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A Critical Review on Seismic Design of Earth-Retaining Structures

Kenan Hazirbaba [1], Omer Mughieda [2] and Ghassan Abu-Lebdeh [3]

[1] Associate Professor, Department of Civil and Infrastructure Engineering, American University of Ras Al Khaimah, P.O.Box 10021, Ras Al Khaimah, UAE. Corresponding Author.
E-Mail: kenan.hazirbaba@aurak.ac.ae
[2] Associate Professor, Abu Dhabi University, Abu Dhabi, UAE.
[3] Professor, American University of Sharjah, Sharjah, UAE.

ABSTRACT

The dynamic response of earth-retaining structures is a complex issue. The design of these structures under seismic loading conditions is, therefore, quite challenging. Because there are not many case histories on field performance of earth-retaining structures, the theory of dynamic response has been based on the results from both model tests and numerical analyses. The most common approach to the seismic design of retaining walls, especially where the expected ground acceleration is less than or equal to 0.29g, involves estimating the loads imposed on the wall during earthquake shaking and then ensuring that the wall can resist those loads. Because the actual loading on retaining walls during earthquakes is extremely complicated, seismic pressures are usually estimated using the Mononobe-Okabe pseudo-static method. The Mononobe-Okabe equation is an extension of the Coulomb's classical solution, which accounts for inertial forces. Another approach for designing against seismic loading is to allow for an "acceptable permanent deformation" and determine the dimensions of the structure accordingly. This approach is typically more appropriate for retaining structures to be built in highly seismic regions, where the peak ground acceleration for the design earthquake is larger than 0.29g. Neither the Mononobe-Okabe method nor the displacement method accounts for the characteristics of the ground motion. More realistic dynamic response analysis can be performed using the results from laboratory studies and numerical models. This paper presents the state-of-the-art for the design of earth-retaining structures under dynamic loading conditions. The commonly used simplified design procedures were presented. Relatively recent laboratory studies and numerical solutions were reviewed. Some case histories regarding the seismic performance of retaining walls were also listed.

KEYWORDS: Earth-retaining structures, Seismic design of earth-retaining walls, Earthquake performance of earth-retaining structures, Case histories of earth-retaining structures, Force and displacement methods.

INTRODUCTION

Earth-retaining structures are commonly used for supporting excavations and earth fills. The design principle of earth-retaining structures is to safely transmit the pressure of earth being retained to the supporting ground. From statics point of view, this is a relatively straightforward task; the design requirements dictated by the dominant lateral earth pressure (i.e., sliding, overturning) are analyzed and the structure is designed using proper factors of safety. However, this gets much more challenging in case of seismic forces, such as those created by an earthquake, because of the

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nature of dynamic loading. Both the retaining structure itself and the supporting ground are impacted. Lateral earth pressures increase while the resistance and bearing capacity of the supporting ground decrease under dynamic loading. The common and simplified approach for seismic deign of earth-retaining structures is known as the pseudo-static method, where dynamic loading is converted into an equivalent uniform static load. Other approaches for seismic design of earth-retaining structures include displacement-based method and more advanced finite element/finite difference (FE/FD) methods. This paper provides a practical review of the most common procedures of seismic design for earth-retaining structures.

Background Information

Seismic design of earth-retaining structures is one of the most challenging tasks undertaken by geotechnical engineers, as discussed above. In the following sections, the details of: (1) force-based approach and (2) displacement-based approach are presented with a comparative discussion.

Force-Based Approach (The Mononobe-Okabe Pseudo-static Method)

The most common method for evaluating dynamic lateral forces for design of retaining structures is the Mononobe-Okabe equation. This equation is a modified version of Coulomb's classical solution to account for intertial forces. The earthquake shaking is represented by pseudo-static accelerations that produce inertial forces, F_h and F_v, which act through the centroid of the failure mass in the horizontal and vertical directions, respectively (Figure 1). The magnitudes of the pseudo-static forces are:

$$F_h = \frac{a_h.W}{g} = k_h.W \tag{1}$$

$$F_v = \frac{a_v.W}{g} = k_v.W \tag{2}$$

where a_h: horizontal pseudo-static acceleration, a_v: vertical pseudo-static acceleration, k_h: coefficient of horizontal pseudo-static acceleration, k_v: coefficient of vertical pseudo-static acceleration and W: weight of the failure mass.

