Dynamic behavior of a breakwater triggered by seepage, earthquake action, and wave pressure

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Abstract. The dynamic stability analysis of the breakwater is investigated under the actions of seepage, earthquakes, and waves. The research indicates that a non-liquefiable layer is a useful way to prevent the adverse effects of soil liquefaction and to improve the stability of the breakwater. Because of a covering layer of thicker non-liquefied soil, the breakwater will not undergo large liquefied deformation under the coupled actions of seepage, earthquakes, and waves. The dynamic soil pressure response on both sides of the breakwater is more noticeable as the soil depth increases.

1. Introduction

Artificial islands are located at sea, and as such represent complex environments, subject to potentially adverse effects such as waves, ocean currents, wind, ice, and earthquakes. Breakwaters are important enclosure structures designed to protect artificial islands, and an instability of a breakwater will have catastrophic consequences for the artificial island it is built upon.

Many scholars have carried out relevant studies on breakwaters. Matsuda et al. [1] examined the destabilization of caisson-type breakwaters under tsunami-induced seabed damage and used centrifuge model testing, finite-element analysis, and smooth particle hydrodynamic simulations to study the damage mechanisms of breakwaters. Chen et al. [2] presented a failure mode in their stability analysis of breakwaters under seepage flow. In their study, a relationship was obtained between the height of a tsunami and the safety factor of a breakwater. Ye and Wang [3] applied a fully coupled numerical model to investigate the seismic dynamics of a composite breakwater on a liquefiable seabed foundation and found the highly nonlinear dynamic interaction present in an offshore breakwater.

Chaudhary et al. [4, 5] analyzed the stability of a breakwater under the combined effects of an earthquake and tsunami and proposed a pseudo-dynamic method for estimating the seismic inertia forces on breakwaters, to calculate stability. Iwamoto [6] used a coupled Smoothed Particle Hydrodynamics (SPH) – Distinct Element Method (DEM) analysis method to analyze scouring and seepage flow phenomena at caisson-type breakwaters during the tsunami, examined this coupled method in Experimental simulations of caisson-type breakwater failures.

As mentioned above, the stability of breakwaters has been analyzed by numerous methods. However, the majority of these were pseudo-dynamic methods, and unable to consider dynamic characteristics and breakwater stability completely. It can be concluded that the dynamic characteristics and stability of breakwaters need further investigation.

In this study, a modified Hardin–Drnevich model and finite strength reduction method are used to study the dynamic stability of a breakwater in the Hong Kong–Zhuhai–Macao Bridge project [7]. Further, the dynamic characteristics and the stability of this breakwater are studied by considering the
coupled actions of seepage, earthquakes, and waves. This research provides an important reference for engineering construction.

2. Soil-fluid interaction

Based on Biot’s consolidation theory, we can consider the soil-fluid interaction using Darcy’s law [8]. The response equation for the pore fluid can be formulated as follows.

2.1 Transport Law

\[ q_i = -k_i \hat{k}(s)[P - \rho_f x_j g_j], \]  

(1)

Here, \( \rho_f \) is the fluid density; \( k_i \) is the permeability tensor; \( g_j, j = 1, 2, 3 \) are the three components of the gravity vector; \( x_j \) is the projection value in \( j \) direction; \( \hat{k}(s) \) is the relative mobility coefficient and \( P \) is the fluid pore pressure.

2.2 Compatibility Equations

\[ \varepsilon_{ij} = 0.5(v_{ij} + v_{ji}) \]

(2)

Here, \( v \) is velocity.

2.3 Boundary condition

\[ q_n = h(p - p_s) \]

(3)

Here, \( p_s \) is the pore water pressure at the seepage exit; \( q_n \) is the specific discharge normal vector at the boundary; \( P \) is the pore water pressure at the boundary and \( h \) is the leakage factor.

