls for shore protection, support structures for marine bridges as well as for wave and tidal power extraction devices). To suit the increasing draught of larger vessels, the competitiveness of caisson structures (as combined breakwaters and quay walls) becomes particularly obvious for larger strategic ports in greater water depths, where the costs of any other type of structure become more prohibitive. As a caisson can also easily be given any shape and constructional features (e.g. perforations) to reduce the impact on the environment and to adapt it for multi-purpose use, more acceptance of such types of structures is expected.

Despite the extensive research efforts at interdisciplinary and multinational level such as EU-Projects PROVERBS (e.g. Oumeraci et al. (2001)) and LIMAS (e.g. Kudella et al. (2006)) and despite recent advances in numerical modelling (e.g. Ülker et al. (2012); Jianhong et al. (2014)), 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, which consists of irreversible small soil deformations and subsequent small residual displacements of the caisson which develop incrementally under a series of wave impact loads. In fact, no reliable model yet exists to predict this type of failure observed in the laboratory [Oumeraci et al., 2001] and under field conditions [Oumeraci, 1994], including its role in the often observed seaward tilt of vertical breakwaters [Oumeraci et al., 2001]. 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.

Figure 1 illustrates typical results from caisson breakwater tests in the Large Wave Flume (GWK) in Hanover [Kudella et al., 2006]. Considering regular (repetitive) breaking waves hitting the structure, for each wave event, a small irreversible (residual) vertical displacement ($y_{v,b}$) is recorded together with the associated pore pressure as exemplarily shown in Figure 1 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.

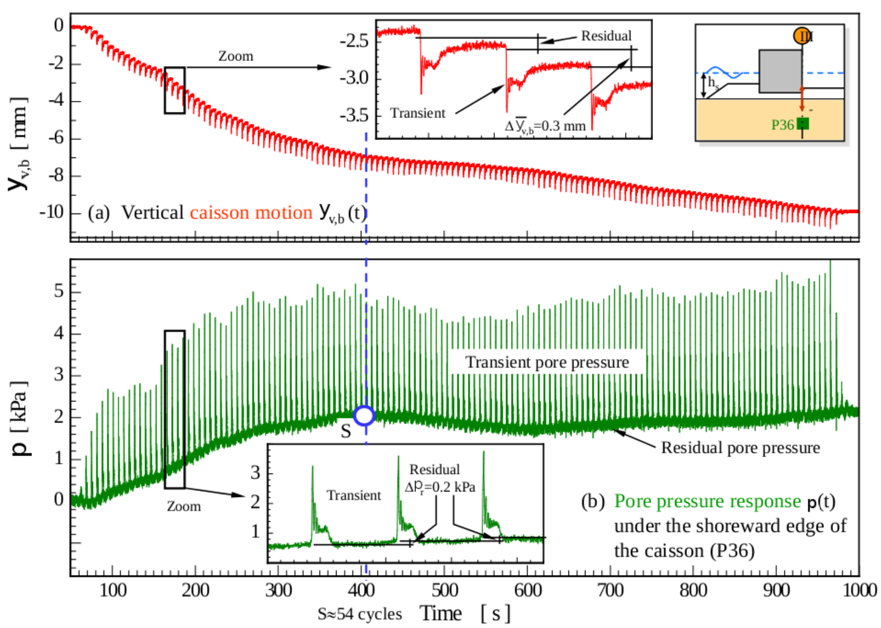


Figure 1: Typical GWK test results [Kudella et al., 2006]

The main objectives of this study are to enhance the understanding of the stepwise failure mechanism for marine gravity structures and, based on the enhanced understanding, to develop a reliable simplified model thereof as well as to draw implications for the engineering design practice. To achieve these objectives, a new CFD-CSD model system [Elsafti, 2015] is developed in OpenFOAM with the main focus on the stepwise failure mechanism. The components of the model system are systematically validated and the entire model system is validated with the large-scale GWK tests. Further, the CFD-CSD model is used for a parameter study to extend the range of available results from the GWK tests.

2. THE CFD-CSD MODEL SYSTEM

The stepwise failure mechanism is a result of the highly complex processes associated with wave-structure-seabed interaction. A one-way coupled CFD-CSD model system is developed in OpenFOAM, to reproduce relevant processes (e.g. breaking wave impact and soil response to cyclic loading). The CFD model is an extension of the incompressible multiphase Eulerian solver of OpenFOAM by introducing different seepage laws for non-deformable porous media. Moreover, a term is added in the continuity equation to account for fluid compressibility without noticeable increase in computational time. The extended CFD model is successfully applied to reproduce measured breaking wave impact loads including the effect of entrapped air as compared to incompressible solvers, Figure 2.

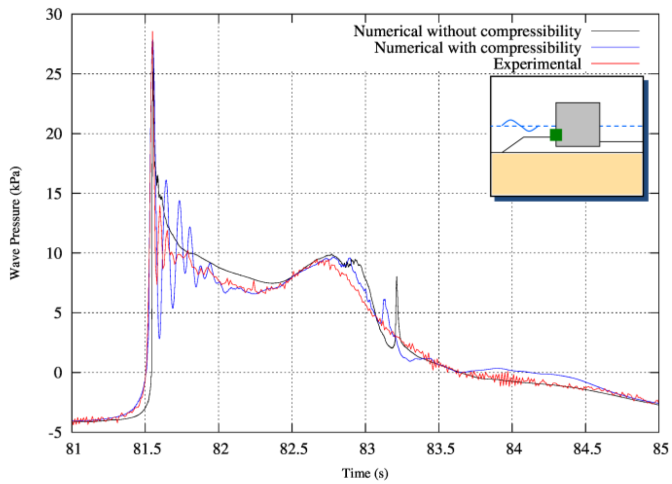


Figure 2: Modelling large-scale GWK tests; breaking wave impact

A new CSD model is developed to solve the fully coupled, fully dynamic Biot equations. A new approach to solve these equations is implemented taking advantage of the segregated approach and using the PISO algorithm to resolve pore fluid velocity-pressure coupling. Soil-structure interaction is introduced via a frictional contact model that can simulate sliding, separation and reattachment of the structure and the underlying soil. A multi-surface plasticity model is implemented to reproduce the most relevant aspects of the response of soil beneath a marine gravity structure (e.g. cyclic mobility). The CSD model is validated against five benchmark cases and two sets of laboratory experiments (centrifuge tests of an embankment foundation and a rocking plate on a sand box from Sumer et al. (2008), Figure 3). The model succeeded to reproduce measured cyclic loading induced residual pore pressure buildup and soil densification followed by pore pressure dissipation.

