

DYNAMIC EARTH PRESSURE AGAINST RETAINING WALLS

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ABSTRACT. The paper presents a simple method for calculation of static and dynamic active force on the rigid retaining wall following the pseudo-static approach of analysis and includes the effects of cohesion of the backfill and the friction between the backfill and the wall face. The resultant earth pressure obtained by this method has been compared with the experimentally observed values during small scale tests on retaining walls by other investigators. The displacement of retaining wall must not exceed specified values. The methods for calculation of displacement of rigid retaining walls are also discussed.

INTRODUCTION

There are two aspects in the design of retaining walls for earthquake loading:

- Calculation of static and dynamic lateral pressures.
- Calculation of displacement of the retaining wall.

The lateral earth pressure for static case is generally determined using either Rankine's [1] or Coulomb's [2] method. The earthquake induced forces on the retaining wall are generally computed using the modified Coulomb's approach in which the earthquake force on the backfill is replaced by an equivalent static force. This is known as Mononobe-Okabe method [3,4]. A solution for determination of static and dynamic active earth pressure for c-φ soils was developed earlier by Saran and Prakash [5]. This provides a convenient method for determination of static and dynamic lateral pressures for a typical soil. However, it has the following limitations:

- The effect of the vertical component of acceleration been neglected.
- The backfill surface is assumed to be horizontal, which may actually be inclined in many cases.
- The unit adhesion between the back face of the retaining wall and soil was assumed to be equal to the unit cohesion of the soil

A method for calculation of dynamic active pressure for a c-φ soil accounting for the effect of the following factors is presented here(Fig. 1).

The effect of cohesion, c, and adhesion, c_a.

The inclination of the backfill, i

Horizontal and vertical seismic coefficients, α_h and α_v respectively.

Surcharge, q

Inclination of the wall face, α

Depth of tension cracks, H_c

EARTH PRESSURE CALCULATION

A schematic diagram of dynamic earth pressure problem is shown in Fig.1. ABEC is an assumed failure wedge. Considering the unit length of the wall, (P_a)_{dyn} = total active force, R = soil reaction, I_F = horizontal inertia force, W = weight of assumed failure wedge, W_r=resultant of weight W and I_F, C_r = cohesion force, C_a =adhesive force, q = surcharge, α = inclination of wall face with vertical (α ≥ 0) and i = inclination of the backfill (θ ≤ i < φ).

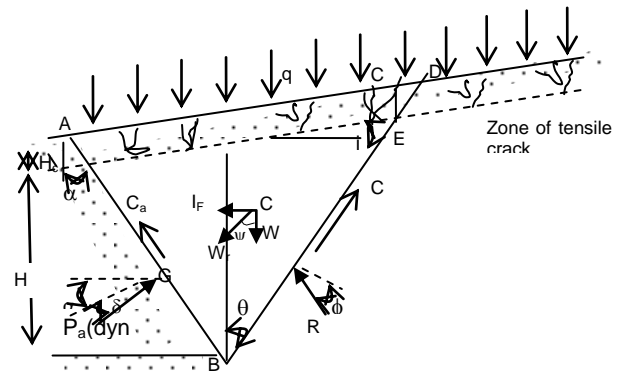


Fig. 1 Typical assumed failure wedge with forces acting on it.

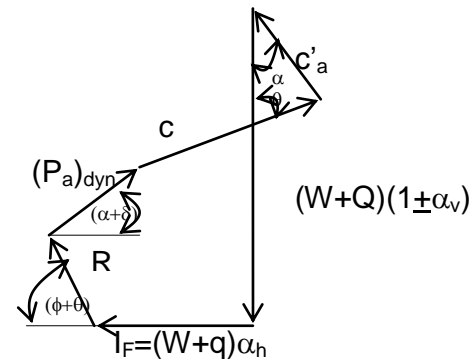


Figure 2. Polygon of Forces

The polygon of forces acting on the assumed failure wedge may then be drawn as shown in Fig.2. Applying the conditions of equilibrium and simplifying the results a relationship for (P_a)_{dyn} can be obtained as

$$(P_a)_{dyn} = \frac{1}{2} \gamma H^2 (N_{a\gamma})_{dyn} + qH(N_{aq})_{dyn} - cH(N_{ac})_{dyn} \dots\dots\dots(1)$$

in which,

$$(N_{ac})_{dyn} = \left[\frac{\cos(\alpha - i) \cos \phi}{\cos \alpha \cos(\theta + i)} + \frac{e \cos(\alpha + \phi + \theta)}{\cos \alpha} \right]$$