3. Wave action

The velocity potential \( \Phi \) at any point in a field can be expressed as [9]:

\[ \Phi(x, y, z, t) = \Phi_i(x, y, z, t) + \Phi_s(x, y, z, t) \]

(4)

\[ \Phi_i(x, y, z, t) = -i \frac{gH \cosh(kz)}{2\omega \cosh(kd)} e^{i(k\omega - xt)} \]

(5)

Here, the total velocity potential \( \Phi(x, y, z, t) \), the velocity potential of the incident wave \( \Phi_i(x, y, z, t) \), and the velocity potential of the reflected wave \( \Phi_s(x, y, z, t) \) all obey Laplace’s equation. In the above equations, \( \omega = 2\pi \times T^{-1}, \ k = 2\pi \times L^{-1}, \ L \) is the wavelength, \( \omega \) is the frequency, \( z \) is the relative height of the calculated point to the water surface, \( d \) is the water depth and \( T \) is the wave period; \( x \) and \( y \) are the exact location of the calculation point, respectively; \( t \) is the time; \( i \) is the imaginary part of the complex number. To convert Eq. (5) into an \( N \)-order Bessel function, the velocity potential of the incident wave \( \Phi_i \) can be expressed as

\[ \Phi_i = -i \frac{gH \cosh(kz)}{2\omega \cosh(kd)} \left[ \sum_{j=0}^{\infty} \varepsilon_j J_j(\kappa r) \cos(\eta \theta) \right] e^{i\omega t} \]

(6)

Where \( j = 0 \) and \( \varepsilon_0 = 1 \). When \( j > 0 \) and \( \varepsilon_j = 2 \), \( J_j(\kappa r) \) is a Bessel function of the first kind. At the same time, \( \Phi_s(x, y, z, t) \) should also satisfy the Sommerfeld radiation condition [10]:

\[ \lim_{r \to \infty} \sqrt{r} \left( \frac{\partial \Phi_s}{\partial r} - ik\Phi_s \right) = 0 \]

(7)
The total velocity potential $\Phi$ can be expressed as follows.

$$\Phi = G(z,t) \left[ \sum_{j=0}^{\infty} \beta J_j(kr)\cos(j\theta) + \sum_{j=0}^{\infty} \beta B J_j(kr)\cos(j\theta) \right]$$

(8)

Based on the Bernoulli equation, the dynamic water pressure $P_1$ can be calculated as

$$P_1 = \rho \frac{\partial \Phi}{\partial t}$$

(9)

4. Test case

The breakwater environment has smooth ocean floors and a simple underwater landscape, which consists of clay soil, backfill sand, and compaction sand, proportionately (as shown in Figure 1(b)). The breakwater is composed of plug-in steel cylinder structure. The steel cylinder is 40 m long, with a thickness of 16 mm and a diameter of 22 m. The top and bottom elevations of the cylinder head are 3.5 m and -36.5 m, respectively. The top level of the soil layer is -8.5 m, and the water level is 2.57 m, as shown in Figure 1(a).

The numerical model is 190 m long and 80.5 m high. The water level for the breakwater is 2.57 m, and the artificial pumping water level is -12.5 m on the inland side. To improve the calculation efficiency and reduce the grid size, a typical cross section was selected in calculation. The cross-sectional element was chosen to simulate the interactions between the cylinder and surrounding soils. The backfill and compaction sand are considered as liquefiable soil, and the parameters are shown in Table 1. and Table 2.

![Figure 1](https://example.com/figure1.png)

(a) Model dimensions, (b) Calculation model.

Table 1. Geotechnical material parameters.

| Materials       | Dry density $\text{kg/m}^3$ | Elastic modulus $\text{Mpa}$ | Poisson's ratio | Angle of internal friction | Cohesion $\text{kPa}$ |
|-----------------|-----------------------------|-----------------------------|-----------------|---------------------------|----------------------|
| Backfill Sand   | 1380                        | 40                          | 0.35            | 30                        | 4                     |
| Clay            | 1350                        | 30                          | 0.38            | 13                        | 22                   |
| Compaction Sand | 1380                        | 45                          | 0.35            | 32                        | 2                    |
| Cylinder        | 3500                        | 2000                        | 0.17            | Elastic material          |                      |

The typical earthquake intensity for the project area is category VII on the modified Mercalli intensity scale, but considering the importance of the project’s strategic location and economic status, category
VIII is chosen here for an effective earthquake-resistant design. The peak values of the Kobe earthquake (Japan, 1995) were 0.2 g in numerical calculations, as shown in Figure 2.