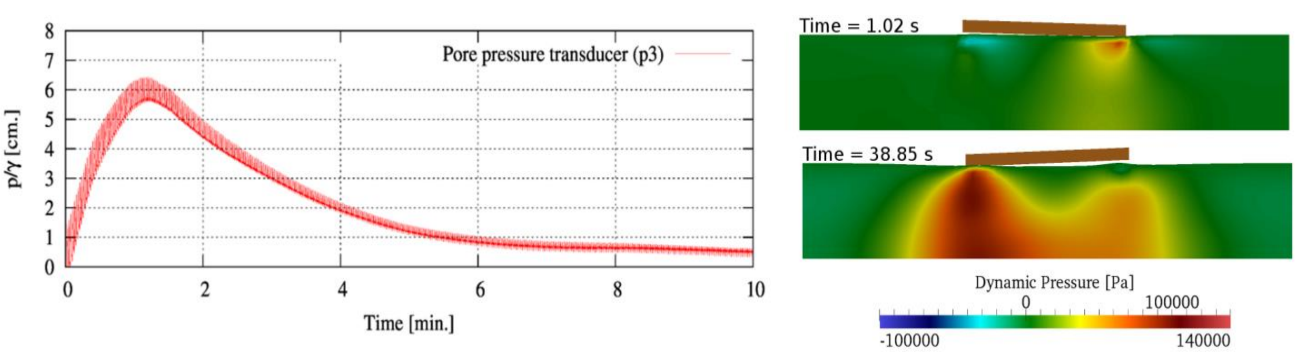


Figure 3: Excess pore pressure inside sand under a rocking plate

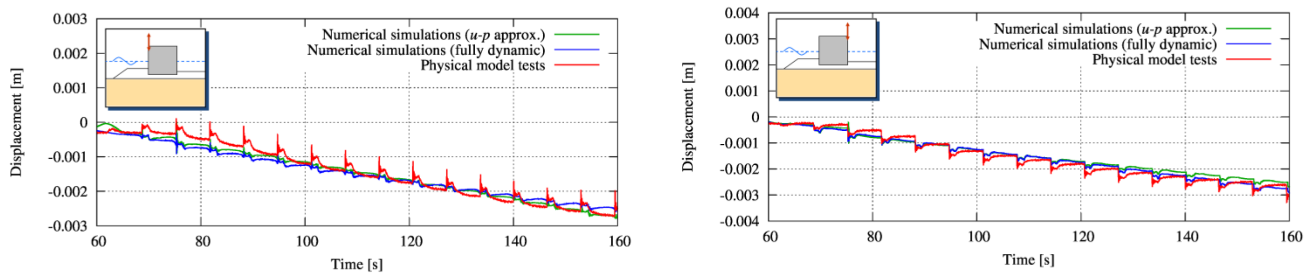


Figure 4: Vertical displacement at the top of the caisson breakwater

A one-way coupling of both CFD and CSD models is implemented, which transforms the CFD model output into input for the CSD model. The CFD-CSD model system is applied successfully to reproduce selected results of the structural response of a caisson breakwater (from the GWK tests) subject to breaking wave impact loads [Elsafti, 2015], as shown in Figure 4.

3. ANALYSIS OF THE RESULTS

For the analysis of the behaviour of gravity structures subject to water wave loading, a representative parameter is proposed to account for the wave loading (horizontal and uplift) and the structural cross sectional properties (geometry and mass). This parameter is the maximum shoreward eccentricity of the vertical load resultant F_V under wave loads, which will be further referred to as 'load eccentricity'. The load eccentricity parameter is illustrated in Figure 5, in which B is the width of the structure-foundation interface (caisson base), M is the total rotating moment calculated at the middle of width B by considering maximum horizontal wave force F_H, corresponding wave-induced uplift force F_U and caisson effective own weight W' (including bouyancy effects).

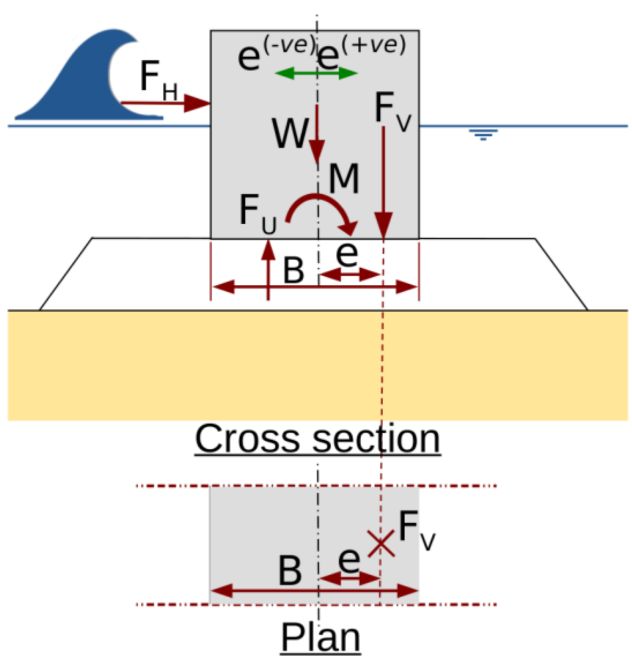


Figure 5: Sketch illustrating the load eccentricity parameter (positive eccentricity is shoreward of base center)

The load eccentricity is calculated as $e = \frac{M}{F_V}$; where $F_V = W' - F_U$. The load eccentricity is further normalised by the structure-foundation interface width B to get the relative load eccentricity $\frac{e}{B}$. For this study, the relative load eccentricity ($\frac{e}{B}$) is calculated for the GWK tests from CFD simulations. Two examples for calculating the relative load eccentricity for GWK tests are given in Figure 6. The two examples represent low eccentricity (non-breaking and slightly breaking waves) and high eccentricity (breaking wave impact). Further, eccentricity of breakwater's own weight will be referred to as the own-weight eccentricity e^{own} (including eccentricity from possible tilt caused during breakwater placement).

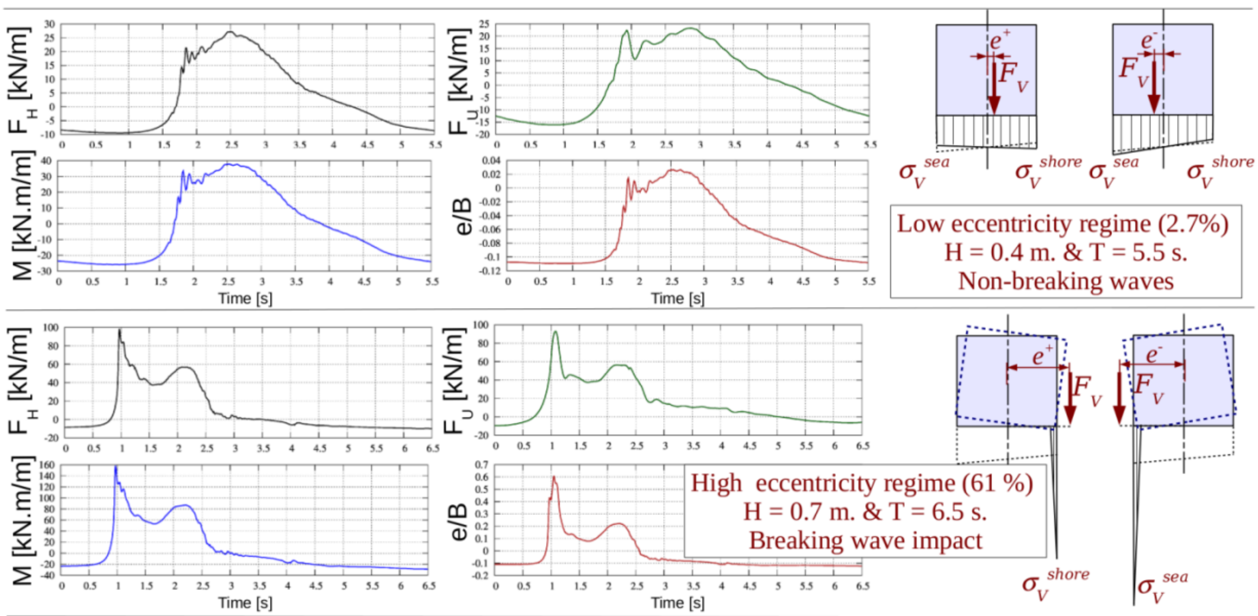


Figure 6: Relative load eccentricity $\left(\frac{e}{B}\right)$ calculated from CFD simulations of selected GWK tests with $\frac{e^{own}}{B} = -8\%$

In fact, the 'relative load eccentricity' $\left(\frac{e}{B}\right)$ can provide a generic method to describe wave loads on gravity structures, implicitly including the effects of overtopping and other factors affecting wave loading on the structure (e.g. forces introduced by side berms). $\frac{e}{B}$ is used to develop a tentative classification system for stepwise failure of marine gravity structures named 'the load eccentricity concept'. For the development of the load eccentricity concept, a parameter study using the developed CFD-CSD model system for $\frac{e}{B}$ =1.6 % - 200 % and four relative sand densities D_r: loose (15 %-35 %), medium (35 %-65 %), medium dense (65 %-85 %) and dense (85 %-100 %), Figure 7. Typical values for sand properties are considered based on the value of D_r. The effect of seabed erosion was eliminated in the GWK tests by placing a sheet underneath the rubble foundation. Therefore, seabed erosion is likewise not considered in the CFD-CSD model and the focus is set on soil plasticity as the most relevant aspect to the stepwise failure mechanism.