$$x \frac{1}{\sin(\alpha + \delta + \phi + \theta)} \dots\dots\dots(2)$$

$$(N_{aq})_{dyn} = \left[\frac{\sin(\alpha + \theta)}{\cos \alpha} + n \tan \alpha \cos \theta \right] x$$

$$\left[\frac{\alpha_h \sin(\phi + \theta) + (1 \pm \alpha_v) \cos(\phi + \theta)}{\sin(\alpha + \delta + \phi + \theta) \cos(\theta + i)} \right] \dots \dots \dots (3)$$

$$(N_{ay})_{dyn} = \frac{1}{\sin(\alpha + \delta + \phi + \theta)} \left\{ \tan \alpha + \tan \theta + \frac{n}{\cos i \cos(\theta + i)} [(2 + n) \tan \alpha \cos \theta + 2 \sin \theta] + \frac{\sin^2(\alpha + \theta) \sin i}{\cos^2 \alpha \cos \theta \cos(\theta + i)} \right\} \times [\alpha_h \sin(\phi + \theta) + (1 \pm \alpha_v) \cos(\phi + \theta)] \dots \dots \dots (4)$$

and

$$e = \frac{c_a}{c} \dots \dots \dots (5)$$

Where c_a = unit adhesion between the back face of the wall and the backfill, and c = unit soil cohesion.

The static earth pressure can be obtained from Eqs. 1-5, by making α_h and α_v as zero. The value of $(P_a)_{dyn}$ and $(P_a)_{stat}$ obtained from Eq. (1) are for the assumed failure wedge. In order to obtain the maximum values of the total dynamic earth force, $(P_a)_{dyn}$, the earth pressure coefficients $(N_{aq})_{dyn}$, $(N_{a\square})_{dyn}$, and $(N_{ac})_{dyn}$ were optimized. A computer code was developed for this purpose. It must be mentioned here that these earth pressure coefficients were individually optimized and then $(P_a)_{dyn}$ was obtained by superimposing their effect, i.e., using Eq. (1). The same procedure was followed for the maximum value of static earth force, $(P_a)_{stat}$. From known values of $(P_a)_{dyn}$ and $(P_a)_{stat}$, the dynamic increment $(\Delta P_a)_{dyn}$ can be obtained as

$$(\Delta P_a)_{dyn} = (P_a)_{dyn} - (P_a)_{stat} \dots \dots \dots (6)$$

COMPARISON WITH OBSERVED DATA

Sherif, et al. [6] reported results of measurements of dynamic active earth pressure on a 1 m high rigid retaining wall. The backfill properties are as follows:

- Unit weight, $\gamma = 16.28 \text{ kN/m}^3$
- Angle of internal friction, $\phi = 40.9^\circ$
- Angle of wall friction $\delta = 23.9^\circ$
- Slope of backfill = 0°

The wall was subjected to sinusoidal acceleration of up to 0.5 g. The results are shown in Fig..3. It may be noted that

$$K_{AE} = \frac{(P_a)_{dyn}}{\frac{1}{2} \gamma H^2} \dots \dots \dots (7)$$

The result obtained from Mononobe-Okabe theory and from the present study are also shown in Fig.3. It may be observed from Fig. 3 that the computed dynamic active earth pressures

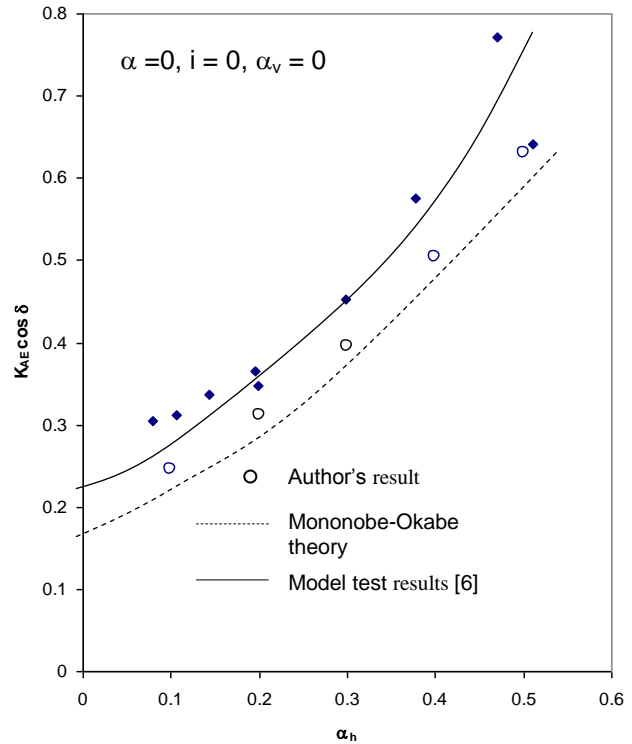


Fig.3. Translation of wall for active pressure – comparison of theory with model test results