According to the Mononobe-Okabe method, the total active thrust against a wall, for active earth pressure conditions (Figure 1), can be expressed in the following form:

$$P_{AE} = \frac{1}{2}\gamma(1-k_v)H^2 K_{AE} \tag{3}$$

where γ: the unit weight of the backfill, H: the height of the wall and K_{AE}: the active stress coefficient, which is a function of the friction angle of the backfill, the friction angle between backfill and wall and the acceleration coefficients. For practical purposes, Whitman (1990) presented an approximate, linear relationship between the static coefficient of active earth pressure, K_A and K_{AE}:

$$K_{AE} = K_A + \frac{3}{4}k_h \tag{4}$$

The total active thrust, P_{AE}, consists of two components; the static component P_A and the dynamic component ΔP_{AE}. Thus,

$$P_{AE} = P_A + \Delta P_{AE} \tag{5}$$

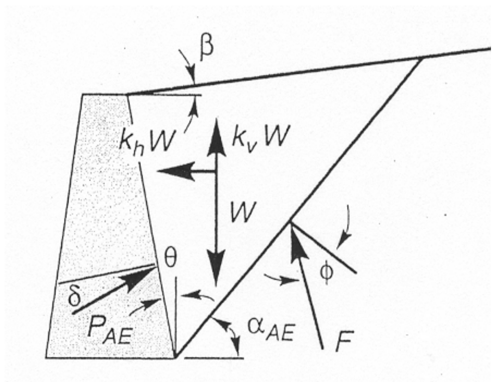


Figure (1): Forces acting on active wedge in Mononobe-Okabe analysis (Kramer, 1996)

The static component acts at H/3 above the base of the wall and the dynamic component can be assumed to act at 0.6H (Kramer, 1996). Hence, the point of application for the total active thrust can easily be obtained from moment equilibrium.

The Mononobe-Okabe method is based on a number of assumptions. The most important assumptions are:

(1) The earth-retaining structure has already deformed outwards, thus the generated active pressure is minimum.

(2) A Coulomb wedge, with a planar sliding surface running through the base of the retaining structure, is on the point of failure with a maximum shear strength developed along the length of the surface.

(3) The soil behind the retaining structure behaves as a rigid body, so that accelerations can be assumed uniform throughout the backfill at the instant of failure.

The first two assumptions are generally satisfied. On the other hand, for the last assumption to hold true, the soil needs to be rigid with an infinite shear wave velocity. In reality, the finite shear wave velocity, and hence the shear modulus, tends to decrease towards the ground surface in a cohesionless backfill. This will cause an amplification of the ground motion between the base of the retaining structure and the ground surface (Steedman, 1998). Amplification of the ground motion will increase the dynamic lateral pressures exerted on the earth-retaining structure. It is obvious that an assumption of uniform acceleration throughout the backfill will result in unrealistic dynamic pressures.

Displacement-Based Approach

The method proposed by Richards and Elms (1979) for dynamic evaluation of retaining structures is based on the Newmark's sliding block analysis (1965). The Richards-Elms method assumes that:

(1) The retaining wall is rigid.

(2) The inertial forces due to the mass of the wall are included.

(3) The backfill is dry and the backfill failure wedge slides as a rigid body with the retaining wall.

(4) After the horizontal ground acceleration (*Ag*) exceeds the yield acceleration (*Ng*), the wall moves away from the backfill until the direction of the wall motion changes.

Then, the displacement can be expressed in the following form:

$$d_s = 0.087.\frac{V^2}{Ag}.\left[\frac{N}{A}\right]^{0.25} \tag{6}$$

where d_s: total relative displacement in inches, A: peak horizontal ground acceleration coefficient, V: peak earthquake velocity in inches/sec and N: coefficient of limiting wall acceleration.

The displacement-based analysis can be performed using the following steps:

1. Select the permissible displacement d_s.
2. Determine A and V from a given seismic zone.
3. Determine the coefficient of limiting wall acceleration (N), also known as the cutoff acceleration, by rearranging Equation 6:

$$N = A.\left[\frac{0.087.V^2}{Ag.d_s}\right] \tag{7}$$

4. Compute dynamic active lateral earth pressure behind the wall by using the Mononobe-Okabe method for N computed in step 3.
5. Compute wall weight by using the inertial force of the wall and consider force equilibrium.
6. Apply a factor of safety to the calculated weight and then, determine the dimensions of the wall. Typically, a factor of safety of 1.5 is recommended, but smaller values have proven to be sufficient.