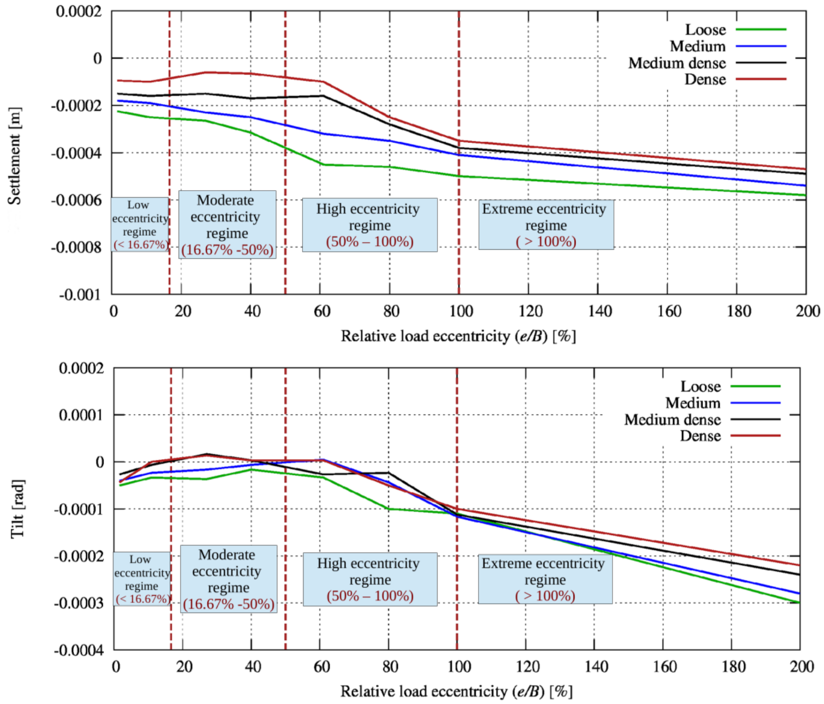


Figure 7: Residual response of marine gravity structures from numerical parameter study for a period of 100 s. for each test with $\frac{e^{own}}{B} = -8\%$

According to the load eccentricity concept, the response of marine gravity structures can be classified in four eccentricity regimes:

i. **Low eccentricity regime (*e*/*B* < 16.67 %):** Generally associated with quasi-static wave loads (Figure 8) causing no structure-foundation separation and no zero vertical stresses under either caisson edges ($e < B/6$), here e^{own} has the highest effect on the exerted stresses under both sides of the caisson (Figure 9a) and is the governing factor for the wave-induced tilt direction (Figure 9c). For $e^{own} \geq 0$, the difference between induced stresses under both edges of the structure is higher when the wave crest is at the structure resulting in shoreward tilt. On the other hand, for $e^{own} < 0$ the difference is higher when the wave trough is at the structure resulting in seaward tilt. Residual displacements in the low eccentricity regime are relatively small.

ii. **Moderate eccentricity regime (*e*/*B* = 16.67 % - 50 %):** For the moderate, high and extreme regimes, partial loss of contact between the structure and its foundation is expected at wave impact. The three regimes are associated with the dynamic force peak rather than the quasi-static part of loading (Figure 8). For these regimes, the structural-induced stresses on its foundation can be idealised as two concentrated vertical forces: under shoreward edge for the moment of impact F_{max}^{shore} and an *inertial* force under the seaward edge (just after impact) when the structure restores its pre-impact position $F_{\max}^{sea}$, Figures 9b and 6. For the moderate eccentricity regime $F_{max}^{shore} > F_{max}^{sea}$ (Figure 9d) resulting in a higher tendency of the structure to tilt in the shoreward direction.

iii. **High eccentricity regime (*e*/*B* = 50 % - 100 %):** Relatively violent breaking wave impact causing increase in inertial force under the seaward edge so that $F_{max}^{shore} < F_{max}^{sea}$ (Figure 9e) resulting in a higher tendency of the structure to tilt in the seaward direction.

iv. **Extreme eccentricity regime (*e*/*B* > 100 %):** Extreme breaking wave impacts most likely causing excessive sliding. The coupling between sliding and rotation of the structure at impact leads to erosion of the rubble foundation underlying the shoreward edge (Figure 10). This process results in a higher tendency of the structure to tilt in the shoreward direction. The extreme eccentricity regime may be considered to cause catastrophic rather than stepwise failure of the structure.

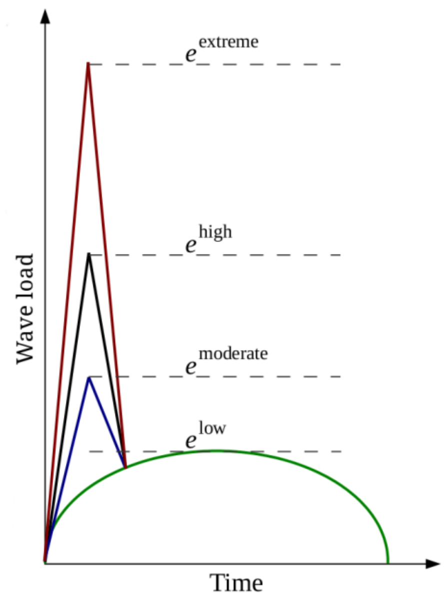


Figure 8: Eccentricity regimes related to wave load history (schematic)

From the load eccentricity concept, it can be inferred that marine gravity structures should not be designed for extreme eccentricity conditions resulting in catastrophic failures. Indeed, many of the stepwise failures of vertical breakwaters reported by Oumeraci (1994) resulted in seaward tilt, and therefore most likely belong to structures that were subject to repetitive high eccentricity regime wave loading.

By considering the transient processes for a single breaking wave event (high load eccentricity) on a caisson breakwater (Figure 11), they can be directly linked to the increase in settlement and tilt. It is shown how the structural response and pore pressure is much higher at wave impact compared to the subsequent structural rocking motion. Therefore, the rocking motion post-impact is of less significance to the stepwise failure. The complete development of the seaward tilt is only realised after a consolidation process that takes place at the end of the wave event. From the results of the GWK tests and numerical simulation, it is observed that the pore pressure amplitude is smaller and the consolidation process is slower under the seaward edge of the structure, which may suggest a smaller degree of saturation (smaller pore fluid bulk modulus) for the foundation underneath the seaward edge as compared to the shoreward edge. The increase of air content in the foundation underneath the seaward edge can be linked to high entrapped/entrained air in wave breaking and may also be affected by structure-induced loading of the foundation.

