REVIEW OF PSEUDO-DYNAMIC DESIGN APPROACH FOR WATERFRONT RETAINING STRUCTURES SUBJECTED TO EARTHQUAKE AND TSUNAMI

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ABSTRACT: Latest design techniques for waterfront retaining structures or sea walls subjected to combined effects of earthquake and tsunami have been reviewed. Both sliding and overturning modes of failures have been studied to obtain the closed-form design equations for factor of safety using modern pseudo-dynamic approach which considers body waves of earthquake motion, frequency of shaking, duration of earthquake, soil amplification along with horizontal and vertical seismic acceleration profiles with depth and time. These additional considerations of earthquake forces made the pseudo-dynamic approach more reliable and acceptable compared to conventional pseudo-static approach after experimental validation. Stability of sea walls found to decrease significantly with increase in both the earthquake accelerations and tsunami height. Design factors for sliding mode of failure to obtain the safe weight of sea walls under combined effect of earthquake and tsunami have been proposed. Pseudo-dynamic results are found to provide economic design factors than pseudo-static approach.

1. INTRODUCTION

Waterfront retaining structures or seawalls like gravity type cellular cofferdams, gravity dams, various types of earth retaining walls, reinforced soil structures etc. are used in the earthquake prone areas around the world. These are the key elements in ports and harbours, transportation systems, lifelines and other geotechnical structures. Seismic vulnerability and the associated detrimental economic implications have been highlighted earlier in a number of damage reconnaissance reports for waterfront structures. Under normal service conditions (i.e. when there is no earthquake) waterfront retaining structures are exposed to wave forces. However, the situation changes when such a waterfront retaining wall which retains a submerged backfill on one side (downstream side) is subjected to an earthquake. In that situation, additional hydrodynamic pressure gets generated with the seismic lateral earth pressure on the downstream side of the wall.

One of the most important reasons of failure of these waterfront retaining structures is the hydrodynamic pressure, the effect of which according to the classical Westergaard (1933) approach is neither excessively large nor negligible. Although, in the past, few analytical studies were carried out to examine the effect of hydrodynamic pressure, it still seems that due to the complex overall seismic behaviour of waterfront retaining structures—involving interaction between the structure, the retained soil and the water—a better understanding is needed. Currently the design approach proposed by Ebeling & Morrison (1992) for waterfront retaining structures using pseudo-static approach is widely used however it considers the uplift of the wall also leading to a non-critical condition.

Similarly, tsunamis triggered by an earthquake (or otherwise too) cause severe damage to the waterfront retaining structures. The combination of earthquake and tsunami forces on a waterfront retaining wall severely challenges its stability, both in terms of sliding and overturning modes of failure. Due to complexity of the study, this important topic was not thoroughly researched earlier and was generally confined to the consideration of the above mentioned forces individually but not simultaneously.

2. ANALYSES OF WATERFRONT RETAINING WALL

A rigid vertical waterfront retaining wall with width ‘b’ and height ‘H’, which is subjected to different forces during an earthquake, is shown in Figure 1a. It retains backfill to its full height on one side, referred to as the ‘downstream side’, and holds water to height of ‘h_{wd}’ on the other side, called as the ‘upstream side’. The water table on the downstream side of the wall is to a height ‘h_{wu}’. Depending on the consideration of the forces due to tsunami, two separate cases are chosen viz., (1) Case 1: without tsunami forces and active state of earth pressure based on the direction of wall movement (Fig. 1a) and (2) Case 2: with tsunami forces and passive state of earth pressure based on the direction of wall movement (Fig. 1b). Case 1 details the stability of the waterfront retaining wall under the combined action of seismic and hydrodynamic forces, while in Case 2 more complex nature of forces is taken into account by considering tsunami force as well.