The real challenge in designing by the displacement-based method is faced during the selection of the permissible displacement. Richards and Elms did not specify how to determine a permissible displacement.

Richards and Elms procedure is relatively simple, yet it has large sources of uncertainty arising from the determination of the actual soil properties, from assumptions in the modelling and from the nature of the expected ground motions.

Mononobe-Okabe Method versus Richards and Elms Procedure

In the following numerical example in Table 1, the force-based method and the displacement method were compared for a 6m high wall with the following data:
c =0; f=34^o; d=17^o; g=17.3 kN/m^3; k_h=k_v=0.3.

Table 1. Comparison of Mononobe-Okabe solution and Richards and Elms procedure

Methods	Mononobe-Okabe (Force-based)	Richards and Elms (Displacement-based)
Seismic active earth pressure, P_{AE} (kN/m)	145.16	100.92
Point of application of P_{AE} from base (m)	2.72	2.33
Displacement (mm)	-	100

Laboratory and Numerical Studies

The pseudo-static force-based and displacement methods do not take into account the effects of duration of earthquake loading, frequency content and acceleration amplification through the backfill soil. Laboratory and numerical models become very useful in understanding the effects of dynamic loading on earth-retaining structures and in developing more realistic design procedures. In this section, laboratory and numerical studies will be reviewed.

Laboratory Studies

Elms and Richards (1990) carried out small scale, two-dimensional tests to investigate whether the Newmark's sliding block model could be justified. They used a special test set-up, which accommodated a 0.81m high wall with varying mass (Figure 2). The unreinforced wall was free to fail by sliding. The backfill soil was beach sand, placed in a reasonably dense state. Decaying sinusoidal excitations were applied by means of a large spring. In their effort, Elms and Richards (1990) compared the base input acceleration with the wall response and found that the sliding-block behavior occurred only after the failure plane started to form in the backfill. Figure 3a shows the initial acceleration response for the first spring impulse. The wall response does not have the flat plateau presumed by the sliding block model. In contrast, after failure plane developed during the third spring impulse, the wall started to exhibit a flat plateau at approximately the same amplitude for all cycles (Figure 3b). Based on these findings, Elms and Richards concluded that the sliding block model can accurately predict the cutoff acceleration and total displacement provided the residual and not the peak value of friction angle of backfill soil is used for calculation of Mononobe-Okabe dynamic earth pressure.

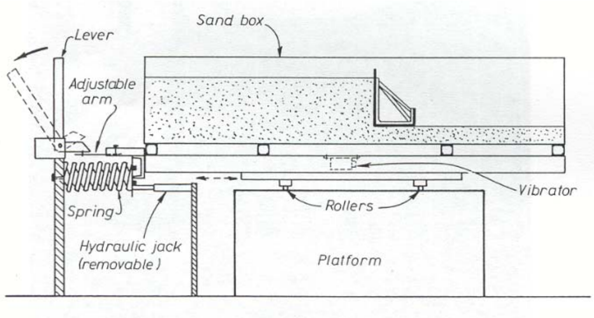
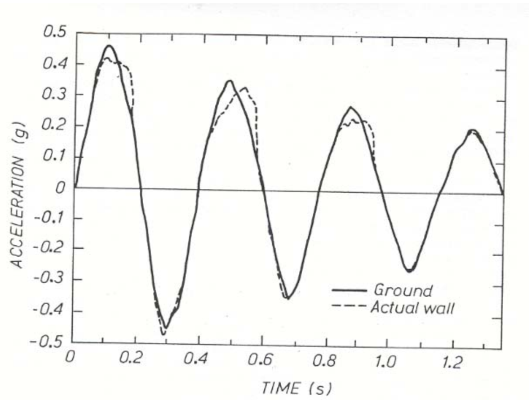
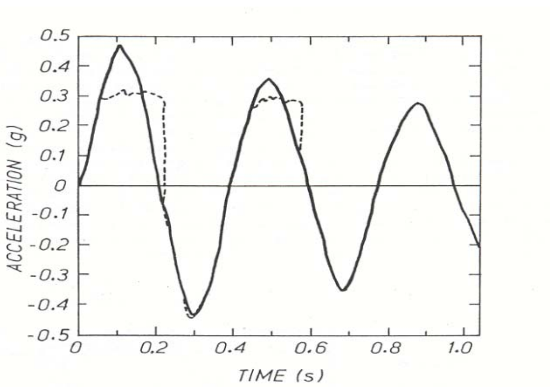


Figure (2): Experimental set-up for verification of the sliding block theory (Elms and Richards, 1990)



(a) Initial acceleration response



(b) Response after failure developed

Figure (3): Acceleration response of the tested wall by Elms and Richards

Elms and Richards also tested a reinforced wall model using the same set-up. The response was similar to that of unreinforced case, except that the failure plane was found to be bi-linear rather than linear in the case of reinforced wall. They recommended the use of residual strength for the reinforced wall design as well.