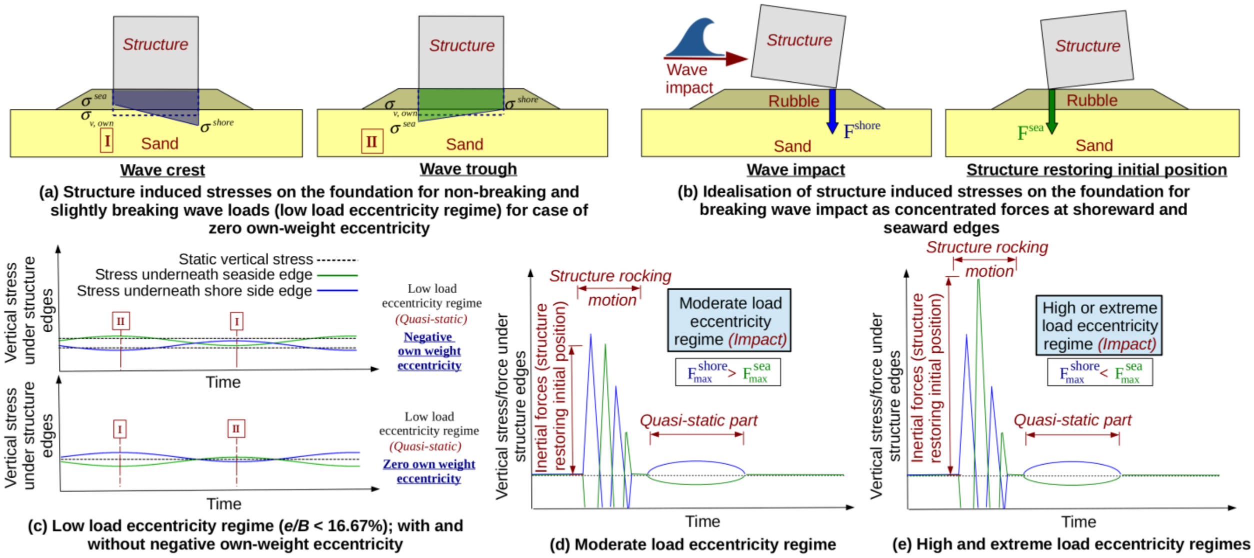


Figure 9: Idealisation of structure-induced stresses/ forces under both edges for different load eccentricity regimes

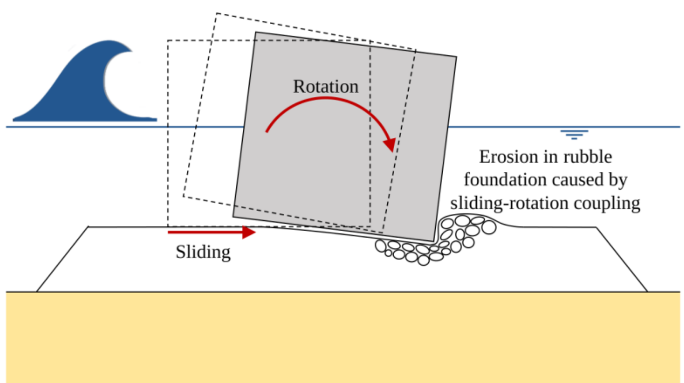
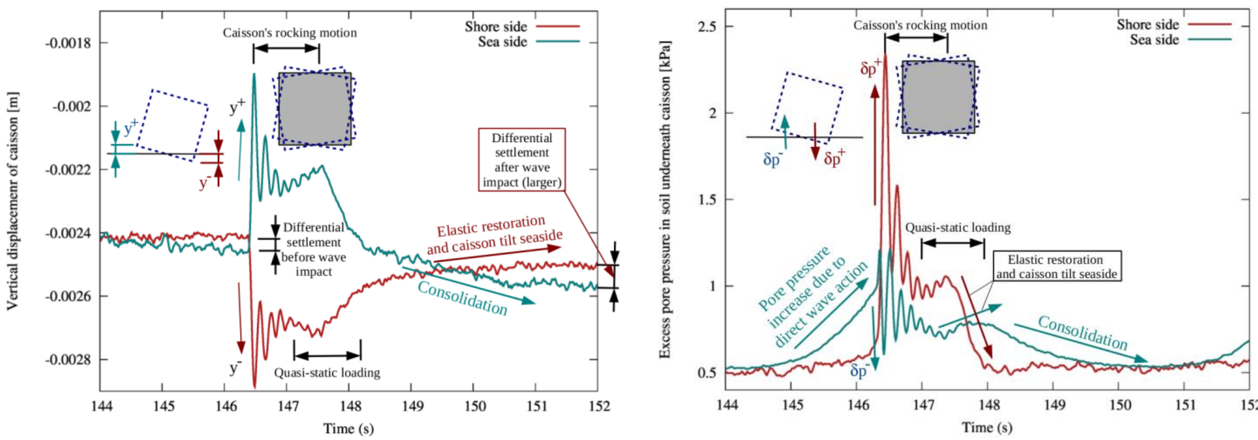


Figure 10: Extreme eccentricity regime: coupled sliding-rotation and induced foundation erosion (not reproduced by the numerical model)

Cyclic loading-induced residual pore pressure in soil is caused by soil plastic volumetric strain. The high eccentricity regime is often associated with asymmetric cyclic loading and hence, every wave event is most likely to contribute a small increase in residual pore pressure. The positive residual pore pressure development (caused by plastic soil contraction) in the sand foundation was found almost equal underneath both edges of the caisson, in both the GWK tests and the numerical simulations. Although pore pressure build-up contributes significantly to reduction in soil strength (partial liquefaction), its contribution to the direction of tilt is less obvious. Kudella et al. (2006) found that nontrivial residual pore pressure develops only in case of regular breaking wave impacts, even under unfavourable drainage conditions. Therefore, the effect of residual pore pressure build-up on the breakwater response would be much more apparent in controlled tests of regular breaking wave impacts as compared to natural sea states.



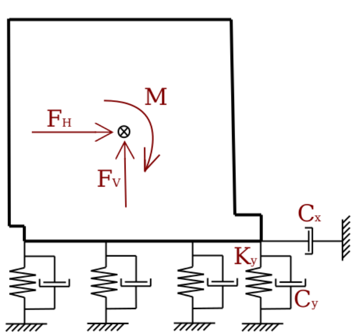
(a) Vertical displacements at top of caisson

(b) Pore pressure inside underlying seabed soil

Figure 11: Transient response for a caisson breakwater subject to breaking wave impact (high eccentricity regime $\frac{e}{B} = 61\%$)

4. SIMPLIFIED MODEL FOR STEPWISE FAILURE

A simplified 3-DOF mass-spring-dashpot model for marine gravity structures is developed with the focus on reproducing the permanent displacements from stepwise failure (sliding, settlement and tilt), which provides a time dependent framework for dynamic analysis. The model is implemented in the OpenFOAM framework as a new solver named 'caissonFoam'. In this model, the structure is assumed rigid and considered as a single mass with three degrees of freedom: vertical, horizontal and rocking motions. The structure is supported by an arbitrary number of vertical supports (springs and dashpots) at the structure-foundation interface to simulate soil-structure interaction for vertical and rotational motions. A horizontal dashpot is used to represent horizontal friction resistance against sliding (viscous friction) that is activated and deactivated according to Coulomb's law. A sketch of the model concept and the governing equation of motion are given in Figure 12.



$$\begin{bmatrix} m_x & 0 & 0 \\ 0 & m_y & 0 \\ m_x y_P & m_y x_P & I_z \end{bmatrix} \begin{Bmatrix} \frac{d^2 u_x}{dt^2} \\ \frac{d^2 u_y}{dt^2} \\ \frac{d^2 \theta}{dt^2} \end{Bmatrix} + \begin{bmatrix} c_x & 0 & 0 \\ 0 & \sum_{i=1}^{n} c_{y_i} & -\sum_{i=1}^{n} c_{y_i} x_{P_i} \\ 0 & -\sum_{i=1}^{n} c_{y_i} x_{P_i} & \sum_{i=1}^{n} c_{y_i} x_{P_i}^2 \end{bmatrix} \begin{Bmatrix} \frac{du_x}{dt} \\ \frac{du_y}{dt} \\ \frac{d\theta}{dt} \end{Bmatrix} + \begin{bmatrix} 0 & 0 & 0 \\ 0 & \sum_{i=1}^{n} K_{y_i} & -\sum_{i=1}^{n} K_{y_i} x_{P_i} \\ 0 & -\sum_{i=1}^{n} K_{y_i} x_{P_i} & \sum_{i=1}^{n} K_{y_i} x_{P_i}^2 \end{bmatrix} \begin{Bmatrix} u_x \\ u_y \\ \theta \end{Bmatrix} = \begin{Bmatrix} F_H \\ F_V \\ M \end{Bmatrix}$$

