ABSTRACT

Seawalls are constructed to defend the shorelines from wave attack. Depending on the ratio of maximum design wave height to the depth of still water at the wall the wave forces acting on the seawall are classified as non-breaking wave force, breaking wave force and broken wave force. Seawall will be subjected to non-breaking wave force, when the depth of water at the wall is greater than 1.5 times the maximum design wave height. In the present analysis a vertical face rigid waterfront retaining wall supporting submerged backfill, and is subjected to non-breaking wave force is considered. Stability analysis has been carried out by considering two cases viz., when wave trough is at the wall, which causes active earth pressure condition, and when wave crest is at the wall, which leads to passive earth pressure condition and the stability is reported in terms of factor of safety against sliding and overturning modes of failures. Sensitivity analysis has been conducted for investigating the effects of different parameters, such as, relative wave height, soil and wall friction angles and height of water on landward side. It is observed that factor of safety against overturning mode of failure decreases by 80% when the ratio of non-breaking wave height to water depth on seaward side changed from 0 to 0.6. The proposed closed form solutions and design charts provide a better guideline for design of seawall subjected to non-breaking waves against sliding and overturning modes of failures.

Keywords: Seawall, stability, non-breaking wave, factor of safety, sliding, overturning

1 INTRODUCTION

Retaining structures are designed for the purpose of supporting a vertical or near-vertical face of soil. The prediction of actual earth pressure acting on the retaining wall is one of the most critical and complex soil-structure interaction problems. The presence of water in front of retaining wall (i.e., in the case of waterfront retaining wall) increases the complexity of the problem, where soil-water-structure interaction comes into picture. The common types of waterfront retaining walls are seawalls, bulkheads, and quay walls. A seawall is an earth retaining structure, which protects the shore from wave attack from sea. These are not intended for berthing facilities. Bulkheads are soil retaining structures, which are not subjected to appreciable wave attack and are commonly used as berthing facilities. When the required wall height is more, then bulkheads are not suitable as they are flexible in nature, so quay walls are more suitable in those cases. Quay wall is a gravity wall structure, which provides both shore protection from light to moderate wave attack and a berthing face for ships. These structures do not necessarily retain a soil backfill. These retaining structures play an important role in ports and harbors, coastal areas. Seawalls will be continuously experiencing wave forces which are time varying in nature. So, first these structures need to be designed for these wave forces which are transient in nature. Depending on the location of seawall, it can be subjected to non-breaking wave, breaking wave and broken wave. Seawall will be subjected to non-breaking waves when depth of water at the structure is greater than about 1.5 times the maximum expected wave height (Shore Protection Manual, 1984). Forces due to non-breaking waves can be approximated as hydrostatic.

Sainflou (1928) conducted the first theoretical study for assessment of wave loads exerted by non-breaking waves on vertical wall. Miche (1944) and Rundgren (1958) improved the work of Sainflou (1928) considering the second order terms in relation to the wave height, which is recommended by shore protection manual (1984). After that several researchers like Tadjbaksh and Keller (1960), Goda (1967), Kachoyan and Mckee (1985), Fenton (1985),
Mallayachari and Sundar (1995) and Goda (2000) studied the non-breaking wave force acting on the vertical wall. The problem of active and passive earth pressures acting against rigid retaining structures had been extensively studied in the literature since 1776 by the pioneering work of Coulomb (1776) [see Terzaghi 1943]. Rankine (1857) [see Terzaghi 1943] presented a different approach to the lateral earth pressure problem. Numerous investigators like Terzaghi (1943), Chen (1975), Basudhar et al. (1979), Chen and Liu (1990), Kobayashi (1998) and few others had also studied the problem of lateral earth pressure. Okabe (1926) and Mononobe and Matsuo (1929) did the pioneering work on seismic lateral earth pressures. In recent past, solutions for computing seismic active earth pressures using limit equilibrium approach are given by Choudhury and Singh (2006), Choudhury and Nimbalkar (2005), Nimbalkar and Choudhury (2007). Seismic analysis of waterfront retaining walls had been studied extensively by Ebeling and Morison (1992), Choudhury and Ahmed (2007a, 2007b), Choudhury and Ahmad (2008), Ahmad and Choudhury (2008) Chakraborty and Choudhury (2014a, 2014b) and few others.

In the present study the stability of a seawall under the action of non-breaking earth pressures is assessed in both active and passive earth pressure conditions for static case by using limit equilibrium method.

2 METHOD OF ANALYSIS

A typical gravity type rigid seawall with vertical face having width ‘b’ and height ‘h’, retaining a horizontal cohesionless backfill to its full height which is submerged with water to a height $h_\text{L}$ is considered.

The height of water on seaward side is $d_\text{S}$. For the present analysis, a non-breaking wave of incident height ‘$H$’ and length ‘$L$’ with its trough and crest at the wall is considered for active and passive earth pressure conditions respectively.

