SEISMIC DESIGN OF GRAVITY RETAINING WALLS WITH REFERENCE TO THE MASSACHUSETTS BUILDING CODE

Cetin Soydemir[1]

I. INTRODUCTION

The comprehensive state-of-the-art report by Seed and Whitman (22) in 1970 brought a new awareness to the importance of earthquake resistant design of earth retaining structures. The report also incorporated design recommendations expanding the pioneering work of Mononobe (17) and Okabe (19) presented in the 1920's. The so-called Mononobe-Okabe pseudo-static method is still the most widely used procedure in practice for the seismic design of earth retaining structures.

A survey of literature indicates that other than Prof. L. S. Jacobsen's (11) illuminating work carried out at Stanford University (1930's) relative to the major projects of the Tennessee Valley Authority, research on the seismic behavior of earth retaining structures prior to the 1960's was primarily undertaken in Japan. However, during the last decade, researchers in the United States, New Zealand, and India have made significant theoretical and experimental contributions.

Up until the 1970's, work carried out has been almost exclusively an expansion of the Mononobe-Okabe pseudo-static model dealing only with unsaturated backfills. Seismic design of retaining structures with saturated backfills (not subject to liquefaction) is still at an early stage of development and quite empirical, even though major structures such as quay walls and drydock walls belong to this category.

In 1979 Richards and Elms (21, 4) proposed a new approach for the seismic analysis of gravity retaining walls as an alternative to the previous pseudo-static (seismic coefficient) methods. This fundamentally sound approach is based on the fact that under seismic activity gravity walls essentially experience a finite, permanent displacement rather than a complete collapse (i.e., failure).

This paper in general deals with the Richards-Elms model, and reviews it with particular reference to the Massachusetts State Building Code (25), Section 716.6.10, Retaining Walls. Richards-Elms method has been adopted in current design practice in the United States (5) and New Zealand (15).

---

(1) Senior Engineer, Haley & Aldrich, Inc., Cambridge, MA.

II. MONONOBE-OKABE PSEUDO-STATIC METHOD

The Mononobe-Okabe (M-O) pseudo-static (seismic coefficient) method is an extension of the Coulomb sliding-wedge theory taking into account horizontal and vertical inertia forces acting on the retaining wall. The analysis was described in detail by Seed and Whitman (22). The method makes the following assumptions:

1. The wall is free to move sufficiently that the soil shear strength will be mobilized along the potential failure surface (i.e., active limiting equilibrium condition).
2. The backfill consists of unsaturated cohesionless material.

Force equilibrium consideration on the soil wedge (Figure 1)* behind the wall leads to the magnitude of P_{AE}, the combined static and dynamic (seismic) load exerted on the wall (and vice versa):

*At end of paper.

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

where,

K_{AE} = the seismic active pressure coefficient given by the M-O expression:

$$K_{AE} = \frac{\cos^2(\phi - \theta - \beta)}{\cos\theta\ \cos^2\beta\ \cos(\delta + \beta + \theta)\left[1 + \left(\frac{\sin(\phi + \delta)\ \sin(\phi - \theta - i)}{\cos(\delta + \beta + \theta)\ \cos(i - \beta)}\right)^{1/2}\right]^2} \tag{2}$$

(see Figure 1c):

γ = total unit weight of soil (unsaturated)

H = height of wall (soil face)

ϕ = angle of internal friction of backfill

δ = angle of friction at wall-backfill interface

i = backfill slope angle

β = slope angle of wall at soil-wall interface (negative, as shown)

$\theta = \tan^{-1} \dfrac{k_h}{1 - k_v}$

k_h, k_v = horizontal and vertical acceleration (seismic) coefficient, respectively.

Also in reference to Figure 1,

W_s = weight of the soil failure-wedge

W_w = weight of the wall (includes the weight of the confined backfill in Figure 1b)

h_a = line of action of the resultant soil force measured from the base of the wall.

Whitman (26) noted that Eq. 1 essentially corresponds to the static Coulomb equation for the condition where the wall and backfill are rotated by an angle θ such that i becomes $(i + \theta)$ and β becomes $(\beta + \theta)$.

In Figure 2, $K_{AE} \cos \delta$ has been plotted vs. k_h for the condition $k_v = \beta = i = 0$, $\delta = 1/2 \ \phi$. A straight line approximation for the curves has also been incorporated (26). From Figure 2, for the simplified straight line approximation:

$$K_{AE} \cos \delta = K_A + 3/4 \ k_h \tag{3}$$

where, K_A = the static active coefficient.

The first and second term (right side) of Eq. 3 corresponds to the static and dynamic components of the M-O coefficient, K_{AE}, respectively. Eq.3 was originally proposed by Seed and Whitman (22).