In an unpublished study in Japan, a series of shaking table tests were performed on a half-scale reinforced soil wall model (Bathrust and Hatami, 1998). The model wall was 3m high and 5.2m long from the toe to the back of the shaking table container. The reinforcement was provided by 4m long metal strips that were applied in eight layers. The wall was constrained at the toe against horizontal movement and the base was excited by sinusoidal input accelerations. The response of the wall was recorded for different peak base accelerations ranging between 0.1g and 0.4g. The frequency effect was also studied for a range from 2 Hz to 7 Hz. This investigation showed that the resulting maximum reinforcement loads were approximately the same in all layers and increase linearly with increasing peak base acceleration. It was also found that the backfill caused significant amplification in the peak base acceleration.

Sakaguchi (1996) conducted shaking table tests on a 1.5 m high reinforced wall model. He found that wall displacements and permanent strains in the reinforcement accumulated with time. The reinforced section of the wall was observed to act as a rigid body even after large wall displacements developed.

Numerical Studies

Whitman and Liao (1985) reconsidered the sliding block model. They proposed alternative design equations based upon the probability of non-exceedance of a chosen allowable displacement. They suggested the following equations to be used instead of Equations (6 and 7) in Richards and Elms procedure:

$$d_s = \frac{37.V^2}{Ag}.e^{(-9.4\frac{N}{A})} \quad (8)$$

$$N = 0.106.\ln\left[\frac{37.V^2}{Ag.d_s}\right] \quad (9)$$

A mathematical model for simulating the response of rigid walls during an earthquake has been proposed by Rafnsson and Prakash (1994). This model considered the sliding and rocking motions of a rigid retaining wall subjected to horizontal ground motion. The model also considered nonlinear behavior of soil both at the base as well as the backfill. The details of the model can be found in Rafnsson and Prakash (1994). Wu and Prakash (1996) presented the following seismic design steps:

1. The wall dimensions are determined for given factors of safety under static condition.
2. Displacements and rotation of the wall are computed for different loading cycles for a given ground motion.
3. The computed displacements are compared with the permissible displacement. If computed displacements are larger, then the wall needs to be redesigned for permissible displacements.

Comprehensive analysis of seismic performance of geosynthetic-reinforced soil walls was carried out through a series of numerical studies by Bathrust and Cai (1995). The research demonstrated that increased lateral pressures behind segmental walls due to seismic loading could be accounted for using the Mononobe-Okabe equations. Internal and external stability was addressed by increased reinforcement layers and increased reinforcement lengths. It was concluded that the vertical component of seismic motion might be disregarded in terms of practical seismic stability design, because peak vertical accelerations do not occur simultaneously with peak horizontal accelerations.

Ling et al. (1997) and Ling and Leshchinsky (1998) proposed a pseudo-static analysis considering the internal and external stabilities of reinforced soil structures. The procedure is an extension of the design procedure proposed by Leshchinsky et al. (1995). The authors also considered the procedure for a permanent displacement analysis. From a series of parametric studies, the authors concluded that in the event of large earthquake, external stability, typically by direct sliding, might govern the design. That is, a longer geosynthetic length is required for design in addition to stronger reinforcement in resisting the earthquake inertia force.

Case Histories

Observed performance of reinforced soil structures following the occurrence of significant earthquakes is presented below.

Current Practice in North America

In North America, geosynthetic- and metal strip-reinforced soil walls are generally designed using limit-equilibrium pseudo-static methods for sites with peak horizontal accelerations ≤ 0.29g (Bathrust and Hatami, 1998). The most commonly used guidelines for seismic design of earth-retaining structures are: (1) The FHWA (2001) guidelines for MSE walls and (2) NCMA (1998) seismic design manual for segmental retaining walls. The FHWA design procedure evaluates both external seismic stability and internal seismic stability. The external stability evaluation is performed using a modified version of the Mononobe-Okabe method. The details can be found in the design manual.