Figure 12: Concept of the simplified 'caissonFoam' model and its equation of motion

where F_H and F_V are resultants of horizontal and vertical forces acting on the structure, respectively, M is the rotating moment acting at the current rotation pivot, u_x and u_y are the horizontal and vertical displacements of the structure as a rigid body and θ is the structure's rotation at its current pivot. The horizontal and vertical distances between the structure centre of gravity and the current pivot location are x_p and y_p, respectively. Additionally, x_{p_i} is the horizontal distance between the i^{th} vertical support (spring and dashpot) and the current pivot. The structure is supported by n number of vertical supports. m is the mass including the hydrodynamic mass, I_z is the moment of inertia at location of pivot. No geodynamic mass is needed in this model because the elastoplastic springs are fit to results. The mass matrix elements can be calculated as:

$$
\begin{aligned}
m_x &= m_{caisson} + 0.543\ \rho_w d^2 \\
m_y &= m_{caisson} \\
I_z &= I_{CG} + \Delta I + 0.218\ \rho_w d^2
\end{aligned}
\tag{1}
$$

where $m_{caisson}$ is the caisson mass, I_{CG} is the moment of inertia at the CG (Centre of Gravity), ΔI is the difference between moment of inertia at the pivot and at the CG, d is the water depth in front of the caisson and ρ_w is the water density.

The structure cross section is input as the coordinates of any number of vertices and 'caissonFoam' calculates the cross sectional properties and CG position for the given polygon. The horizontal and vertical forces and moment from wave loads are calculated at the CG using the parameter map by Kortenhaus and Oumeraci (1998), Figure 13. Figure 14 shows a comparison between forces from 'caissonFoam' and CFD simulations for non-breaking and breaking waves. For non-breaking and slightly breaking wave loads, the time variation of the forces was considered harmonic with an amplitude calculated from maximum or minimum value of force (e.g. from the Sainflou method). For breaking wave impact, the time history of the impact is considered according to the triangular shape suggested by the PROVERBS method [Oumeraci et al., 2001]. The model uses linear superposition to calculate the resultants (F_H, F_V and M) of wave loads on a caisson breakwater caused by many wave components to simulate irregular waves. Alternatively, the forces time history calculated at the CG can be given directly to 'caissonFoam' (e.g. from CFD simulations for unconventional cross sections).

The 'caissonFoam' model has nonlinear features, for which the model parameters are solution dependent. Therefore, for each time step the solution is iterated until a given tolerance is satisfied. The model nonlinearity is induced by the following:

i. Activation/deactivation of the horizontal dashpot according to the horizontal and vertical forces (sliding of caisson)
ii. Dependency of the damping coefficient of the horizontal dashpot upon the horizontal velocity of the caisson (sliding) motion as well as upon the horizontal and vertical force resultants
iii. Activation/deactivation of the vertical springs/dashpots based on the vertical displacement at each support to simulate soil-structure separation/reattachment
iv. Nonlinear vertical springs with the stiffness coefficient updated each time step for a spring reaction which is nonlinearly dependent upon the vertical displacement and its incremental increase rate at the position of the spring
v. Force-displacement relationship for the vertical springs with two expressions for loading and unloading conditions to describe residual displacements under cyclic loading. Further, dependency of spring parameters on the number of loading cycles to account for soil densification
vi. Location of pivot changes according to caisson motion. Consequently, the rotational moment and the moment of inertia are updated accordingly

The friction at the soil-structure interface is modelled via a horizontal dashpot which is only activated (nonzero damping coefficient) if the horizontal wave-induced force is larger than the static friction resistance at the contact surface. If the friction dashpot is activated, the damping coefficient is calculated from the caisson horizontal speed and the dynamic friction coefficient. The damping coefficient of the horizontal friction dashpot is calculated as:

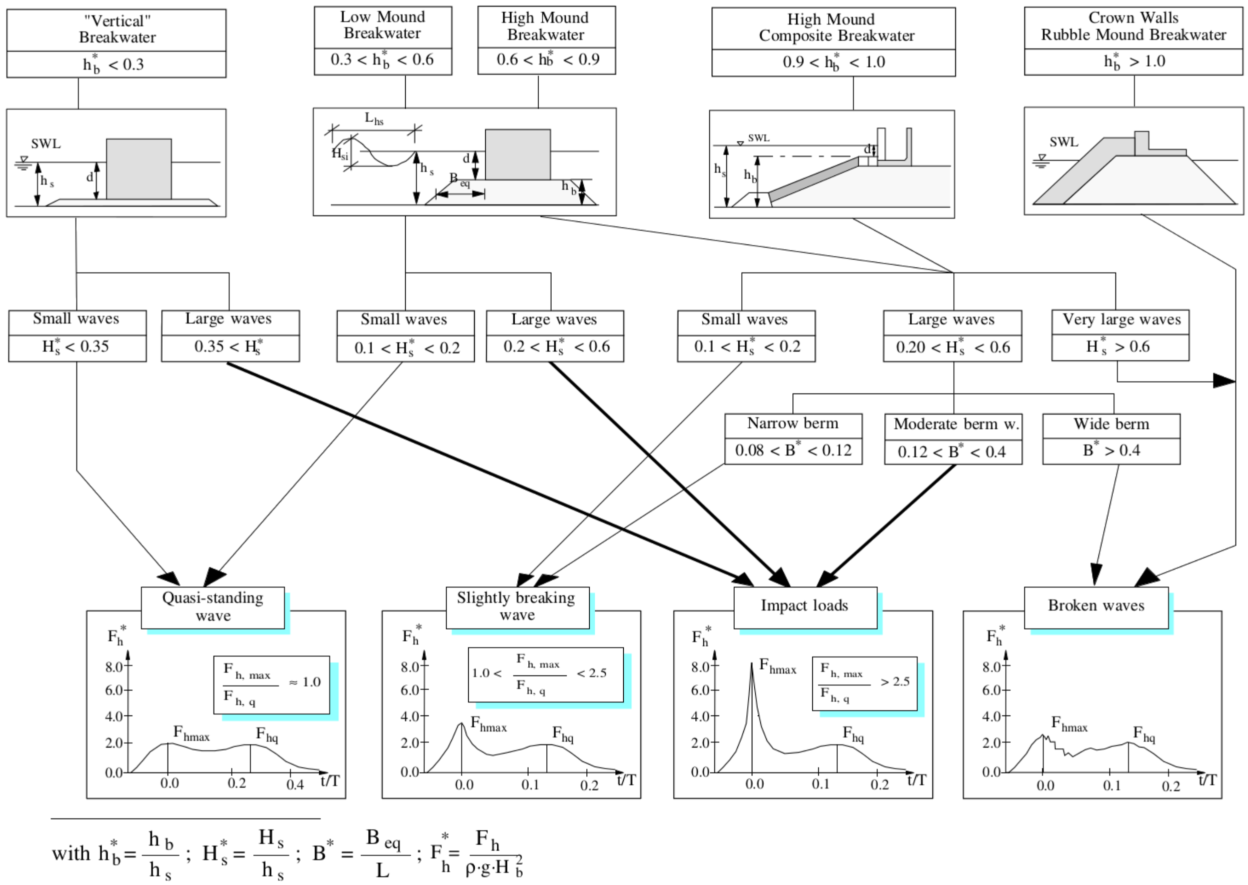
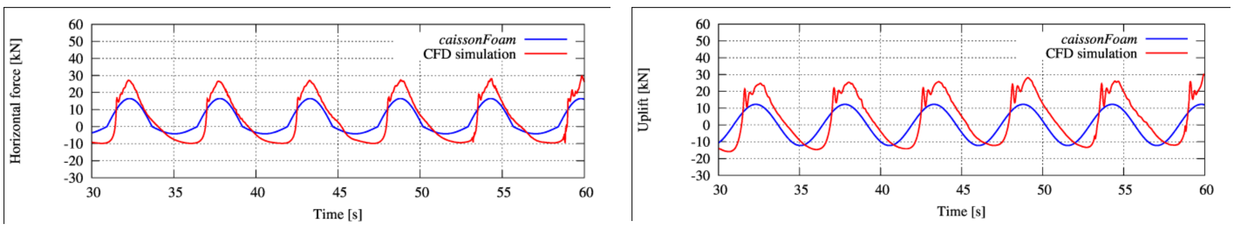