Rewriting Eq. 3 with $\delta = 0$:

$$K_{AE} = K_A + \Delta K_{AE} \tag{4}$$

where,

ΔK_{AE} = active coefficient for the dynamic increment, one obtains:

$$\Delta K_{AE} \cong 3/4 \ k_h \tag{5}$$

Thus, the dynamic soil force component is:

$$\Delta P_{AE} \cong 1/2 \ \gamma \ H^2 \ 3/4 \ k_h = 3/8 \ \gamma \ H^2 \ k_h \tag{6}$$

III. MASSACHUSETTS BUILDING CODE ON SEISMIC DESIGN OF RETAINING WALLS

Section 716.6.10 (Retaining Walls) of the Code (25) specifies that:

"Retaining walls shall be designed to resist at least the superimposed effects of the total static lateral pressure, excluding the pressure caused by any temporary surcharge, plus an earthquake force of 0.045 $\gamma_t H^2$ (horizontal backfill surface). The earthquake force from the backfill shall be distributed as an inverse triangle over the height of the wall...If the backfill consists of loose saturated granular soil, consideration shall be given to the potential liquefaction of the backfill during the seismic loading."

In this study, only unsaturated backfills are considered. Liquefaction problem and its related effects were investigated earlier (23) in reference to Massachusetts.

"DESIGN EARTHQUAKE" FOR MASSACHUSETTS

In Section 716.7 (Dynamic Analysis), the Code (25) specifies that:

"Any building or structure is deemed to have complied with the provisions of Section 716.0 if a qualified registered engineer determines that there is negligible risk to life safety if the building or structure experiences an earthquake with a peak acceleration of 0.12 g and a frequency content similar to that implied by the appropriate response spectrum in Figure 716.2".

Figure 716.2 of the Code (25) also incorporates amplification effects of local subsoil conditions (i.e., thickness, strength) supporting a structure. Section 716.7 does not make any direct reference to the magnitude or duration of the "design earthquake".

The Code's "design earthquake" with a peak acceleration of 0.12 g is primarily proposed for buildings. However, it also provides a basis for the seismic input to be considered in earthquake design of retaining walls. The selection of a "design acceleration coefficient" for pseudo-static seismic analysis of a proposed facility (e.g., building, retaining wall, gravity or embankment dam) is indeed a difficult task and often done arbitrarily.

SOME NOTES ON SECTION 716.6.10 OF THE CODE

Section 716.6.10 (25) considers retaining walls in general. Quite understandably, the Code does not make specific reference to the type of wall (e.g., gravity wall, cantilever wall, bridge abutment, building basement wall, quay wall). This truly important aspect must not escape the attention of the design engineer who may use the same criterion for all types of earth retaining structures regardless of their structural rigidity-flexibility and yielding-nonyielding character.

It is also noted that Section 716.6.10 (25) does not make a reference to the inertia effect of the retaining wall itself. It is not known whether this is intentional or not, since "current procedures generally assume that the inertia forces due to the mass of the retaining wall itself may be neglected in considering the seismic behavior and seismic design of gravity retaining walls." (21, pg. 454) This is certainly not correct, since the weight of the wall provides the primary resisting potential against movement of the wall. The inertial loading on the wall itself should definitely be included in design analysis (22, 27).

BASIS OF SECTION 716.6.10.

Considering Eq. 6 and substituting k_h=0.12, one obtains:

$$\Delta P_{AE} \cong 3/8\ \gamma H^2\ (0.12) = 0.045\ \gamma H^2 \qquad (7)$$

which is the expression specified by the Code to determine the dynamic (seismic)loading acting on a retaining wall during ground shaking.

As to the line of action of the dynamic component, ΔP_{AE}, the original"Mononobe-Okabe solution by itself tells nothing about location of the dynamic earth pressure." (26, p. 1442) Based on test data (model walls on shaking table) reported by Jacobsen (11) and Matsuo (16) and theoretical analysis of Prakash and Basavanna (20), Seed and Whitman (22) suggested that the resultant dynamic increment acts at a height varying from 0.5H to 0.67H above the base of the wall. Section 716.6.10 (25) reflects this opinion. Later Whitman (26) restated that as a simplification, while the static component, P_A, acts at 1/3 of the wall height, the additional dynamic component, P_{AE}, acts at about 2/3 of the wall height.