The forces acting on the seawall are non-breaking wave pressure including hydrostatic force ‘$P_\text{stL}$’, active earth thrust ‘$P_\text{a}$’, passive earth resistance ‘$P_\text{p}$’, hydrostatic force ‘$P_\text{stL}$’, weight of the wall ‘$W$’, and uplift force ‘$U_\text{b}$’ at the base as shown in Figures 1 and 2. Calculations of all these mentioned forces are discussed below.

2.1 Force due to non-breaking wave

The Non-breaking wave pressure ($P_\text{w}$) on seawall is calculated as per shore protection manual (1984) using Miche (1944) and Rundgren (1958) formula when wave trough and wave crest at the wall respectively are given as,

$$P_\text{w} = \frac{1}{2}(\gamma_\text{w} \cdot d_\text{S} - P_1)(d_\text{S} + h_\text{w} - H) \tag{1}$$

$$P_\text{w} = \frac{1}{2}(\gamma_\text{w} \cdot d_\text{S} + P_1)(d_\text{S} + h_\text{w} + H) \tag{2}$$

Where $P_1$ is given by

$$P_1 = \left(1 + \chi \right) \frac{\gamma_\text{w} \cdot H}{\cos(2\pi d_\text{S}/L)} \tag{3}$$

In which $\chi$ is a reflection coefficient and is assumed as 1.0 (for vertical wall with complete reflection). It is to note that the non-breaking wave pressure formula mentioned above includes the hydrostatic pressure acting on the seaward side also. The points of applications of the total non-breaking wave pressure, when wave trough and wave crest at the wall are $y$ and $y_c$ respectively.

2.2 Hydrostatic force on landward side

The hydrostatic pressure from landward side ($P_\text{stL}$) acting on the seawall due to the submergence of backfill can be calculated as

$$P_\text{stL} = \frac{1}{2} \gamma_\text{w} \cdot d_\text{L}^2 \tag{4}$$
2.3 Uplift force at the base

The uplift pressure acting at the horizontal base of the wall is computed as

\[ U_b = 0.5(u_L + u_s)b \]  

(5)

Where

- \( u_L = \) pore water pressure on landward side (i.e., \( \gamma \)sludge)
- \( u_s = \) pore water pressure on seaward side (i.e., \( \gamma \)soil)

\[ u = \gamma \cdot d \]

2.4 Lateral earth pressures

When the wave trough is at the wall, the force acting on the seaward side is less than the hydrostatic force acting on the backfill, creating a wave with its crest at the wall, the wall has been pressure condition and similarly when non-breaking wave and ratio of incident wave height to still water level (H/d) are considered as 0.51 and 0.10653 respectively.

Similarly in passive case these values are considered as 0.66 and 0.09726 respectively, but in reality these values can be calculated by knowing the time period of non-breaking wave and ratio of incident wave height to depth of water at structure. Effects of various parameters on both sliding and overturning stability are detailed in the following sections.

4 RESULTS AND DISCUSSIONS

The factor of safety values against sliding and overturning modes of failure can be obtained by substituting the suitable values for the different non-dimensional parameters involved in Eqs. (8), (9), (10) and (11). It should be noted that, in active case for the parametric variation, the values of non-dimensional parameters, ratio of height of mean water level above still water level to height of incident wave (\( h_H/H \)) and ratio of depth of water at structure to the wave length (\( d/H \)) are considered as 0.51 and 0.10653 respectively. Similarly in passive case these values are considered as 0.66 and 0.09726 respectively, but in reality these values can be calculated by knowing the time period of non-breaking wave and ratio of incident wave height to depth of water at structure. Effects of various parameters on both sliding and overturning stability are detailed in the following sections.

4.1 Effect of non-breaking wave height (H)

The effect of non-breaking wave height (H) on the factor of safety against sliding (FSs) and overturning (FSo) modes of failure of the wall can be interpreted from Figures 3 and 4 respectively. It is observed that the values of FSs and FSo are found to be decreasing considerably with an increase in value of H/d. For a typical value of b/h=0.2, when H/d=0 the value of FSs is 12.737; whereas the same is reduced to a value of 2.492 when H/d=0.60 i.e., there is a decrease in the value of FSo by about 80% for an increase of H/d value from 0 to 0.6. For the same data, the decrease in the value of FSs is about 67%.
4.2 Effect of submergence of backfill \((d_L)\)

The effect of submergence of backfill \((d_L)\) on the stability of wall for active case is shown in Figures 5 and 6. The values of \(F_{So}\) and \(F_{Ss}\) are decreasing significantly with the increase in value of \(d_L/h\). For example, for \(b/h=0.4\), when \(d_L/h\) changes from 0 to 1 the value of \(F_{So}\) decreases from 1.705 to 0.532. For the same data, the values of \(F_{So}\) decreases from 1.634 to 0.801. Hence, the stability of the structure will become more critical as the water level in the backfill increases and one should consider these variations appropriately in the design.