### Table 2. Material parameters in calculation.

| Materials   | Permeability coefficient cm/s | Porosity | Standard penetration number | Damping ratio | C1  | C2   |
|-------------|-------------------------------|----------|-----------------------------|---------------|-----|------|
| Backfill    | 5.00E-2                       | 0.5      | 12                          | 0.05          | 0.38| 1.028|
| Sand        |                               |          |                             |               |     |      |
| Clay        | 8.70E-6                       | 0.45     | --                          | 0.05          | --  | --   |
| Compactin   | 2.66E-4                       | 0.5      | 15                          | 0.05          | 0.29| 1.356|
| sand        |                               |          |                             |               |     |      |

![Input earthquake wave (Kobe wave)](image)

Figure 2. Input earthquake wave (Kobe wave).

Monitoring points for the breakwater were set up at intervals of 3 m on the inside and outside of the steel cylinder, to record the time-history curve of dynamic earth pressure. The distribution of the dynamic earth pressure is shown in Figure 3. Based on the dynamic earth pressure distribution on both sides of the breakwater under earthquake conditions, it can be concluded that the deeper the buried depth of the breakwater, the greater the dynamic earth pressure. The pressure response on both sides of the breakwater is more notable between 4 and 6 s under earthquake excitation.

![Dynamic earth pressure distribution of the breakwater during earthquakes](image)

Figure 3. Dynamic earth pressure distribution of the breakwater during earthquakes. (a) Left-hand side of the breakwater, (b) Right-hand side of the breakwater.
When the soil is liquefied, the effective stress $\sigma' = 0$, and the total stress $\sigma = u$, where $u$ is the pore water pressure. The excess pore water ratio $r_u$ is used to describe the soil liquefaction and can be formulated as follows.

$$r_u = 1 - \frac{\sigma_m'}{\sigma_{m0}}$$

(10)

Here, $\sigma_{m0}'$ and $\sigma_m'$ represent the initial effective stress and the effective stress during calculation, respectively.

Figure 4 show the pore water pressure response of points A1 and A2. It can be seen that the pore water pressure increased rapidly after 2.5 s, and the dynamic response was considerable. The maximum pore pressure ratio appeared at 9.89 s, and its value was less than 1.0, indicating that the soil here was not liquefied. The reason was that the locations of points A1 and A2 were covered with a thick non-liquefied clay layer, and the input seismic action was also small.

![Figure 4. Pore water pressure and pore pressure ratio of monitoring points A1 and A2. (a) Pore water pressure at A1 and A2, (b) Pore pressure ratio at A1 and A2.](image)

The pore water pressure responses of points B1 and B2 are shown in Figure 5. The pore water pressure also increased rapidly after 2.5 s, and the maximum value of the pore pressure ratio $r_u$ was 0.589. Because $r_u < 1.0$, the nearby soil is not liquefied, which is also due to the thick non-liquefied clay layer.

![Figure 5. Pore water pressure and pore pressure ratio of monitoring points B1, B2. (a) Pore water pressure at B1 and B2, (b) Pore pressure ratio of B1 and B2 points.](image)
5. Conclusions
(1) Because of the thicker non-liquefied layer and small earthquake intensity, the artificial island breakwater will not undergo large liquefied deformation under earthquake conditions.
(2) The deeper the buried depth of the breakwater, the greater the dynamic earth pressure will be. The dynamic earth pressure response is more notable on both sides of the breakwater from 4 to 6 s under earthquake conditions.
(3) The dynamic safety factor of the breakwater, calculated using the strength reduction method, concerns the water level. When the water level is pumped to the design height, it can meet the safety requirements. The ultimate failure surface appeared at the breakwater-soil interface.
(4) A non-liquefiable layer is a useful way to prevent the adverse effects of soil liquefaction and to improve the stability of the breakwater. Due to the thicker non-liquefied layer and small earthquake intensity, the artificial island breakwater will not undergo a large liquefied deformation during an earthquake.

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