computed by the proposed method show an excellent agreement with Mononobe-Okabe's method and are somewhat smaller than those observed by Sheriff et al [6]. The effect of various parameters on dynamic active earth pressure was investigated using equation 1. It was observed that a conservative value of dynamic active earth pressure can be obtained by assuming $c_a=c$.

POINT OF APPLICATION

Mononobe-Okabe's solution had assumed that the resultant active thrust acts at a distance of H/3 from the bottom of the wall similar to the static case ($\alpha_h = \alpha_v = 0$). The laboratory observations indicate that the resulting active thrust acts somewhat higher than H/3 measured from the bottom of the wall. Seed and Whitman [7] have suggested that for the case of rotation about the bottom of the wall, the static pressure may be assumed to act at H/3 and the dynamic increment at 0.6 H from the base of the wall. For a wall undergoing rotation about the top, the resulting active thrust may be assumed to act at 0.55H from the bottom of the wall [8]. It has been suggested that for wall undergoing translation, the line of the static active thrust may be assumed to act at 0.42 H and dynamic increment at 0.48 H above the bottom of the wall [7]. For all these cases the active thrust and dynamic increment are assumed to act at angle 'delta' with the normal to the wall face.

DISPLACEMENT OF RETAINING WALLS

The usual design procedure for a retaining wall does not ensure that its displacement will be within tolerable limits during an earthquake. Richard and Elms [8] developed a design procedure for gravity retaining walls based on limiting displacement. Nadim and Whitman [9] proposed a slight modification to Richard and Elm’s procedure primarily to account for the effect of ground amplification. It may be observed that while Richards and Elms procedure is relatively simple, uncertainties may arise due to errors in determination of material soil properties, assumptions in modeling and from nature of expected ground motion. Prakash et al [11] developed a model for simulating the response of rigid retaining walls subjected to seismic loading. This model consists of a rigid wall resting on the foundation soil and subjected to a horizontal ground motion and analyzed the problem as a case of combined sliding and rocking vibrations including the effect of various important parameters such as soil stiffness in sliding, soil stiffness in rocking, geometrical damping in sliding, geometrical damping in rocking ,material damping in sliding and , material damping in rocking. Only dry backfill was considered and seismic ground motion was represented by an equivalent sinusoidal motion. This model was further modified to accommodate both the dry and submerged backfills [12,13]

Several cases of 6.0 m high retaining walls were analyzed for typical cases of foundation soil conditions varying from well graded gravel (GW) to silt (ML) and the backfill soil varying from silty gravel (GM) to poorly graded sand (SP). Ground motions corresponding to El Centro, Loma Prieta and North Ridge earthquakes were used in the analysis. Typical case of a reference retaining wall , 6.0 m high , with nine different inclination angles of the wall face in contact with the backfill ‘ α ’ (0° , 1.25° , 2.5° , 3.75° , $+5^\circ$, -1.25° , -2.5° , -3.75° , and -5°) subjected to Northridge earthquake is used for illustration. Fig:4 shows the computed displacements of a wall with 0° , $+5^\circ$ and -5° inclination angles at the back. The negative angle at the back of the wall is the case of the wall resting on the backfill. Figure 4 shows cumulative displacement of the retaining wall away from the backfill due to combined sliding and rocking effects for $\alpha = -5^\circ$, 0° and $+5^\circ$ for a base width of 3.57 m. The foundation soil for this case was well graded sand (SW) and the backfill consisted of submerged silt gravel (GM). It can be observed from Fig. 4 that the negative values of ‘ α ’ result in somewhat smaller cumulative displacements compared to the case of vertical wall face ($\alpha = 0$) or for positive value of ‘ α ’ within range of values of ‘ α ’ considered in the analysis.