![Fig. 3 Factor of safety against sliding mode of failure for different \(H/d_S\) values in passive earth pressure condition](image1)

![Fig. 4 Factor of safety against overturning mode of failure for different \(H/d_S\) values in passive earth pressure condition](image2)

![Fig. 5 Factor of safety against sliding mode of failure for different \(d_L/h\) values in active earth pressure condition](image3)

![Fig. 6 Factor of safety against overturning mode of failure for different \(d_L/h\) values in active earth pressure condition](image4)

![Fig. 7 Factor of safety against sliding mode of failure for different \(\phi\) values in active earth pressure condition](image5)

![Fig. 8 Factor of safety against overturning mode of failure for different \(\phi\) values in active earth pressure condition](image6)

![Fig. 9 Factor of safety against sliding mode of failure for different \(\delta\) values in active earth pressure condition](image7)
4.4 Effect of wall friction angle (δ)

The typical variations of the values of \( FS_s \) and \( FS_o \) for different values of wall friction angle (δ) for active case are indicated in Figures 9 and 10 respectively. The stability of the wall found to be increasing significantly with increase in value of wall friction angle (δ). As an illustration, for \( b/h=0.3 \) in Figure 9, as the value of δ increases from 0° to 2φ/3, the value of \( FS_s \) increased from 0.937 to 1.138, i.e., the wall is failed when the wall is smooth and it becomes stable as δ increases to 2φ/3. For the same data, there is an increase in the value of \( FS_o \) by about 32% for an increase of wall friction angle (δ) from 0° to 2φ/3.

The effect of wall friction angle (δ) on the stability of wall for passive case can be depicted from Figures 11 and 12. It is observed that the values of \( FS_s \) and \( FS_o \) are increasing with an increase in value of wall friction angle (δ). For a typical value of \( b/h=0.2 \), when the value of δ increases from 0° to 2φ/3 the values of \( FS_s \) and \( FS_o \) are increased by 58% and 70% respectively.

5 CONCLUSIONS

This paper presents a design methodology for a typical seawall supporting a submerged cohesionless backfill under the action of non-breaking waves. In the present study, closed form solutions to obtain the factor of safety against sliding and overturning modes of failure have been proposed by using simple limit equilibrium approach. From the results of the present study it is observed that non-breaking wave height, depth of submergence of backfill, soil and wall friction angles had a major effect on the stability of seawalls. When the value of \( H/d_S \) increases from 0 to 0.60, there is a decrease in the value of factor of safety against overturning by about 80% in passive case. It is observed that as the soil friction angle (φ) increased from 25° to 35°, the value of factor of safety against sliding increased by about 63% in active case. It is also observed that, when the value of δ increases from 0° to 2φ/3 the values of factor of safety against sliding and overturning modes of failure are increased by about 58% and 70% respectively, in passive case. The expressions proposed, which are in terms of non-dimensional parameters and the design charts presented in this study can be readily used by engineers for assessing the stability of seawalls under the action and non-breaking wave.

Notations

- \( b, h \): width and height of the wall
- \( H \): height of non-breaking wave
- \( d_S, d_L \): height of water on seaward and landward sides of the seawall
- \( FS_s \): factor of safety against sliding mode of failure
- \( FS_o \): factor of safety against overturning mode of failure
- \( K_a \): active earth pressure coefficient
\[ K_p \quad \text{passive earth pressure coefficient} \]
\[ C_a \quad \text{a constant } = 0.5 K_a \gamma \]
\[ C_p \quad \text{a constant } = 0.5 K_p \gamma \]
\[ P_a \quad \text{active earth thrust} \]
\[ P_p \quad \text{passive resistance} \]
\[ P_w \quad \text{non-breaking wave pressure including hydrostatic water pressure on seaward side} \]
\[ P_{stl} \quad \text{hydrostatic pressure on landward side} \]
\[ \psi_s, \psi_L \quad \text{pore water pressure at the seaward and landward sides of the seawall} \]
\[ U_b \quad \text{uplift pressure at the base of the seawall} \]
\[ W \quad \text{weight of the wall} \]
\[ y_a \quad \text{point of application of } P_a \]
\[ y_p \quad \text{point of application of } P_p \]
\[ x \quad \text{point of application of uplift pressure} \]
\[ h_o \quad \text{height of mean water level above the still water level at the wall} \]
\[ \chi \quad \text{wave reflection coefficient} = 1(\text{assuming complete reflection of incident wave}) \]
\[ \gamma_t \quad \text{depth of wave trough} \]
\[ \gamma_c \quad \text{depth of wave crest} \]
\[ \delta, \phi \quad \text{wall and soil friction angles} \]
\[ \delta_b \quad \text{friction angle at the base of the seawall} \]
\[ \gamma_w, \gamma_c \quad \text{unit weight of water and concrete} \]
\[ \gamma_d, \gamma_{sat} \quad \text{dry and saturated unit weight of soil} \]
\[ \gamma \quad \text{equivalent unit weight of soil due to submergence} \]
\[ \mu \quad \text{coefficient of base friction} \]

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