COUPLED SLIDING AND TILTING OF GRAVITY RETAINING WALLS DURING EARTHQUAKES

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SUMMARY

This paper presents a model for evaluating the earthquake-induced permanent rotation and translation of a gravty retaining wall with dry and cohesionless backfill. The governing equations are derived from equilibrium and continuity conditions. Recommendations are made to esimate the amount of wall movement for a design earthquake that is characterized by its peak acceleration and peak velocity.

INTRODUCTION

Since 1920's, the common practice in seismic design of gravity retaining walls has been the use of the Mononobe (Ref. 1) and Okabe (Ref. 2) analysis, with expected peak ground acceleration. The Mononobe-Okabe analysis of dynamic earth pressures is an extension of Coulomb's theory of static earth pressures, where the inertial forces of backfill soil are taken into account. Conventional methods for the seismic design of gravity retaining walls and the Mononobe-Okabe analysis are discussed extensively by Seed and Whitman (Ref. 3). The inertial forces of the wall itself are usually neglected in the conventional design methods. However, Richards and Elms (Ref. 4) show that the inertial forces may be as large as the dynamic earth pressures during an earthquake. If the wall inertia is included in the analysis, the conventional design methods lead to uneconomical, and sometimes impossible, designs.

Richards and Elms (Ref. 4) suggest a new design procedure for gravity retaining walls with dry and cohesionless backfill, where the wall is designed for an acceleration less than the expected peak ground acceleration. In their procedure, the design criteria is to keep the earthquake-induced relative displacement between the wall and its supporting soil less than an allowable amount. Richards and Elms use Newmark's (Ref. 5) block-on-plane model to compute the earthquake-induced permanent wall displacements (Fig. 1). Some of the conceptual shortcomings of the Richards-Elms model were later eliminated by Zarrabi (Ref. 6). Displacement time histories predicted by Zarrabi's model compare very well with the experimental results obtained by Lai (Ref. 7) in a series of shaking table tests on model retaining walls.

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Richards-Elms type models provide a systematic and rational basis for the seismic design of gravity retaining walls, where both the inertial effects and the dynamic earth pressures are accounted for.

In the Richards-Elms model and Zarrabi's model, it is assumed that the wall fails by sliding, and the tilting mode of failure is ignored. However, most gravity retaining walls fail by overturning during strong earthquakes. In this paper, the Richards-Elms' concept is extended to study the tilting, and the coupled tilting-sliding modes of failure of gravity retaining walls during earthquakes.

PROPOSED MODEL

The proposed model allows one to compute the amount of permanent tilting and sliding of a gravity retaining wall due to earthquake loading. During earthquakes, when the horizontal ground acceleration is away from the backfill, the inertial forces act towards the backfill and create a dynamic passive condition. In this situation, the horizontal ground acceleration required to cause failure in the backfill is, typically, an order of magnitude greater than the horizontal ground acceleration required to create a dynamic active condition. Thus, generally, permanent wall distortions (sliding and/or tilting) initiate when the ground acceleration is towards the backfill and a dynamic active condition is created. For this reason, the dynamic active condition is studied in this section and the equations that are derived are valid only for this condition.

The retaining wall problem is highly indeterminate. In order to make the problem determinate, all elastic deformations are neglected and the following simplifying assumptions are made: (1)The wall is long enough for end effects to be neglected (i.e., plane-strain conditions exist). (2)The backfill is dry and uniform sand with constant properties. (3)The wall-backfill interface and the interface between the wall and the foundation soil are frictional. (4)The foundation soil has a constant moment capacity below which no rotational movements take place. Once the moment capacity is reached, the foundation soil deforms plastically in rotation. (5)The wall is rigid and the center of rocking is at a fixed point at the base of the wall. (6)When active conditions exist, a failure zone, which consists of infinitely many parallel planes, develops in the backfill. This assumption allows one to satisfy continuity when the wall is tilting (Fig. 2).

For simplicity, a vertical wall-backfill interface is assumed in this paper.

The total dynamic force on the wall and its line of action are evaluated by considering the equilibrium of a slice of backfill (Fig. 2) and integrating the force increment along the wall-backfill interface. Continuity requires that the acceleration normal to each failure plane be the same on both sides of the plane. For known ground acceleration and wall accelerations (translational and rotational), the continuity requirement establishes the magnitudes of the horizontal and the vertical accelerations of each backfill slice.