IV. MODE OF EARTHQUAKE INDUCED DAMAGE ON RETAINING WALLS

In their state-of-the-art review Seed and Whitman (22) for unsaturated backfills cited a retaining wall failure in 1970 Chilean earthquake, and outward movement of the wingwalls of a bridge in 1964 Niigata earthquake. On the lack of reported cases of earthquake induced damage on retaining walls, Seed and Whitman suspected that this may be more so due to the type of damage not being as dramatic compared to other rather catastrophic failures.Seed and Whitman, however stated that "many earthquake damage reports contain accounts of the movements of bridge abutments due to increased lateral pressures resulting from earthquake effects. In such cases, wall movement causes severe distortion or possibly collapse of the bridge superstructure." (22, p. 140) Thus, Seed and Whitman concluded that the possibility of movements of earth retaining structures due to earthquake induced increased lateral pressures must be a significant design problem in seismic regions.

More recent reported cases have confirmed that earthquake induced movements of abutments indeed played a major role in associated bridge damage. Evans (7) surveyed the damage to bridges in the 1968, M = 7 Inangahna earthquake in New Zealand. Out of 39 bridges within 30 miles of the epicenter, 23 showed distinct abutment movement and 15 experienced structural abutment failures. Abutment movements followed the pattern of outward lurching relative to the base, and rotation about the top due to restraint by the bridge deck. Ellison's (3) pictorial report illustrated the damage on bridge abutments in the 1970, M = 7.1 Madang earthquake in New Guinea, where some abutments moved as much as 20 inches. Fung et. al. (9) and, Clough and Fragaszy (2) reported damages to abutments and floodway retaining structures in the 1971 San Fernando, California earthquake. A common feature in these case histories is the small to excessive outward movement of bridge abutments and other retaining structures caused by earthquake induced large lateral earth pressures.

V. RICHARDS-ELMS MODEL BASED ON SLIDING BLOCK ANALOGY

In 1979 Richards and Elms (21, 4) proposed a procedure for earthquake design of retaining walls, which was based on field observations that a retaining wall does not

fail when the base ground acceleration results in a sliding factor of safety equal to 1.0, but rather the wall simply develops a finite, permanent displacement relative to the base ground. Further, Richards and Elms demonstrated that this relative displacement is calculable. On this basis, they developed a design model fundamentally different than the previous pseudo-static (seismic coefficient) models.

Using the Richards-Elms model, the design engineer initially chooses an allowable permanent (irreversible) displacement for the wall, uses it to compute a "rational" (27) design acceleration (seismic coefficient), and then estimates the wall weight required for the specified condition.

Richards-Elms model is based on the analogy of sliding block which was originally introduced and used by Newmark (18) in analysis of earthquake induced displacements in embankment dams.

Whitman (27) presented an illustration of the sliding block analogy applied to the seismic behavior of gravity retaining walls, as follows. The forces acting on a gravity wall during an earthquake for the condition of active limiting equilibrium are shown in Figure 3. At the moment the earthquake induced acceleration acting upon the failure wedge and the wall (Figure 1) exceeds the limiting value of Ng, the wall and the failure wedge are unable to follow the base ground motion and a slip occurs along the base of the wall as well as the failure plane through the backfill. The limiting acceleration coefficient N is, by definition, equal to k_h in Eq. 1 and 2. The slip will stop when the base ground acceleration falls below Ng. The permanent relative displacement as a result of a single slip will be rather small. With a base ground motion, having a number of peak accelerations, there will be a number of intervals of slip, each followed by intervals during which the wall moves together with the base ground (i.e., they both have the same velocity). This progressive behavior is illustrated in Figure 4.

Referring to Figure 3, and disregarding any tilting and vertical acceleration of the wall and the failure wedge, the value of the limiting acceleration Ng, may be derived as follows (27). Applying equations for force equilibrium (Figure 3):

$$F = W_w + P_{AE}\ (N) \sin \delta \qquad (8)$$

$$W_w N + P_{AE} (N) \cos \delta = F \tan \phi_b \tag{9}$$

Assuming $\delta = 0$, and using Simplified Seed model (Eq. 3):

$$P_{AE} = 1/2 \gamma H^2 (K_A + 3/4 N) \tag{10}$$

Introducing a safety factor (F.S.) against sliding under static loading,

$$F.S. = \frac{W_w \tan \phi_b}{1/2 \gamma H^2 K_a} \tag{11}$$

and substituting Eqs. 11, 10 and 8 into Eq. 9:

$$N = \frac{F.S. - 1.0}{(3/4 K_a + \frac{F.S.}{\tan \phi_b})} \tag{12}$$

Having established the limiting acceleration Ng for a gravity retaining wall-backfill system, cumulative relative displacement may be estimated for a given earthquake record (Figure 4). Newmark (18) performed such analyses for four different earthquake records. For comparison of resulting displacements, Newmark (18) normalized the earthquake records in each case to a maximum acceleration of 0.5g and a maximum velocity of 30 in. per sec. Franklin and Chang (8) extended Newmark's work by analyzing 169 horizontal, 10 vertical accellerograms and several synthetic records, also normalized to Newmark's scale of 0.5g and 30 in.per sec. Upper bound envelope curves of permanent displacements analyzed by Franklin and Chang (8) are depicted in Figure 5. Upper bound envelope curves proposed by Newmark (18) are also incorporated in Figure 5.