2.1 Computation of Different Forces Acting on the Waterfront Retaining Wall

A rigid vertical cantilever retaining wall AB of height H, supporting cohesionless backfill is considered as shown in
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Figure 2(a) and Figure 2(b). Similar to the pseudo-dynamic approach proposed by Steedman & Zeng (1990) with horizontal seismic acceleration and shear wave velocity only, Choudhury & Nimbalkar (2005, 2006) had considered both finite shear wave velocity ($V_s$) and primary wave velocity ($V_p$) in the modified and latest improved pseudo-dynamic analysis. The phase and the magnitude of both the horizontal and vertical seismic accelerations are varying along the depth of the wall. In the present analysis, by using the relationship between the primary and shear wave velocities with poisson’s ratio ($\nu$) of the material, $V_p/V_s = 1.87$ for $\nu = 0.3$ is used (Das 1993). Period of lateral shaking, $T = 2\pi/\omega = 4H/V_s$ is considered in the analysis.

The base of the wall is subjected to harmonic horizontal seismic acceleration of amplitude $k_{hg}$, where $g$ is the acceleration due to gravity and harmonic vertical seismic acceleration of amplitude $k_v$. The exact nature of soil amplification is dependent on many factors, including the geometry and rigidity of adjacent structures, the stiffness and damping in the soil, the depth of the soil layer and so on. Again a simplified assumption is made that the horizontal and vertical acceleration vary linearly from the input acceleration at the base to the higher value (depending upon the soil amplification) at the top of the retaining wall, such that $k_{h(z=0)} = f k_{h(z=H)}$ and $k_{v(z=0)} = f k_{v(z=H)}$, where $f$ is a constant and is termed as amplification factor (Nimbalkar & Choudhury 2008). For no amplification, $f = 1.0$. For obtaining the critical design value of seismic earth pressure, in the present analysis, it is assumed that both the horizontal and vertical vibrations with amplitude of accelerations $k_{hg}$ and $k_{vg}$ respectively, start at exactly the same time and there is no phase shift between these two vibrations. Referring to Figure 2(a) and Figure 2(b), the acceleration at any depth $z$ and time $t$, below the top of the wall can be expressed as,

$$a_h(z,t) = \left\{ 1 + \frac{H-z}{H} \right\} \left( f - 1 \right) k_{h} g \sin \left( \omega t \right) \left( \frac{H - z}{V_s} \right)$$ (1)

$$a_v(z,t) = \left\{ 1 + \frac{H-z}{H} \right\} \left( f - 1 \right) k_{v} g \sin \left( \omega t \right) \left( \frac{H - z}{V_p} \right)$$ (2)

Fig. 1a: Forces on Typical Waterfront Retaining Wall Under Seismic Conditions for Active Case [Case 1] (Choudhury & Ahmad 2008)

Fig. 1b: Forces on Typical Waterfront Retaining Wall Under Seismic Conditions for Passive Case [Case 2] (Ahmad & Choudhury 2008)

Now, the total seismic active earth pressure ($P_{ae}$) (in Fig. 1a) calculated by using the pseudo-dynamic approach is given by,

$$P_{ae} = \frac{1}{2} K_{ae} H^2 \gamma (1 - \nu_a)$$ (3)

Fig. 2a: Model Rigid Retaining Wall for Computation of Pseudo-Dynamic Active Earth Pressure (Choudhury & Nimbalkar 2006)

Fig. 2b: Model Rigid Retaining Wall for Computation of Pseudo-Dynamic Passive Earth Resistance (Choudhury & Nimbalkar 2005)
where \( \gamma = \) modified unit weight of the backfill soil and it is actually the weighted average of the total unit weight of the backfill soil below and above the water table. \( \gamma \) is calculated using the approach suggested by Kramer (1996) as,

\[
\gamma = \left( \frac{h_{dl}}{H} \right)^2 \gamma_{sat} + \left( 1 - \left( \frac{h_{dl}}{H} \right)^2 \right) \gamma_s
\]