When the horizontal acceleration of the ground is towards the backfill, at some limiting acceleration the wall begins to slide, and at some other limiting acceleration the wall begins to tilt plastically. These limiting accelerations and the wall accelerations during sliding and/or plastic tilting are evaluated by considering the dynamic equilibrium of the wall (Fig. 3). Once slippage or plastic tilting starts, it will continue until the relative velocity (translational in case of sliding and rotational in case of tilting) between the wall and the ground becomes zero.

The equilibrium and continuity conditions of the wall and the backfill lead to Eqs. (1) through (7), which are used in computing the permanent wall movements.

The dynamic active force on the wall and its height of action are:

$$P_{AE} = \frac{1}{2}\gamma H^2 c \left(b - \frac{a\ddot{\theta}}{3}\right) \tag{1}$$

$$L = \frac{H}{3}\left[\frac{b - a\,\ddot{\theta}/2}{b - a\,\ddot{\theta}/3}\right] \tag{2}$$

where $a = \dfrac{H \cos\phi}{g \cos\alpha}$

$$b = \sin(\alpha - \phi)(1 - k_{vg} - k_{hg}\tan\alpha) + k_{hw}\frac{\cos\phi}{\cos\alpha}$$

$$c = \frac{\cos i \ \cos\alpha}{\sin(\alpha - i)\ \cos(\alpha - \delta - \phi)}$$

H = height of wall

g = gravitational acceleration

i = backfill slope

α = inclination of failure planes in backfill

ϕ = friction angle of backfill

δ = friction angle of wall-backfill interface

γ = unit weight of backfill

$\ddot{\theta}$ = rotational acceleration of wall

Parameter "k" is a dimensionless acceleration coefficient, with the first subscript "h" or "v" meaning "horizontal" or "vertical", and the second subscript "g" or "w" meaning "ground" or "wall".

The inclination of failure planes in the backfill is evaluated from:

$$\alpha = \phi - \psi + \tan^{-1}\left\{\frac{\tan\xi + \sqrt{\tan\xi\,[\tan\xi + \cot(\phi-\psi)][1+\tan(\delta+\psi)\cot(\phi-\psi)]}}{1+\tan(\delta+\psi)[\tan\xi + \cot(\phi-\psi)]}\right\} \tag{3}$$

where $\xi = \phi - i - \psi$

$$\psi = \tan^{-1}\frac{k_{hw} - \ddot{\theta}H/3g}{1 - k_{vg} + (k_{hw} - k_{hg} - \ddot{\theta}H/3g)\tan\alpha} \tag{4}$$

In absence of vertical ground acceleration, the horizontal ground acceleration coefficient that initiates plastic tilting is:

$$N_{tilt} = \frac{M_r + P_{AE}\,(B_2\sin\delta - H\cos\delta/3) + W_w B_1}{W_w B_3} \tag{5}$$

where M_r = ultimate moment capacity of foundation soil

W_w = weight of wall

B_1, B_2, B_3 = dimensions defined in Fig.3

and the horizontal ground acceleration coefficient that initiates sliding is:

$$N_{slid} = \tan\phi_b\,(1 + \frac{\ddot{\theta}B_1}{g}) + \frac{\ddot{\theta}B_3}{g} - \frac{P_{AE}\cos(\delta+\phi_b)}{W_w\cos(\phi_b)} \tag{6}$$

where ϕ_b = friction angle between soil and base of wall

When the vertical ground acceleration is not zero, the values of N_{tilt} and N_{slid} from Eqs. (5) and (6) must be multiplied by $(1-k_{vg})$.

When plastic tilting is occuring, the rotational acceleration of the wall is computed from:

$$\ddot{\theta} = \frac{P_{AE}\,(L\cos\delta - B_2\sin\delta) + W_w\,[k_{hw}L + (1-k_{vg})\,B_1] - M_r}{I_{CG} + W_w\,(B_1^2 + B_3^2)\,/\,g} \tag{7}$$

where I_{CG} = moment of inertia of wall with respect to its center of gravity

Permanent wall rotation is computed by integrating Eq. (7) twice with respect to time.

Note that all the equations mentioned in this section are coupled and must be solved by iteration. Detailed derivation of these equations and an efficient iterative scheme for their solution are given in Ref. 8.