Based on Figure 5, Richards and Elms (21, 4) proposed that for standardized maximum displacements in the medium to low range, a suitable approximation is given by the expression:

$$d \text{ (inch)} = 0.087 \frac{V^2}{Ag} \left(\frac{N}{A}\right)^{-4} \tag{13}$$

where,

d = cumulative relative displacement of a wall subjected to an earthquake record whose maximum horizontal acceleration coefficient is A and maximum horizontal velocity is V (in. per sec.),

N = the limiting acceleration coefficient (equal to k_h in Eq. 1, 2).

EFFECT OF WALL INERTIA

A key feature in the Richards-Elms analysis is the consideration of the effect of wall inertia. The free-body diagram of a retaining wall at seismic active limiting equilibrium condition is shown in Figure 6.

From force equilibrium:

$$F = (1 - k_v)\ W_w + P_{AE} \sin(\delta + \beta) \tag{14}$$

$$T = P_{AE} \cos(\delta + \beta) + k_h\ W_w \tag{15}$$

At incipient sliding,

$$T = F \tan \phi_b \tag{16}$$

Substituting from Eqs. 16 and 14 into Eq. 15,

$$P_{AE}\ [\cos(\delta + \beta) - \sin(\delta + \beta) \tan \phi_b] = W_w\ [(1 - k_v) \tan \phi_b - k_h] \tag{17}$$

Recalling from Eqs. 1,2;

$$P_{AE} = 1/2\ \gamma\ H^2\ (1 - k_v)\ K_{AE}, \text{ and } \tan \theta = \frac{k_h}{1 - k_v}$$

Eq. 17 may be re-written for W_w:

$$W_w = \frac{1/2\ \gamma\ H^2\ [\cos(\delta + \beta) - \sin(\delta + \beta) \tan \phi_b]}{\tan \phi_b - \tan \theta}\ K_{AE} \tag{18}$$

or as,

$$W_w = \frac{[\cos(\delta + \beta) - \sin(\delta + \beta) \tan \phi_b]}{(1 - k_v)\ (\tan \phi_b - \tan \theta)}\ P_{AE} \tag{19}$$

Eq. 18 or 19 is used to compute the required wall weight for the condition of F.S. = 1.0 against sliding during ground shaking.

To investigate the additional wall weight required to resist the wall inertia effect in comparison to the total design weight, Richards and Elms (21,4) expressed Eq. 19 in the form:

$$W_w = C_{IE} \; P_{AE} \tag{20}$$

where,

$$C_{IE} = \frac{\cos(\delta + \beta) - \sin(\delta + \beta) \tan \phi_b}{(1 - k_v)(\tan \phi_b - \tan \theta)} \tag{21}$$

Further, they introduced two factors; a soil thrust factor, F_T and a wall inertia factor, F_I by normalizing the dynamic effect of the soil failure wedge and the wall with respective static values;

$$F_T = \frac{K_{AE}(1 - k_v)}{K_A} \tag{22}$$

$$F_I = \frac{C_{IE}}{C_I} \tag{23}$$

where,

$$C_I = \frac{\cos(\delta + \beta) - \sin(\delta + \beta) \tan \phi_b}{\tan \phi_b} \tag{24}$$

Richards and Elms (21,4) demonstrated that magnitudes of F_T and F_I are of the same magnitude in the range of k_h = 0 to 0.5, and therefore the wall inertia force is as important as the dynamic soil thrust. Thus, the wall inertia effect cannot be ignored in the seismic design of gravity retaining walls.