Another typical plot of cumulative displacement of a rigid retaining wall of 6.0 m height and having a base width of 4.61m and subjected to North Ridge earthquake motion is shown in Fig 5. The foundation and backfill soils in this case were silt of low compressibility and silty gravel respectively. The trend of the results in Fig: 5 is similar to that in Fig: 4. Similar results were observed for other cases.

Table 1 shows a summary of new base widths and computed displacement for various inclinations. The computed cumulative sliding, , rocking and total displacements are also shown in this table . The base widths decreased from 3.57m to 3.38m as the inclination changed from 0° to -5° , since the active earth

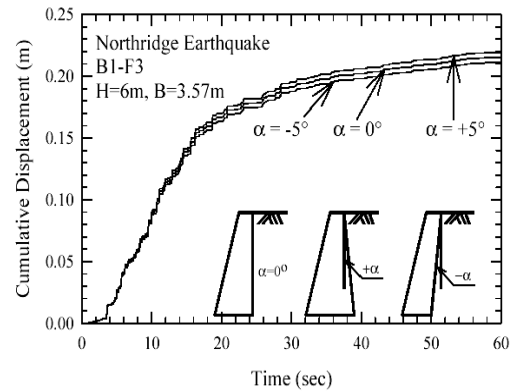


Fig: 4. Cumulative displacements of walls (B1-F3) with different inclinations with the vertical

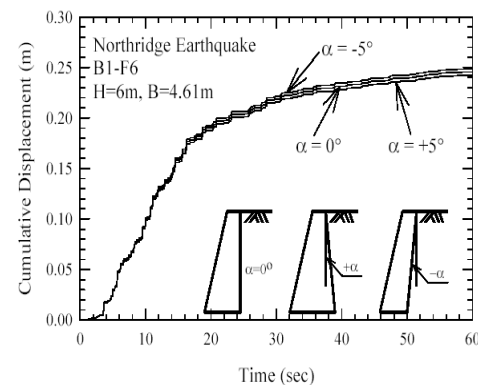


Fig:5.Cumulative displacements of walls (B1-F6) with different inclinations with the vertical forces decrease with negative inclination. Therefore, the base width was smaller for a wall with a negative inclination.

The angular rotation in rocking (Table 1) decreased from 1.29° ($\alpha=0$) to 1.25° ($\alpha=-5^\circ$), and the total displacements decreased slightly from 0.2155m to 0.2112m. The cumulative displacements for these walls will not be significantly altered by changing the inclination at the back of the wall.

For the wall built as a leaning-type rigid retaining wall with $\alpha = -5^\circ$ lying on the backfill, the wall experienced a rocking movement of 1.25° during the Northridge earthquake. Therefore, when the wall was subjected to the same earthquake event up to 3 or 4 times, the wall experienced a total rocking close to 5° . At this time, the wall may become vertical. Further analysis was conducted for 21 backfill and foundations soil combinations for a typical reference wall 6m high, subjected to three earthquakes . The backfill soil was varied from silty gravel to poorly graded sand the foundation soil varied from well graded gravel to silt of low compressibility.

Table1. Cumulative displacement for several angles of inclination at the back of a wall subjected to Northridge earthquake condition (B=3.57m).

Inclination angle (degree)	Base width (m)	Cumulative Displacement by Fixed Base Width (3.57m)			
		Sliding (m)	Rocking (degree)	Rocking (m)	Total (m)
+5.00°	3.81	0.0820	1.31	0.1374	0.2194
+3.75°	3.76	0.0820	1.30	0.1366	0.2186
+2.50°	3.70	0.0815	1.30	0.1361	0.2176
+1.25°	3.63	0.0808	1.29	0.1355	0.2163
0.00°	3.57	0.0808	1.29	0.1347	0.2155
-1.25°	3.50	0.0806	1.28	0.1338	0.2144
-2.50°	3.43	0.0805	1.27	0.1329	0.2134
-3.75°	3.35	0.0803	1.26	0.1320	0.2123
-5.00°	3.38	0.0801	1.25	0.1311	0.2112

The results generally indicated that the design widths of foundations for 21 cases of backfill – foundation soil combinations used in analysis generally reduced with values of α from 0° to -5°. This may result in saving of 8 - 10 % in the material cost.

CONCLUSIONS

1. Procedure is presented to determine the magnitude of the static and dynamic active thrust for a typical $c-\phi$ soil accounting for the effect of wall friction, adhesion between the soil and the wall face and the inclination of the backfill surface.

2. A retaining wall with negative inclination seems to be a better alternative in seismic areas to ensure smaller overall displacement and economy.

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