DISCUSSION OF RESULTS AND RECOMMENDATIONS

In all the available literature on seismic earth pressures, it is suggested that the dynamic force on the wall acts at, or above, the lower third point of the wall. However, Eq. (2) shows that, for a tilting wall, this is true only when the rotational acceleration of the wall is negative (i.e., when the wall is decelerating). When the wall starts to tilt, its rotational acceleration is positive and the line of action of the force on the wall drops below the lower third point.

The dynamic pressure distribution is important in conventional strength-criterion design methods. The important factor in the seismic performance of the wall, however, is the magnitude of permanent wall movement that is induced by the earthquake. Studies performed by Nadim (Ref. 8) show that when sliding starts before tilting (i.e., $N_{slid} < N_{tilt}$), only sliding movements take place, but when tilting starts before sliding, usually the wall movement is coupled tilting and sliding, with tilting movements dominating the displacement pattern.

For design purposes, the following predictive equation suggested by Wong (Ref. 9) can be used to evaluate the earthquake-induced wall displacement:

$$D = \frac{37\ V^2}{Ag} \cdot e^{-9.4\ (N/A)} \tag{8}$$

where Ag = peak earthquake acceleration

V = peak earthquake velocity

N = minimum of N_{slid} and N_{tilt}

When $N_{slid} \leq N_{tilt}$, Eq. (8) predicts the translational wall movement, and when $N_{tilt} < N_{slid}$, Eq. (8) predicts the displacement of the upper third point of the wall.

The most uncertain parameters in these calculations are the moment capacity of the foundation soil M_r, and the location of the center of rocking. As a first estimate, one might assume that $M_r = 0$, and that the wall tilts about its toe. However, Nadim (Ref. 8) shows that this is not necessarily a conservative assumption. When the center of rocking is inside the toe of the wall, the foundation soil has some (non-zero) moment capacity. However, the wall has a smaller moment of inertia about such a point (e.g., point O in Fig. 3) than about the toe of the wall. Therefore, the location of the critical center of rocking depends on the variation of the ultimate moment capacity and the moment of inertia of the wall with distance at the base of the wall. Due to these uncertainties, it is advisable to use a high factor of safety on allowable displacements when the tilting mechanism of failure is likely to dominate the wall behavior.

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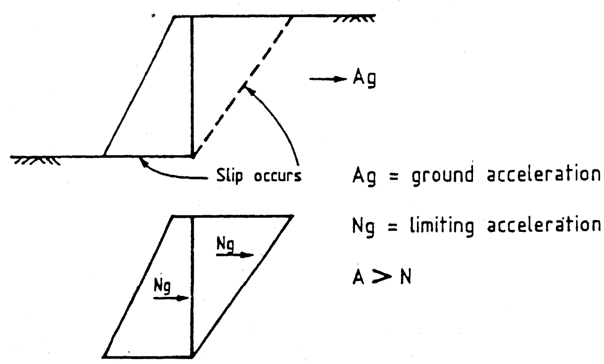

Fig.1 Block-on-Plane Analogy for Gravity Retaining Walls (Richards-Elms Model).

