STUDY OF SEISMICALLY INDUCED PERMANENT DISPLACEMENT OF GRAVITY RETAINING WALL

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ABSTRACT: Gravity walls are widely used as earth retaining systems. The permanent displacement of retaining structures was seen in many historical earthquakes. Newmark sliding block approach has been extensively used in dynamic displacement analyses to determine the permanent displacement of retaining structures. In this paper, a gravity retaining wall is modelled using FLAC 3D, to investigate the dynamically induced displacement. To assess validity of the proposed FLAC 3D model, results obtained from the numerical model were compared with results obtained according to Richard and Elms (1979) analytical model. The developed model is used to investigate the effects of ground acceleration, frequency and properties of backfill soil on permanent displacement of the retaining wall. The displacement of the model increase with acceleration and acceleration decreases with increase in damping of the material.

1. INTRODUCTION

Retaining structures are susceptible to failure during strong earthquake and are damaged frequently. Such failures are documented in almost all post-earthquake damage reports, e.g., the 1960 Chilean earthquake damage reported on by Duke & Leeds (1963), the 1964 Alaska earthquake reported on by Ross et al. (1969), the 1971 San Fernando earthquake reported on by Clough & Fraga (1977) and the 2004 Chuetsu earthquake reported on by Trandafir et al. (2009).

Gravity retaining walls that supported dry cohesionless backfills form a major group of earth retaining structures. These walls are damaged during strong earthquakes because of seismically induced lateral earth pressures and inertial effect of the wall itself. Two design methods are available for the seismic design of gravity retaining walls. The first method, which has been the common practice for many years, is based on design rules suggested by Mononobe & Okabe (Mononobe & Mutsumi, 1929; Okabe, 1924). The Mononobe and Okabe analysis is an extension of the Coulomb sliding wedge theory in which horizontal and vertical inertia terms take into account the earthquake loading. Richard et al. (1999), Choudhury & Singh (2005) and few others considered pseudo-static method to compute seismic active earth pressure behind the retaining wall. Steedman & Zeng (1990) proposed pseudo-dynamic method in which the time and phase difference due to shear wave propagation behind wall was taken into consideration. Zeng & Steedman (1993) compared the theoretical results with centrifuge model test. Choudhury & Nimbalkar (2006) used pseudo-dynamic method to compute the distribution of seismic active earth pressure on a rigid retaining wall.

Alternative method is the performance based design criteria in which the evaluation of the deformations and displacements, suffered by earth retaining structures under seismic condition, are taken into consideration. Nandkumaran (1973) developed mathematical model to determine the displacement of the wall. Based on Newmark (1965) sliding block theory an alternative design method for the earthquake-induced-displacement of the wall was proposed by Richard and Elms (1979). Zarrabi (1979) considered the equilibrium of the wall and failed soil wedge behind the wall. He developed an iterative procedure for computing the instantaneous values of dynamic wall pressure and corresponding failure plane inclination angle over the duration of input motion. Nadim & Whitman (1983) have proposed a two dimensional finite element model which is capable to assess displacement of the gravity retaining wall. Reddy et al. (1985) developed mathematical model to obtain translation and rotation of the wall simultaneously. Simonelli et al. (2000) conducted shake table test on small retaining wall with an aim to compare the results of experimental studies with analytical model. The results showed that the analytical methods are reliable but still slightly conservative. Watanabe et al. (2003) conducted shake table tests to study the behaviour of the retaining wall under seismic loading.

In this paper a series of retaining wall models were simulated using numerical techniques and the results are compared with analytical model.
2. ANALYTICAL APPROACH

2.1 Richard and Elms (1979) model (R-E model)

In performance based design methods displacement of the wall and the ground were computed by the numerical integration of the excitation time histories. In the R-E model, the wall and rigid portion of the backfill are modelled as the block, and the ground as the plane (Fig. 1). The wall-backfill system has a limiting horizontal acceleration \( N_g \) towards the backfill, where \( g \) is the gravitational acceleration and \( N \) is an acceleration coefficient. When the ground equals the limiting acceleration, the combined effect of the increased lateral earth pressure and the wall inertia completely mobilize the available shear strength at the base of the wall. If the ground acceleration exceeds \( N_g \), the wall begins to slip relative to its supporting ground. The amount of slippage depends on the ratio of limiting acceleration \( N_g \) to the peak ground acceleration \( A_g \). The displacement of the model can be calculated from empirical relation obtained by Franklin & Chang (1977):

\[
d = \frac{0.087V^2\left(\frac{N}{A}\right)}{A_g} \tag{1}
\]

Where \( V \) = maximum ground velocity; \( A \) = maximum ground acceleration per gravitational acceleration; \( N \) = coefficient of limiting wall acceleration; and \( g \) = gravitational acceleration. The limiting acceleration \( N_g \) of this wall is determined as 0.112\( g \) according to the method proposed by Richard and Elms 1979.

3. NUMERICAL APPROACH

Numerical modelling techniques are power full tools that have been used to study the permanent displacement of gravity retaining walls. The three-dimensional finite difference programme Fast Lagrangian Analysis of Continua (FLAC) 3D was used to carry out the numerical experiments. The gravity retaining wall model configuration and its properties adopted (according to Richard & Elms 1979) for numerical simulation are shown in Figure 1.