VI. SEISMIC DESIGN OF GRAVITY RETAINING WALLS FOR LIMITING DISPLACEMENT

Based on the background presented in previous sections, Richards-Elms procedure for seismic design of gravity retaining walls includes the following steps:

1. For the gravity wall under consideration, select an acceptable maximum displacement, d, relative to the base ground.

2. Recalling,

$$d \text{ (inch)} = 0.087 \; \frac{V^2}{Ag} \left(\frac{N}{A}\right)^{-4} \tag{13}$$

and substituting,

$N = k_h$

$A = A_a$ (24)

$V(\text{in./sec.}) = 30\ A_v$ (24, p.301)

$g = 386.4$ in./sec.^{2}

where A_a and A_v, both dimensionless parameters, are defined as "effective peak acceleration coefficient" and "velocity-related acceleration coefficient" respectively (24). Eq. 13 may be inverted as:

$$k_h = A_a \left[\frac{0.2\ A_v^2}{A_a\ d\ (\text{in.})} \right]^{1/4} \tag{25}$$

where $(k_h \cdot g)$ is the cut-off acceleration for slipping corresponding to d (in.) and the regional seismicity represented by A_a, A_v. Eq. 25 provides a rational way of determining k_h.

3. Incorporate k_h in Eq. 2 to compute corresponding K_{AE}. Incorporate K_{AE} in Eq. 18 to estimate the required wall weight, W_w.

4. Apply a suitable factor of safety F.S. to W_w.

In their original work, Richards and Elms (21) recommended a safety factor of 1.5 to be used for the estimated wall weight. However, recently Elms (6) has noted that small scale tests conducted at the University of Canterbury, New Zealand (13, 14) have confirmed the reasonableness of the original Richards-Elms assumptions such that one can have more confidence in the analysis, and thus a lower safety factor (i.e., 1.2 to 1.3) would be appropriate.

5. In proportioning the wall geometry, make certain that it would "fail" by sliding rather than by tilting. In order that the wall would slide rather than tilt (overturn) Elms and Richards (4) showed that the location of the resultant of forces acting on the base of the wall, measured from the inner toe of the wall, x_o (Figure 7) should be at least equal to:

$$X_o = \frac{h\ [\cos(\beta + \delta) + \tan\beta \sin(\beta + \delta) + C_{IE} F.S[k_h \bar{Y}] + (1 - k_v)\ \bar{X}]}{\sin(\beta + \delta) + (1 - k_v)\ C_{IE}\ F.S} \tag{26}$$

where,

h = height of the resultant soil force from the base of wall (may be taken as h = H/2)

$\bar{x}$, $\bar{y}$ = coordinates of wall center of gravity

Whitman (27) used a different approach to determine the cut-off acceleration (coefficient) for sliding, $N = k_h$, from Eq. 13 or other expressions relating A, V and N proposed by Newmark (18)(Figure 5). In order to relate peak velocity, V, to peak acceleration coefficient, A, Whitman (27) suggested two ratios: V/A= 1250 mm/sec.(soft soil) and V/A=750 mm/sec., (very firm soil/rock) which represent the two "ends" of the typical range for this rate.

VII. SAMPLE DESIGN PROBLEM OF GRAVITY RETAINING WALL

A sample gravity wall design problem was worked out to develop a comparative picture of the required wall weights estimated by various procedures reviewed in the study. All analyses were conducted for the condition of "failure" induced by sliding. Seismic parameters proposed for Massachusetts by the Code (25) and the ATC 3-06 (24) were incorporated in respective analyses. The design parameters assigned for the sample problem were as follows:

H (height of gravity retaining wall) = 16 ft.

ϕ (angle of internal friction for backfill) = 33°

δ (angle of friction along wall-backfill interface) = 16.5°

ϕ_b (angle of friction along wall-base ground interface) = 33°

γ_s (total unit weight of unsaturated backfill) = 100 pcf.

γ_c (unit weight of concrete/masonry) = 150 pcf.

β (angle of inclination of wall-backfill interface measured from vertical) = 0°

i (slope angle of backfill surface measured from horizontal) = 0°

d (selected permanent displacement in Richards-Elms analysis) = 0.5 in.

First, a static analysis was made to form a reference base. A static safety factor of 1.5 against sliding was adopted in analysis. It was estimated that a wall weight of 6,605 lb/ft. would be required.

Second, the sample problem was analyzed for earthquake condition in accordance with the Code (25), Section 716.6.10. As stated previously the Code makes reference neither to wall inertia component nor to safety factor(s) to be incorporated in design analysis. Herein, the wall inertia effect was taken into account, and a static safety factor of 1.5 was used against sliding. Furthermore, load factors of 1.15 and 1.0 were adopted for wall inertia effect and dynamic soil thrust, respectively. It was estimated that a wall weight of 10,640 lb/ft. would be required.