Figure 13: Parameter map for selecting method to calculate wave forces on monolithic structures: H_{si} is the incident breaking wave height, H_b is the breaking wave height, L_{h_s} is wave length at water depth h_s, h_b is height of rubble mound, B_{eq} is length starting from $h_b/2$ to caisson foot and F_h is horizontal wave force [Kortenhaus and Oumeraci, 1998]

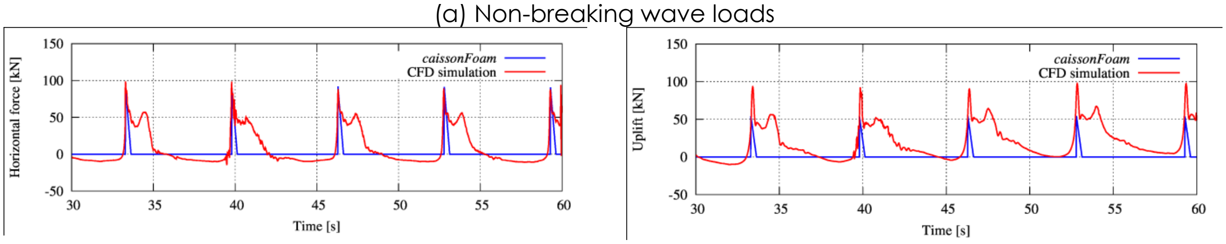
$$c_x = \begin{cases} \frac{\left(W'_{caisson} - F_{uplift}\right)\mu_{dynamic}}{v_x}, & \text{if } F_H > F_{friction} \text{ and } |v_x| > 0 \\ 0, & \text{if } v_x = 0 \end{cases} \tag{2}$$

where $F_{friction}$ is the friction resistance of the soil-structure interface that is calculated from the vertical force ($W'_{caisson} - F_{uplift}$; where $W'_{caisson}$ is the effective weight of caisson) multiplied by a static/dynamic friction coefficient (μ), depending on the current state of the caisson breakwater. The horizontal support can successfully reproduce stepwise sliding of caisson subject to breaking wave impact (Figure 15).

The soil-structure separation and reattachment is simulated by deactivating and reactivating vertical springs and dashpots depending on the current caisson base position in relation to the current underlying foundation surface (including residual displacement) for each spring. In this model, the location of the pivot is assumed to change according to the rotation of the structure and according to the characteristics of the foundation-structure interface. This change is accompanied by changes in the calculated rotating moment and the moment of inertia (both calculated at the current pivot). Although, the pivot location may change continuously, in this model discrete locations of the pivot are considered: (i) the caisson base seaward edge, (ii) the caisson base shoreward edge, (iii) the mid-point of active soil supports (springs under compression) and (iv) the CG of the caisson (if caisson-foundation contact is completely lost), Figure 16. Changing the location of the pivot (according to the afore-mentioned criterion) reduces the amplitude of the computed rocking motion by one order of magnitude, compared to considering a fixed location of the pivot at the mid-point of the structural base.



(a) Non-breaking wave loads



(b) Breaking wave impact loads

Figure 14: Comparison between empirical forces from 'caissonFoam' and forces from CFD simulations for GWK tests

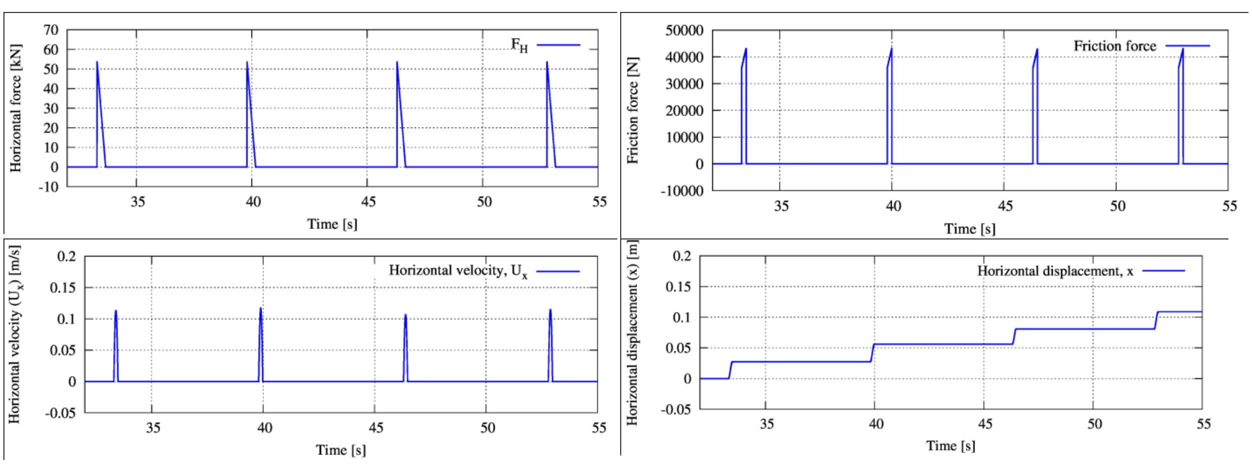


Figure 15: Sliding of a caisson breakwater subject to breaking wave impact simulated by 'caissonFoam'

The concept of the nonlinear elastoplastic springs (with focus on soil response to cyclic loading) is illustrated in Figure 17. For the i^{th} spring, P_i^t and y_i^t are the force and vertical displacement at time t (displacement is positive upward), $P_i^{t-\Delta t}$ is the force from the previous time step, y_i^r is the accumulated residual displacement, y_y is the yield displacement considered as -0.20 m, P_{ult} is the ultimate spring reaction (after which perfect plasticity is reached), P_i^* is the force at which unloading started, y_i^* is the displacement at which the loading/reloading phase started and y_i^{**} is the displacement at which the unloading phase started. The n_l and n_u parameters are fitted to results from parameter study and results of physical model tests for loading and unloading phases.

ζ is a loading flag; one for loading and zero for unloading. Soil densification is simulated by increasing the spring ultimate load as:

$$P_{ult} = P_{ult}^{0} \left(N_{cycle}\right)^{\beta} \tag{3}$$

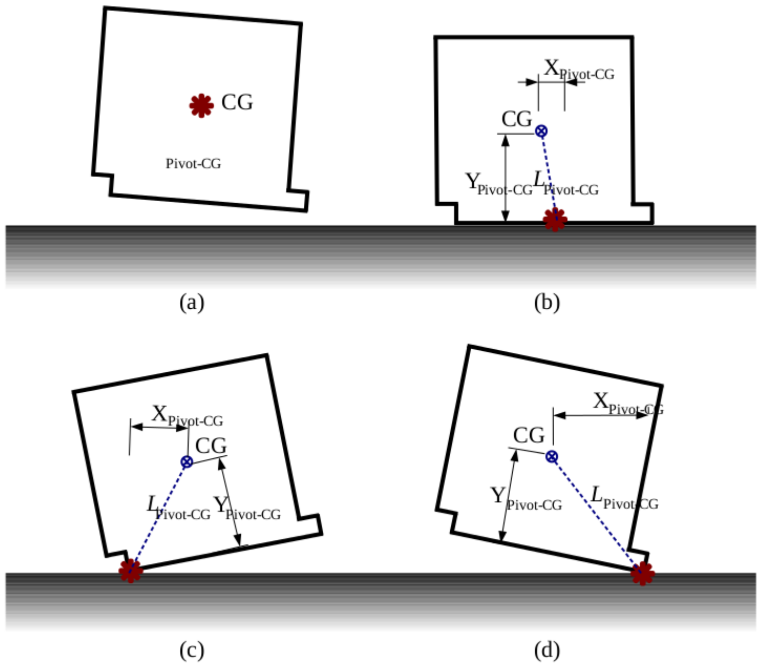
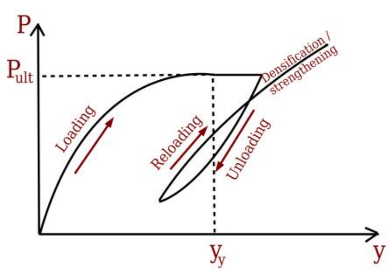