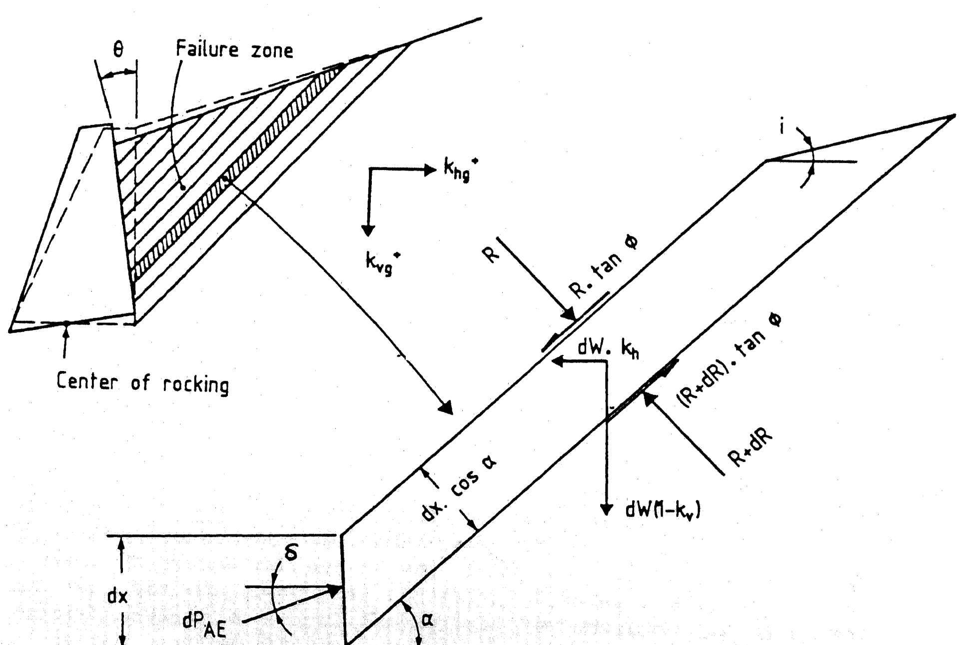

Fig.2 Failure Zone Behind a Tilting Wall and Dynamic Forces Acting on a Slice of Backfill.

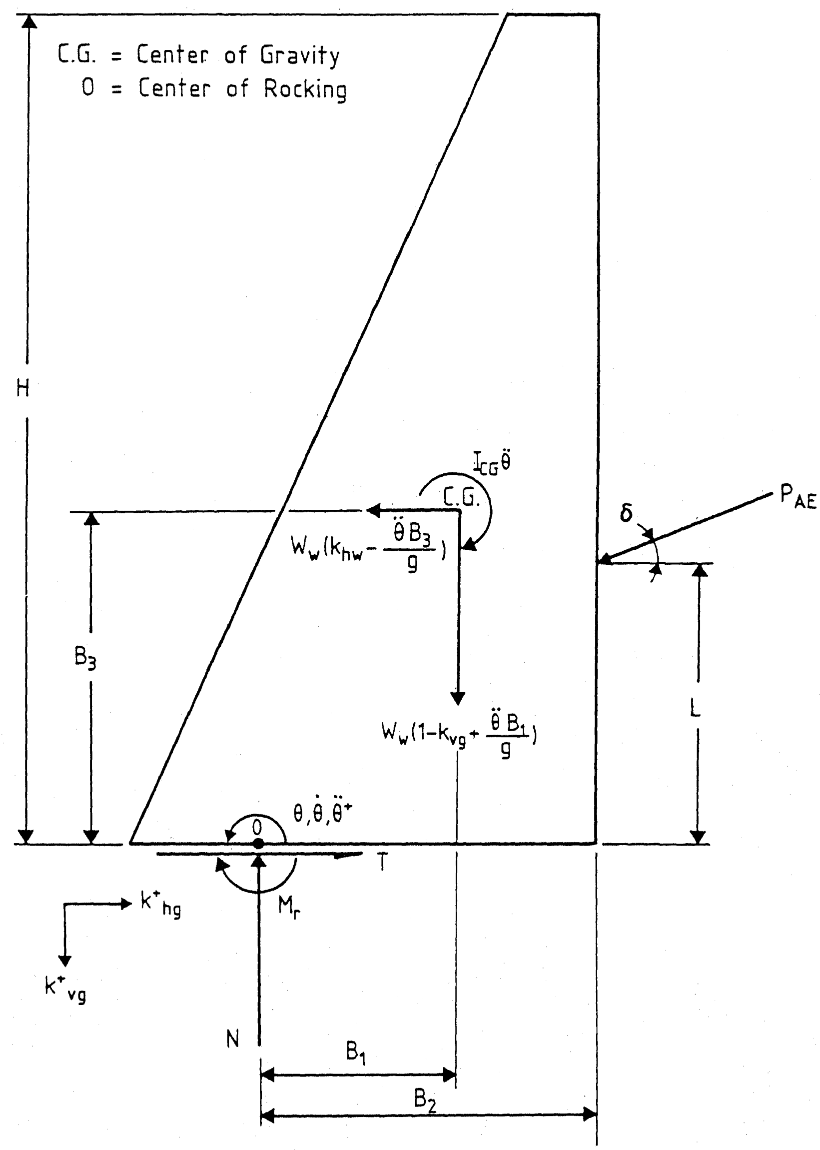

Fig. 3 Dynamic Forces Acting on Wall.

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