(4)
in which \( \gamma_s = \) dry unit weight of the backfill soil, \( \gamma_{sat} = \) saturated unit weight of the backfill soil, \( r_u = \) pore pressure ratio, \( K_{ae} = \) seismic active earth pressure coefficient calculated using the pseudo-dynamic approach (Choudhury & Nimbalkar 2006) and given by,

\[
K_{ae} = \frac{1}{\tan \alpha_{ae}} \cos \left( \delta + \phi - \alpha_{ae} \right) + \frac{k_h}{2\pi^2 \tan \alpha_{ae}} \frac{\left( \frac{TV_s}{H} \right)}{\cos \left( \delta + \phi - \alpha_{ae} \right)} m_1 + \frac{k_v}{2\pi^2 \tan \alpha_{ae}} \frac{\left( \frac{TV_p}{H} \right)}{\cos \left( \delta + \phi - \alpha_{ae} \right)} m_2
\]

(5)

with \( \phi = \) soil and wall friction angles, \( \alpha_{ae} = \) wedge angle with the horizontal for the active case, then \( m_1 \) and \( m_2 \) appearing in Eq. (5), are given by,

\[
m_1 = 2 \pi \cos 2 \pi \left( \frac{t}{T} \right) \frac{H}{TV_s} + \left( \frac{TV_s}{H} \right) \sin 2\pi \left( \frac{t}{T} \right) - \sin 2\pi \left( \frac{t}{T} \right)
\]

(6a)

\[
m_2 = 2 \pi \cos 2 \pi \left( \frac{t}{T} \right) \frac{H}{TV_p} + \left( \frac{TV_p}{H} \right) \sin 2\pi \left( \frac{t}{T} \right) - \sin 2\pi \left( \frac{t}{T} \right)
\]

(6b)

From the pseudo-dynamic approach, the seismic wall inertia forces in the horizontal and vertical directions, shown by \( Q_{hw} \) and \( Q_{vw} \) in both Figures 1a and 1b, are

\[
Q_{hw} = \frac{TV_{sw} \gamma_c h_{bw}}{2\pi g} \left[ -\cos 2 \pi \frac{t}{T} + \cos 2 \pi \frac{t}{T} \frac{H}{TV_{sw}} \right]
\]

(7a)

and

\[
Q_{vw} = \frac{TV_{pw} \gamma_c h_{bw}}{2\pi g} \left[ -\cos 2 \pi \frac{t}{T} + \cos 2 \pi \frac{t}{T} \frac{H}{TV_{pw}} \right]
\]

(7b)

where \( \gamma_c = \) unit weight of the wall, \( g = \) acceleration due to gravity, \( h_{bw} = \) amplitude of horizontal and vertical seismic accelerations of the wall, \( V_{sw} \) and \( V_{pw} = \) velocity of the shear and primary waves propagating through the wall material. For pseudo-static approach, the seismic wall inertia forces in the horizontal and vertical directions (i.e., \( Q_{hw} \) and \( Q_{vw} \)) will be \( k_h W_w \) and \( k_v W_w \), respectively; where \( W_w = \) weight of the wall.

The pseudo-dynamic approach for handling the geotechnical earthquake engineering problems by considering body waves, time period (or frequency of earthquake excitation), duration of earthquake motion, soil amplification along with other seismic factors as considered in pseudo-static approach is found to be more closer to the experimental results (Nimbalkar & Choudhury 2008) which validates the use of this approach. Because of the practical uses and robustness of this pseudo-dynamic approach over the conventional pseudo-static approach, other than the research group of present author, various researchers are currently using it for solving different geotechnical earthquake engineering problems in recent days (for example see, Ghosh 2007; Nouri et al. 2008; Kolathayar & Ghosh 2009; Reddy et al. 2009; Basha & Babu 2009).