Third, the sample problem was analyzed following the Richards-Elms limiting displacement procedure. As recommended by Richards and Elms (21), ATC 3-06 (24) was used to obtain the representative seismic parameters A_a and A_v proposed for Massachusetts: $A_a = 0.10$ and $A_v = 0.10$. A limiting displacement of 0.5 in. was chosen. Corresponding to d = 0.5 in., the limiting acceleration coefficient was computed to be N = 0.045 (Eq. 25). For an overall (i.e., static and dynamic) factor of safety of 1.0, the required weight of the wall was estimated to be 4,830 lb/ft from Eq. 18. A conservative overall safety factor of 1.5 would require a wall weight of 7,250 lb/ft. On the other hand a more realistic safety factor, such as 1.3 (6) would require a wall weight of 6,280 lb/ft. A somewhat more elaborate analysis, considering a static factor of safety of 1.5 and a load factor of 1.15 for dynamic component disclosed that a wall weight of 7,000 lb/ft. would be required.

Fourth, Richards-Elms analysis was conducted, following the approach used by Whitman (27) to obtain the limiting acceleration for sliding (Eq.13). With the "end" values of V/A = 1250 mm/sec. and V/A = 750 mm/sec., limiting acceleration coefficients N =0.072 and 0.056 were computed, respectively. Using these values, it was estimated by Eq. 18 that wall weights of 5,330 lb/ft. and 5,020 lb/ft. would be required respectively for a safety factor of 1.0. Considering a safety factor of 1.3 would increase these values to 6,930 lb/ft. and 6,520 lb/ft., respectively.

Computed design weights for the sample gravity wall by different procedures are summarized in Table I.

VIII. SOME FINAL REMARKS

1. The results from design analyses of the sample problem given in Table I suggest that a gravity retaining wall which is free to experience a "reasonable" permanent displacement due to ground shaking may be considerably lighter than that designed in accordance with the Code (25), Section 716.6.10 with the wall inertia included. Elms and Martin (5) recommended that acceleration coefficient, k_h, in Mononobe-Okabe model (Eqs. 1, 2) be taken as $0.5A_a$ (i.e., in place of A_a) for the design of gravity retaining walls, provided that an allowance is made for an outward displacement of 10 A_a (inch) to occur.

2. The results (Table I) suggest that if a gravity retaining wall is designed for static loading conditions with a safety factor of 1.5 against sliding, the wall is almost capable of supporting seismic loading as well (i.e., for Massachusetts seismicity), provided that a reasonable permanent displacement of the wall is allowed.

3. The Code (25) does not make a distinction between rigid vs. flexible and yielding vs. nonyielding retaining walls. As a matter of fact, if a retaining wall is fully restrained against movements (i.e., it is rigid and nonyielding), seismically induced soil thrust and wall inertia effect would necessitate considerably heavier walls to meet design requirements. Simplified elastic solutions presented by Wood (28, 29) indicate that for rigid-nonyielding retaining walls, dynamic thrust could be twice that given by the Mononobe-Okabe model.

4. Possible amplification of the ground shaking through wall backfill may also increase dynamic thrust. Such amplification effects are quite complex for analysis and are excluded in the design of ordinary retaining walls.

5. Richards-Elms (21, 4) analysis for design of gravity retaining walls uses a highly simplified model which excludes several relevant factors such as vertical motion of the backfill, vertical acceleration of the base ground and tilting of the wall during seismic activity (27). Zarrabi (30) refined the Richards-Elms model, considering the wall and soil failure wedge as separate components and including the change in the active seismic lateral thrust during wall movement. Vertical base ground accelerations were also accounted for. Reanalysis of Lai's (13) model wall test data

by Jacobson (12) disclosed that Zarrabi model gave a somewhat better description of the seismic behavior of the wall.

6. Aitken's (1) recent shaking tests carried out on a model gravity retaining wall designed to fail by sliding have shown that:

i. If the sand behind a gravity retaining wall is compacted at above its critical density (void ratio), then the initial movement of the wall will require higher horizontal ground accelerations than will subsequent movements.

ii. Once a well developed failure surface (wedge) was formed, the assumptions used in the Richards-Elms model appear to be essentially correct. This includes the basic assumption that once the threshhold acceleration is reached the wall will continue to move at that acceleration until the ground and wall velocities match.

iii. Within the accuracy imposed by limitations on the knowledge of soil properties along the failure surface, the Richards-Elms (21, 4) model provides a gcod prediction of wall displacement, and further refinement of the model is not justified for gravity walls designed to "fail" by sliding.

7. The seismic behavior of overturning (rotating) gravity walls is substantially more complicated than for the mode of sliding. The seismic behavior of overturning gravity walls are currently (1982) being investigated at the Massachusetts Institute of Technology and University of Canterbury. New findings should be incorporated in design analysis as they become available.