Table 2. A list of seismic field performance of reinforced soil structures (after Nova-Roessig, 1999)

Earthquake, Country, Year	Mag. (Mr)	Distance to Epicenter (km)	Horiz. Accel. (g)	# of Walls	Wall Type	Wall Height (m)	Seismic Design (Yes/No)	Reported Damage
Gemona, Italy, 1976	6.4	25-40	-	3	RE™	4-6	No	None
Leige, Belgium, 1983	5.0	0.8	0.15-0.20	2	RE™	4.5-6	No	None
Honshu, Japan, 1983	7.7	80-275	0.1-0.3 (@140km)	49	RE™			One wall only; few cm of settlement
Edgecumbe, NZ, 1987	6.3	30	-	1	RE™	6		None
Chiba-Ken Toho-Oki, Japan, 1987	6.7	40	0.22-0.33	2	Nonwoven Geotextile	5.5		None
Loma Prieta, CA, USA, 1989	7.1	11-100	0.1-0.55	20	RE™	5-10	Some	None
		11-130	0.1-0.4	>1	Geogrid	4.4		One wall; 0.2%H movement (top)
Kushiro-Oki, Japan, 1993	7.8	40	0.30	1	Geogrid	4.4		None
Northridge, CA, USA, 1994	6.7	2.5-84	0.1-0.9	20	RE™	4-17		Panel spalling, cracking
		61	0.1	1	MSE	16		Bulged at center (3%H)
		8-113	0.2-0.5	>1	Geogrid	3-15		None
		19	0.35	1	MSE	12		Cracks;2.5cm differential settlement
Hyogoken-Nanbu, Japan, 1995	6.9 (Mw)	16-40	up to 0.8	3	Fiber grid	3-8	Yes	None
		16	up to 0.8	1	Fiber grid	6	Yes	30cm lateral movement; panel spalling; cracks
Chi-Chi, Taiwan, 1999	7.3	15-40	up to 1.0	6	Geogrid	2-40		Cracks; 2.5cm differential settlement; bulging
Nisqually, WA, USA, 2001	6.8	23	up to 0.25	1	Geogrid	4		Collapse

Concluding Remarks

A review of the common seismic design procedures for earth-retaining structures was presented in this paper. The three commonly used techniques to design earth-retaining structures; namely, the force-based approach, the displacement-based approach and the FE/FD-based approach were discussed. The current paper provides a synthesis from designers' perspective and is meant to serve as a reference for geotechnical practitioners. The following concluding remarks may be drawn:

1. The problem of retaining wall design for earthquake loading is complex. However, either reasonable simplifications or sophisticated computational tools, such as FE/FD methods, can help in providing applicable solutions.
2. Existing solutions appear to provide a good basis for design, at least for commonly used earth-retaining structures.
3. Simple reliance on a force-based approach, where strong ground motions are expected, is not likely to be a reasonable solution. Because it is unrealistic to expect the backfill soil to behave as a rigid body, especially when subjected to strong ground motions, amplification and substantial increase in dynamic lateral pressure should be expected. Therefore, care needs to be exercised when considering the force-based approach.
4. In case of design against strong ground motion, the displacement-based approach or a comprehensive dynamic response analysis through more sophisticated computations is required.
5. Current studies tend to focus on more realistic seismic design which considers all likely modes of deformation.

NOTATIONS

A : Peak horizontal ground acceleration coefficient
c : Cohesion of the backfill soil
d_s : Total relative displacement (inch)
F_h : Pseudostatic horizontal force
F_v : Pseudostatic vertical force
k_h : Coefficient of horizontal pseudostatic acceleration
k_v : Coefficient of vertical pseudostatic acceleration
N : Coefficient of limiting wall acceleration (cutoff acceleration)
P_A : Static active thrust
P_{AE}: Total active thrust
W : Weight of the failure mass
K_{AE}: Dynamic active earth pressure coefficient
K_A : Static active earth pressure coefficient
V : Peak earthquake velocity (in/sec)
ΔP_{AE}: Dynamic component of the total thrust
δ : Angle of interface friction between soil and wall (^{0})
ϕ : Friction angle of the backfill soil (^{0})
γ : Unit weight of backfill soil (kN/m^3)

ABBREVIATIONS

FHWA : Federal Highway Administarion
MSE : Mechanically Stabilized Earth Walls
NCMA : National Concrete Masonry Association
RE™ : Reinforced Earth Co. Wall

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