Figure 16: Criterion for selecting the location of the pivot: (a) caisson is totally separated from the foundation, the CG is the pivot, (b) Caisson resting on the foundation, pivot is the middle of activated supports, (c) rotation around seaward edge of caisson base and (d) rotation around shoreward edge of caisson base



$$P_i^t = \begin{cases} 0 & \text{, if } y_i^t \geq y_i^r \text{ (tension cutoff)} \\ P_i^{t-\Delta t} & \text{, if } \Delta y_i = 0 \text{ (constant load)} \\ P_{ult} & \text{, if } y_i^t \leq y_y \text{ (perfect plasticity)} \\ P_{ult}\left(\frac{y_i^t - y_i^*}{y_y}\right)^{\frac{1}{n_l}} & \text{, if } y_y < y_i^t < y_i^r \text{ and } \zeta = 1 \text{ (loading)} \\ P_i^* - P_{ult}\left(\frac{y_i^{**} - y_i^t}{y_y}\right)^{\frac{1}{n_u}} & \text{, if } y_y < y_i^t < y_i^r \text{ and } \zeta = 0 \text{ (unloading)} \end{cases}$$

Figure 17: Concept of the nonlinear elastoplastic spring for 'caissonFoam'

where P_{ult}^0 is the initial spring ultimate load, calculated according to ultimate bearing capacity by DIN-4017 for continuous shallow footings multiplied by the width of the structure base assigned to the i^{th} support, B_i. The densification depends on the number of loading cycles N_{cycles}. Based on available results, the model parameters were calibrated for ten vertical supports. Recommended values for the model parameters are given in Table 1. The damping coefficient for any vertical dashpot is given as:

$$c_{y_i} = \chi \frac{B_i}{2}\left(1 + 4\nu^2\right)(3.5 - 2\nu)\sqrt{\rho_s G} \tag{4}$$

where G is the soil shear modulus, ρ_s is the density of the soil solid phase, υ is Poisson's ratio and χ is an additional factor of the damping coefficient considered as 15. The model was applied successfully to reproduce stepwise failure of the GWK tests of a caisson breakwater subject to breaking wave impacts (Figure 18).

Soil relative density [%]	Parameters	Relative eccentricity ($\frac{e}{B}$) [%]			
		11 %	61 %	100 %	200 %
loose (15 % - 35 %)	n_l	0.999	0.999	0.999	0.999
	n_u	1.001	1.0016	1.0021	1.0028
$\phi = 29^o$	β	-0.002	-0.005	-0.008	-0.01
medium (35 % - 65 %)	n_l	0.999	0.999	0.999	0.999
	n_u	1.0008	1.001	1.0016	1.002
$\phi = 33^o$	β	-0.0008	-0.001	-0.0011	-0.0012
med. dense (65 % - 85 %)	n_l	0.999	0.999	0.999	0.999
	n_u	1.0002	1.0002	1.001	1.0015
$\phi = 37^o$	β	-0.0003	-0.0004	-0.0005	-0.0006
dense (85 % - 100 %)	n_l	0.999	0.999	0.999	0.999
	n_u	1.0001	1.0001	1.0007	1.001
$\phi = 40^o$	β	0.0	0.0	0.0	0.0

Table 1: Recommended values for the simplified model parameters based on calibration using available results

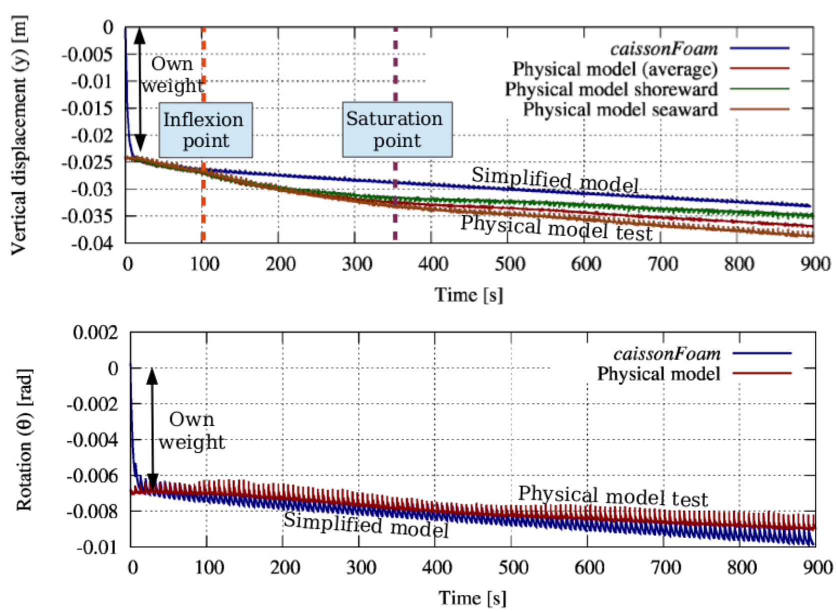


Figure 18: Comparison of breakwater response computed by 'caissonFoam' against GWK measurements for breaking wave impacts

5. SUMMARY AND IMPLICATIONS FOR DESIGN PRACTICE

A new well validated CFD-CSD model system is developed with focus on stepwise failure of marine gravity structures. The model is used to extend the range of available data, which are used in light of the proposed load eccentricity parameter to interpret and classify structural response to wave loading. Further, a simplified model is developed and validated for marine gravity structures that can reproduce sliding, tilt, overturning and settlement. From this study the following can be implied for the design practice of marine gravity structure:

- The developed and well validated CFD-CSD model by Elsafti (2015) can be used for detailed accurate analysis of stepwise failure of marine gravity structures. However, the application of the model is limited by the taxing computational time
- The 'load eccentricity concept' can provide a generic approach for studying and classifying response of gravity structures to wave loads
- According to the load eccentricity concept, it is recommended to avoid designs, for which the structure is subject to extreme load eccentricities. Additionally, most of the field documented stepwise failure cases are most likely caused by high load eccentricities
- Contrary to recommendations in Oumeraci et al. (2001) to consider e^{own} up to -10 %, it was found that negative e^{own} values contribute to seaside tilt (induced by wave loads), most significantly in the low and moderate eccentricity regimes
- Any measure to reduce the effect of stepwise failure of marine gravity structures should contribute to reduction of the load eccentricity. An efficient method would be to consider a convex wall seaside (with possible perforations) to focus the wave forces inside the structure base. Base serrations are recommended to increase sliding resistance. The seaside chamber can be used for Oscillating Water Column devices to enhance cost aspects of the structure, Figure 19
- The simplified 'caissonFoam' model provides a fast tool to analyse stepwise failure of marine gravity structures for practical engineering purposes, even for non-conventional cross-sections (via load eccentricity and forces from CFD simulations)

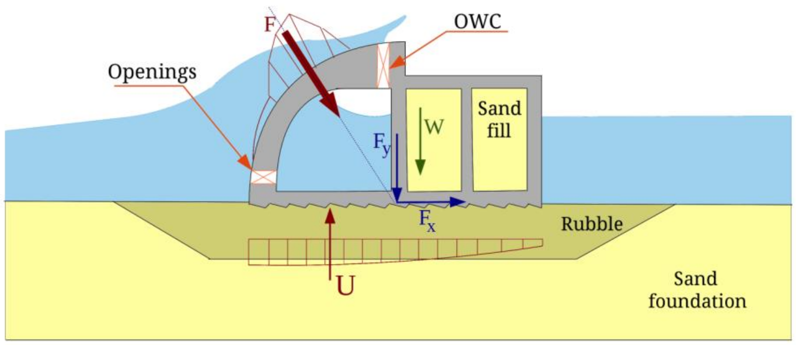


Figure 19: A sketch of an example marine gravity structure with enhanced stability (against sliding/tilting) and multi-purpose use

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SUMMARY

Caisson-type marine gravity structures can be adapted to fulfil any requirements related to their shape, size and multi-purpose use. Given their numerous technical, economic and environmental advantages, more use of such type of structures for many applications is expected. However, they are vulnerable to geotechnical failures from wave attack, especially to the ‘stepwise failure’ mode, in which the structure suffers small irreversible incremental displacements induced by consecutive wave events rather than undergoing a single continuous large displacement resulting in the collapse of the structure.