ACKNOWLEDGEMENT

The author owes much to Prof. R. V. Whitman, Massachusetts Institute of Technology; Prof. D.G. Elms, University of Canterbury, New Zealand; Dr. J. H. Wood, Central Laboratories, Ministry of Works and Development, New Zealand; Prof. R. Richards, Jr., University of Buffalo, New York; and Dr. I. M. Idriss, Woodward-Clyde Consultants, California, who have provided extensive material, guidance and inspiration to the author. Help received from the Earthquake Engineering Research Laboratory Library, University of California is highly appreciated. The author is grateful for the continuing support provided by Haley & Aldrich, Inc., Professional Development Program. Ms. L. Butterworth typed the manuscript and Ms. A. Welch drafted the figures.

TABLE I

Estimated Required Weights, W_w, for the Sample Gravity Retaining Wall Problem

Design Parameters: H = 16 ft., ϕ =33°, δ = 16.5°, ϕ_b= 33°,

γ_s (soil) = 100 p.c.f., γ(concrete) = 150 p.c.f.

Procedure/Condition	Estimated W_w (lb/ft)
1. Static: F.S. = 1.5	6,605
2. Mass Building Code (Section 716.6.10)	
F.S. (static) = 1.5, (dynamic) = 1.0,	
(wall inertia) = 1.15	10,640
3. Richards-Elms; ATC 3-06	
Limiting Displacement = 0.5 in.	
F.S. (static) = 1.5,(dynamic) = 1.5	7,250
F.S. (static) = 1.3, (dynamic) = 1.3	6,280
F.S. (static) = 1.5, (dynamic) = 1.15 (approximate)	7,000
4. Richards-Elms; Whitman (21)	
Limiting Displacement = 0.5 in.	
V/A = 750 mm/sec, F.S. (static) = 1.3, (dynamic) = 1.3	6,520
V/A= 1,250 mm/sec, F.S. (static) = 1.3, (dynamic) = 1.3	6,930

NOTE: The mode of failure is by sliding.

REFERENCES

(1) Aitken, G.H., "Seismic Response of Retaining Walls", Research Report 82-5, Department of Civil Engineering, University of Canterbury, New Zealand, March 1982.

(2) Clough, G.W. and Fragaszy, R.F., "A Study of Earth Loadings on Floodway Retaining Structures in the 1971 San Fernando Earthquake", Proceedings, Sixth World Conference on Earthquake Engineering, New Delhi, Vol.7, 1977, pp 7-37 to 7-42.

(3) Ellison, B.K., "Earthquake Damage to Roads and Bridges-Madang Earthquake, Republic of Papua and New Guinea, November 1970", Bulletin of New Zealand Society of Earthquake Engineering, Vol.4, November 1971, pp. 243-257.

(4) Elms, D.G. and Richards, R. Jr., "Seismic Design of Gravity Retaining Walls", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 12, No. 2, June 1979, pp. 114-121.

(5) Elms, D.G. and Martin, G. R., "Factors Involved in the Seismic Design of Bridge Abutments", Workshop on Research Needs Related to Bridges, Applied Technology Council, Palo Alto, California, 1979.

(6) Elms, D.G., Personal Correspondence, July 1982.

(7) Evans, G.L., "The Behavior of Bridges Under Earthquakes", Proceedings, New Zealand Roading Symposium, Victoria University, New Zealand, Vol. 2, 1971, pp 664-684.

(8) Franklin, A.G., and Chang, F.K., "Earthquake Resistance of Earth and Rock-Fill Dams", Report 5: Permanent Displacements of Earth Embankments by Newmark Sliding Block Analysis", Miscellaneous Paper S-71-17, Soils and Pavement Laboratory, U.S.Army Engineer Waterways Experiment Station, Vicksburg, Miss., Nov. 1977.

(9) Fung, G.G., LeBeau, R.J., Klein, E.D., Belvedere, J. and Goldsmith, A.F., "Field Investigation of Bridge Damages in the San Fernando Earthquake, Preliminary Report, State of California, Department of Public Works, Division of Highways, Bridge Department, 1972.

(10) Idriss, I.M. and Moriwaki, Y., "Lateral Earth Pressures Due to Earthquake Ground Motion", Lecture notes prepared for presentation at the session on Geotechnical Problems, EERI Annual Meeting, San Francisco, February 1982.

(11) Jacobsen, L.S., Civil and Structural Design, Design of TVA Projects, Technical Report No. 24, Vol. 1, U.S.Tennessee Valley Authority, 1952.

(12) Jacobson, P.N. , "Translational Behavior of Gravity Retaining Walls During Earthquakes", M.E. Report, Department of Civil Engineering, University of Canterbury, New Zealand, 1980.

(13) Lai Cho Shim, "Behavior of Retaining Walls Under Seismic Loading", M.E. Report Department of Civil Engineering, University of Canterbury, New Zealand, 1979.