3.1 Numerical Grids and Problem Boundaries

The numerical grid for gravity retaining wall model used in the study is illustrated in Figure 2. The length of the wall is 2m. The wide backfill (3H) is considered, so as to facilitate the complete shear wedge (plastic zone) formation behind the retaining wall during base shaking. The size of foundation zone considered is 5H. Kuhlemeyer & Lysmer (1973) showed that for accurate representation of wave transmission through a model, the spatial element size \( \Delta l \) must be smaller than approximately one tenth to one-eighth of wave length associated with highest frequency component of the input wave. The retaining wall and the attached face of backfill with retaining wall is free to move in \( x \) direction. The horizontal movement of other faces of model are restrained.

![Numerical Grids of Retaining Wall Model](image)

3.2 Material Properties

The concrete gravity retaining wall is modelled as elastic material with the following properties. The bulk \( K \) and shear \( G \) modulus values of the wall were 11430 MPa and 10430 MPa respectively. The granular backfill was modelled as a purely frictional, elastic-plastic soil with a Mohr-Coulomb failure criterion. The reference friction angle of the soil was \( 30^\circ \), and density of soil \( \rho = 2000 \text{Kg/m}^3 \) (Richard and Elms 1979). The soil material is assigned constant values of shear modulus \( G = 80 \text{ MPa}, K =173.33 \text{ MPa} \) and Poisson’s ratio \( \mu = 0.3 \). The foundation is adopted as a rigid foundation. The fundamental frequency of backfill soil is calculated by linear elastic wave theory (Wu 1994) and is found to be 6Hz. The interface element at the base of the wall has been assigned a very large value of normal stiffness \( K_n = 1 \times 10^{12} \text{ N/m}^2/\text{m} \), thus restraining vertical movements of wall relative to the base. The shear stiffness of interface element at the base is kept small as \( K_s = 1 \times 10^5 \text{ N/m}^2/\text{m} \) to allow translational movement of wall with respect to the base. At backfill soil wall interface, very high value of normal stiffness, \( K_n = 1 \times 10^{13} \text{ N/m}^2/\text{m} \), is assigned to allow proper load transfer from backfill to the wall. The shear stiffness of backfill soil wall interface is kept zero to represent smooth back wall (Nadim & Whitman 1983).
3.3 Seismic Loading

The model is first brought to static equilibrium. The dynamic boundary condition is then applied. The free field boundary is applied at the foundation and quiet boundary is applied at the far end of the backfill. Sinusoidal excitation of 6 Hz frequency at 0.3g acceleration is applied as base motion for a period of 0.5 sec (Nadim & Whitman 1983). The base input motion applied to the model is shown in Figure 3. Several values of material damping, 5%, 10% and 15% are considered for backfill in various simulations.

![Figure 3: Typical Input Acceleration History (0.3g Acceleration at 6Hz Frequency)](image)

4. RESULTS AND DISCUSSION

A typical history of the relative displacement at the top of wall with respect to bottom of wall during the dynamic excitation of 6Hz frequency, 0.3g acceleration is shown in the Figure 4. Figure 5 shows typical acceleration histories recorded for the same model at the top, middle and bottom of the backfill. The locations of measuring histories are shown in Figure 2. From Figure 5 it is noticed that the accelerations recorded at different locations are increased with elevation.

![Figure 4: Typical Relative Displacement History at the Top of Wall](image)

4.1 Permanent Displacement of Wall

The permanent displacement of the wall at the end of the three cycles of sinusoidal acceleration is recorded from the model. The results obtained from numerical model study are compared with that of Richard & Elms (1979) model, as shown in Figure 6. The displacements obtained from RE model are slightly higher than that of numerical model. Simonelli et al. (2000) were also observed that the displacement obtained from analytical model at higher side.

![Figure 6: Permanent Displacement at Different Frequencies of Excitation](image)

4.2 Effect of Base Acceleration on Displacement of Wall

Effect of the base acceleration on the permanent displacement of the wall, subjected to dynamic excitation at 6 Hz frequency applied for 0.5 second, is shown in the Figure 7. Acceleration levels of 0.2, 0.3, 0.4 and 0.5 g were considered. From the graph it is observed that the permanent displacement increases with the increase in base acceleration. The increase of the displacement is almost linear with increase in base acceleration.
4.3. Effect of Damping

Effect of the damping property of the backfill soil on the dynamic response of gravity retaining wall is presented in Figures 6 and 7. Three levels of damping were considered like 5%, 10% and 15%. Figure 8 shows the effect of the damping on the maximum displacement of the gravity retaining wall for 6Hz frequency and 0.3g acceleration. It is found that maximum displacement decreases with increase in damping.

Figure 9 shows the effect of damping on acceleration histories at the top of the backfill for 6Hz frequency and 0.3g acceleration. Maximum accelerations recorded are raised up with increase in damping values. However, in general, it is observed that the accelerations recorded at the top of backfill are reduced with increase in the damping. The material damping should be judiciously selected for numerical simulation.

5. CONCLUSIONS

The displacement of gravity retaining wall can be determined with considerable accuracy by using FLAC 3D. But the selection of material properties like material damping influence the behaviour to considerable extent. The acceleration increased with the height of backfill. The accelerations are decreased with increase in damping of backfill.

REFERENCES