The hydrodynamic force on the downstream and upstream sides of the waterfront retaining wall, shown by \( P_{dynd} \) and \( P_{dyne} \), in Figures 1a and 1b, is (Westergaard 1933),

\[
P_{dynd} \cdot P_{dyne} = \frac{7}{12} k_h \gamma_w (h_{wd} \cdot h_{wu})^2
\]

(8)

Hydrostatic force \( (P_{sw}) \) from the upstream side is \( \frac{1}{2} \gamma_w (h_{wu})^2 \); while the hydrostatic force \( (P_{std}) \) from the downstream side is \( \frac{1}{2} \gamma_w (h_{wd})^2 \), where \( \gamma_w = \) unit weight of water, \( \gamma_w = \) modified unit weight of water (Choudhury & Ahmad 2008). The force due to the tsunami, shown by \( P_t \), in Figure 1b, is computed by using both CRATER (2006) and Fukui et al. (1962) approaches as given by Eqs. (9) and (10) respectively.

\[
P_{tc} = 4.5 \gamma_w (h_{t})^2
\]

(9)

and

\[
P_{tf} = \frac{1}{2} \gamma_w (h_{wu})^2 \left[ \left( \frac{h_l}{h_{wu}} \right)^2 + 2 \left( \frac{h_l}{h_{wu}} \right) + K \right]
\]

(10)

where \( P_{tc} = \) tsunami force calculated using CRATER (2006) approach, \( P_{tf} = \) tsunami force calculated using Fukui et al. (1962) approach, \( h_l = \) height of the tsunami, \( K = \) a constant = 0 when \( h_l/h_{wu} = 0 \) (i.e., no tsunami); and \( K = 0.12 \) for any other value of \( h_l/h_{wu} \).

2.2 Factor of Safety for Case 1 and Case 2

Using limit equilibrium approach, the factor of safety against the sliding mode of failure for the restrained and free water cases (i.e., \( FS_Sl \) and \( FS_Fs \)) by considering hydraulic conductivity of backfill soil for Case 1 and adopting the pseudo-dynamic approach is given by (Choudhury & Ahmad 2008).

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the methodology proposed by Choudhury & Nimbalkar (2006) to consider the effect of wet backfill in the present study. For the overturning mode of failure of the wall, the point of application \((H_d)\) for the seismic active thrust by using pseudo-dynamic method as proposed by Choudhury & Nimbalkar (2007) is used and given by,

\[
H_d = \frac{1}{2\pi^2 H^2} \left( m \cos \omega_x - n \cos \omega_y + 2 \pi \dot{h} \sin \omega_x \sin \omega_y \right)
\]

where \(m = \lambda \dot{h}, \cos(\alpha - \phi); \ n = \eta \dot{h}, \sin(\alpha - \phi); \ \omega = 2\pi/T; \ \zeta = 1 - \frac{H}{V_p}; \ \psi = t - H/V_p; \ \lambda = TV_p; \ and \ \eta = TV_p.

For Case 2, in which the tsunami force is also considered, the factor of safety against the sliding mode of failure by the pseudo-dynamic approach is given by (Ahmad & Choudhury 2008) Eqs. (16) and (17) by using CRATER (2006) and Fukui et al. (1962) approach respectively. Similar expressions for the factor of safety against the overturning mode of failure are developed as per the details given in Ahmad & Choudhury (2008) as reported by Eqs. (18) and (19).

\[
\left(\frac{h_{\text{wad}}}{H}\right) = \frac{1}{2} m \left[ \frac{b}{H} \gamma_c - \frac{TV_{\text{w}}}{H} \frac{b}{H} k_m m_4 \gamma_c - C_4 \sin \delta \right] + \frac{1}{2} \gamma_{\text{w}} \left( \frac{h_{\text{wad}}}{H}\right)^2 + C_4 \cos \delta
\]

(16)

\[
\left(\frac{h_{\text{a}}}{H}\right) = \frac{1}{2} \gamma_{\text{w}} \left( \frac{h_{\text{a}}}{H}\right)^2 + C_4 \cos \delta
\]

(17)
to decrease drastically with an increase in the height of water inside the backfill.

\[
FS_{as} = \frac{1}{2} \left( \frac{b}{H} \right)^2 \gamma_c + \frac{TV}{2} \left( \frac{h}{H} \right)^2 \gamma_w k_h m_4 + \frac{1}{6} \gamma_w \left( \frac{h}{H} \right)^3
\]