In this study, a CFD-CSD model system is developed to describe wave-structure-foundation interaction with focus on the stepwise failure mechanism. The model components are systematically validated and the model system is validated using large-scale caisson breakwater tests. The numerical model system is used to extend the range of available data from physical tests. Using the available data, a new interpretation of the stepwise failure mechanism is proposed. Further, a new concept named the ‘load eccentricity concept’ is developed and successfully used to classify the response of gravity structures subject to water wave loads. Moreover, a simplified 3-DOF mass-spring-dashpot model with elastoplastic supports is developed and validated for practical engineering purposes. For the application of the elastoplastic supports in the simplified model, only two variables are needed: the relative load eccentricity and the relative soil density. Finally, implications of the study for the design practice are presented.

RESUME

Les caissons poids utilisés pour les digues en mer peuvent être adaptés pour répondre aux exigences quant à leur forme, leur taille et leurs multiples utilisations. Compte tenu de leurs nombreux avantages techniques, économiques et environnementaux, le développement de l'utilisation de ce type de structures pour des applications variées est attendue. Cela dit, ils sont vulnérables aux défaillances d'ordre géotechnique dues à l'impact de la houle sur la structure, et en particulier au mode de 'défaillance par étapes', dans lequel la structure subit de faibles déplacements irréversibles qui résultent de l'action d'événements successifs qui s'additionnent, contrairement à un seul événement puissant induisant un grand déplacement et pouvant entraîner l'effondrement de la structure.

Cette étude, présente un modèle CFD-CSD (Computational Fluid Dynamics – Computational Structure Dynamics) développé pour décrire l'interaction houle-structure-fondation, avec un intérêt particulier pour le mécanisme de défaillance par étapes. Chaque élément du modèle a été systématiquement validé, et le modèle en lui-même a été testé sur des digues de grande dimension à base de caissons. Le modèle numérique a été utilisé pour générer une grande quantité de données à partir des données disponibles issues des tests physiques. Ces nouvelles données ont permis de proposer une nouvelle interprétation du mécanisme de défaillance par étapes. En outre, un nouveau concept nommé 'concept de charge excentrée' a été développé et utilisé avec succès pour classifier la réponse des structures soumises aux charges de la houle. En outre, un modèle simplifié de type ressort-amortisseur simplifié à 3 degrés de liberté (3-DOF) utilisant des appuis élastoplastiques a été développé et validé pour des questions pratiques d'ingénierie. Ce modèle simplifié, la définition des appuis élastoplastiques ne nécessite que deux variables: l'excentricité relative de la charge et la densité relative du sol. Enfin, des considérations pratiques pour l'utilisation des résultats de l'étude lors de la phase conception sont présentées.

ZUSAMMENFASSUNG

Maritime Schwergewichtsbauwerke in Caisson-Bauweise können für alle Anforderungen in Bezug auf ihre Form, Größe und Mehrzwecknutzung angepasst werden. Angesichts ihrer zahlreichen technischen, wirtschaftlichen und ökologischen Vorteile wird ein häufigerer Einsatz derartiger Konstruktionstypen für viele Anwendungsbereiche erwartet. Sie sind jedoch anfällig für geotechnisches Versagen durch Wellenbelastung. Dies gilt insbesondere für ein „schrittweises" Versagen, wobei das Bauwerk, statt einer einzigen große Verschiebung, kleine stückweise irreversible Verschiebungen erfährt, die durch aufeinanderfolgende Wellenereignisse ausgelöst werden und in einem Einsturz des Bauwerks resultieren.

Mit dem Fokus auf den schrittweisen Versagensmechanismus, wird in dieser Studie ein CFD-CSD Modellsystem entwickelt, um die Interaktion zwischen Wellen, Bauwerk und Gründung zu beschreiben. Die Modellkomponenten werden systematisch validiert und das Modellsystem wird unter Verwendung von großskaligen Tests mit Caisson-Wellenbrechern validiert. Das numerische Modellsystem wird dazu verwendet, die aus physikalischen Untersuchungen verfügbaren Daten zu erweitern. Unter Verwendung der verfügbaren Daten wird eine neue Interpretation des schrittweisen Versagensmechanismus vorgeschlagen. Außerdem wird ein neues Konzept mit der Bezeichnung „Lastexzentrizitätskonzept" entwickelt und erfolgreich angewendet, um die Reaktion von Schwergewichtsbauwerken auf Wellenbelastungen zu klassifizieren. Zusätzlich wird für praktische Ingenieuranwendungen ein vereinfachtes 3-DOF Masse-Feder-Dämpfer Modell mit elasto-plastischen Auflagern entwickelt und validiert. Für die Anwendung der elasto-plastischen Auflagern im vereinfachten Modell werden nur zwei Variablen benötigt: Die relative Lastexzentrizität und die relative Bodendichte. Zum Schluss werden Anwendungsmöglichkeiten der Studie für die Gestaltungspraxis aufgezeigt.

RESUMEN

Las estructuras marítimas construidas mediante cajones pueden adaptarse para cubrir casi cualquier tipo de requerimiento que se les exija, relativo a su forma, tamaño y uso. Dadas sus numerosas ventajas desde los puntos de vista técnico, económico y medioambiental, se espera que el uso este tipo de soluciones tenga un importante desarrollo. En cualquier caso, dichas estructuras resultan vulnerables ante determinados fallos de tipo geotécnico bajo la acción del oleaje, especialmente el denominado "fallo progresivo", en el que la estructura sufre progresivos movimientos incrementales de carácter irreversible inducidos por la sucesiva actuación del oleaje, frente a otro tipo de fallos, que tendrían un carácter inmediato, consistentes en un gran desplazamiento de la estructura que generaría su colapso inmediato por efecto de la acción de una única ola.

En este estudio se ha desarrollado un modelo Computacional de Dinámica de Fluidos (CDF) que permite describir el comportamiento y la interacción del sistema oleaje-estructura-cimentación, centrándose en el modo de fallo progresivo. Las distintas componentes del modelo, y el modelo en sí mismo, son sistemáticamente validadas utilizando ensayos a gran escala de diques de cajones. El modelo numérico se ha utilizado para ampliar el rango de datos procedentes de los modelos físicos. Utilizando los datos disponibles, se propone una nueva interpretación del mecanismo de fallo progresivo. Asimismo, se desarrolla un nuevo concepto, denominado "concepto de carga excéntrica", que se utiliza para clasificar la respuesta de las estructuras de gravedad frente a cargas derivadas del oleaje. Adicionalmente, se ha desarrollado y validado un modelo simplificado 3-DOF masa-muelle-movimiento con apoyos elastoplásticos para su utilización con fines ingenieriles. Para el uso de los apoyos elastoplásticos dentro del modelo simplificado, solo se necesita definir dos variables: la excentricidad relativa de la carga y la densidad relativa del suelo. Finalmente, se presentan algunas implicaciones del estudio para su utilización práctica a efectos de diseño.