(19)

2.3 Seismic Sliding Design Factors

Combined dynamic design factor \( F_{Wwet} \) by using the pseudo-static approach is given by (Choudhury & Ahmad 2009)

\[
F_{Wwet} = \frac{W_w}{W_{stat}} = F_{Twet} F_{iwet} \frac{P_{1iwet}}{P_{iwet} C_1}
\]

(20)

where \( W_{stat} \) = weight of the wall under static condition, which is required for providing sliding stability, \( F_{iwet} = \) dynamic thrust factor for wet condition \( = \frac{K_{as} \gamma (1-k_v)(1-r_u)}{K_{as} \gamma_d} \), \( F_{iwet} = \)

inertia factor for the wet condition \( = \frac{\tan \phi}{\tan (1-k_v)-k_h} \),

\[
P_{iwet} = P_{std} + P_{dyna} - P_{stu} + P_{dydu} \frac{TV}{H} \frac{k_h m_3 + TV \frac{k_h m_4}{H}}{\gamma_w \left( \frac{h}{H} \right)^3}
\]

(21)

where \( F_{iwet} = \frac{K_{as} \gamma (1-r_u)}{K_{as} \gamma_d} \tan \phi \),

\[
P_{iwet} = \frac{1}{2\pi \tan \phi} \left[ TV \frac{TV}{H} k_h m_3 + TV \frac{k_h m_4}{H} \right].
\]

By using the pseudo-dynamic approach, the dynamic thrust factor is given by (Ahmad & Choudhury 2009)

3. RESULTS AND DISCUSSIONS

Typical results for the stability of the waterfront retaining wall against the sliding mode of failure by using the pseudo-dynamic approach for Case 1 is presented in Figure 3. The effect of changing \( h_{w/d}/H \) and \( h_{w/u}/H \) values on the factor of safety for lower \( k_h \) values is significant; while it is marginal at higher \( k_h \) values. For \( B/H = 0.4, \phi = 30^\circ, \delta = \phi/2, k_h = 0.1, k_v = k_h/2, r_h = 0.2, T = 0.3, s, V = 100 m/s, V_p = 187 m/s, V_{sw} = 2500 m/s, V_{wp} = 3900 m/s, \) the factor of safety against sliding mode of failure for the restrained and free water cases (i.e., \( FS_a \) and \( FS_{as} \)) decreases from 2.10 to 1.17 and from 2.08 to 1.10, respectively when the \( h_{w/d}/H = h_{w/u}/H \) is increased from 0.25 to 1.00 (Fig. 3). It shows that the factor of safety tends

To use the pseudo-dynamic approach for Case 2. As the tsunami height is increased, the factor of safety gets reduced drastically. Figure 5 shows the variation of the sliding design factors
The combined effects of the hydrodynamic pressure, coming from the upstream and downstream faces and the seismic forces including the effects of wall inertia on the sliding and overturning stability of the waterfront retaining structure have been reviewed to give closed-form design solution. It is observed that the most important destabilizing forces, which add to the instability of the waterfront retaining structure are hydrodynamic force and tsunami.

- As the ratio of the water height on the downstream face of the wall to the total height of the wall is increased from 0.25 to 1.00, then for values of the horizontal seismic acceleration coefficient above 0.2, the wall section fails for both the sliding and overturning modes.
- About 57% decrease in the factor of safety in sliding mode of failure is observed when the horizontal seismic acceleration coefficient is increased from 0.0 to 0.2; this is because of the critical combination of different forces like the seismic force, hydrostatic and the hydrodynamic forces cause serious instability to the waterfront structure.
- As backfill soil condition is changed from fully dry to fully submerged, the weight of the wall needed for sliding stability is around 3 times more for the later case than that for the former case under seismic conditions.
- The results obtained from the pseudo-static analysis are more conservative than those obtained by the pseudo-dynamic approach, because of the conservative manner of the seismic forces in the pseudo-static approach.

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