(14) Lai Cho Shim and Berrill, J.B., "Shaking Table Tests on a Model Retaining Wall", Bulletin of the New Zealand Society of Earthquake Engineering, Vol. 12, No. 2, June, 1979, pp. 122-126.

(15) Matthewson, M.B., Wood, J.H., and Berrill, J.B., "Seismic Design of Bridges, Section 9, Earth Retaining Structures", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 13, No. 3, September 1980.

(16) Matsuo, H., "Experimental Study on the Distribution of Earth Pressure Acting on a Vertical Wall During Earthquakes", Journal of the Japanese Society of Civil Engineers, Vol. 27, No. 2, 1941.

(17) Mononobe, N., "Earthquake-Proof Construction of Masonry Dams", Proceedings, World Engineering Conference, Vol. 9, 1929.

(18) Newmark, N.M., "Effects of Earthquakes on Dams and Embankments", Geotechnique, London, England, Vol. 15, No. 2, 1965, pp. 139-160.

(19) Okabe, S., "General Theory of Earth Pressure", Journal of the Japanese Society of Civil Engineers, Tokyo, Japan, Vol. 12, No. 1, 1926.

(20) Prakash, S. and Basavanna, B.M., "Earth Pressure Distribution Behind Retaining Wall During Earthquake", Proceedings, Fourth World Conference on Earthquake Engineering, Santiago, Chile, 1969.

(21) Richards, R.Jr., and Elms, D.G., "Seismic Behavior of Gravity Retaining Walls", Journal of the Geotechnical Engineering Division, ASCE, Vol. 105, GT 4, April 1979, pp. 449-464.

(22) Seed, H.B. and Whitman, R.V., "Design of Earth Retaining Structures for Dynamic Loads", Lateral Stresses in the Ground and Design of Earth Retaining Structures, ASCE Specialty Conference, Cornell University, New York, 1970, pp. 103-147.

(23) Soydemir, C. and LeCount, P.L., "Massachusetts Building Code on Liquefaction: A Design Engineer's Overview", Proceedings, Earthquake and Earthquake Engineering, Eastern United States, Knoxville, Tennessee, Vol. 2, September 1982, pp. 665-681.

(24) "Tentative Provisions for the Development of Seismic Regulation for Buildings", Publication ATC 3-06, Applied Technology Council, Palo Alto, California, June 1978.

(25) The Commonwealth of Massachusetts, State Building Code, Third Edition, 1979.

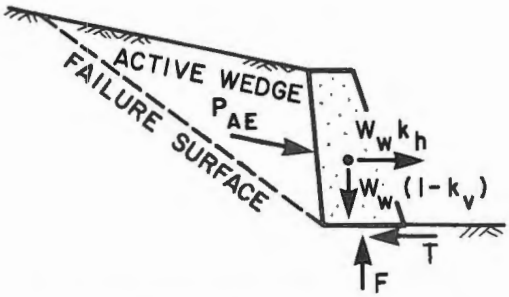
(26) Whitman, R.V., "Comments on Response of Gravity Walls to Earthquake Ground Motion" Proceedings, Earthquake Engineering and Soil Dynamics, ASCE Geotechnical Engineering Division, Specialty Conference, Panel Discussion, Pasadena, California, Vol. 3, June 1978, pp. 1442-1446.

(27) Whitman, R.V., "Dynamic Behavior of Soils and Its Application to Civil Engineering Projects", General Report - Main Session II, Proceedings of the Sixth Panamerican Conference on Soil Mechanics and Foundation Engineering, Lima, Peru, Vol. 1, December 1979, pp. 59-105.

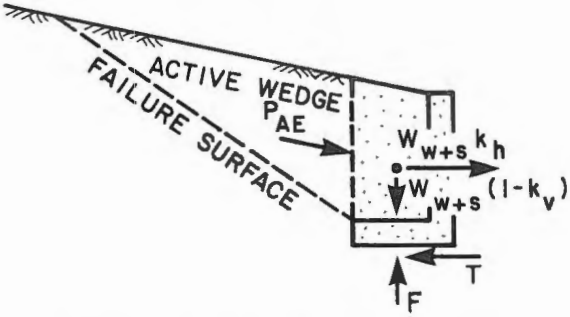
(28) Wood, J.H., "Earthquake Induced Soil Pressures on Structures", Report No. EERL 73-05, Earthquake Engineering Research Lab., California Institute of Technology, Pasadena, California, 1973.

(29) Wood, J.H., "Earthquake Induced Pressures on a Rigid Wall Structure", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 8, No. 3, September 1975, pp. 175-186.

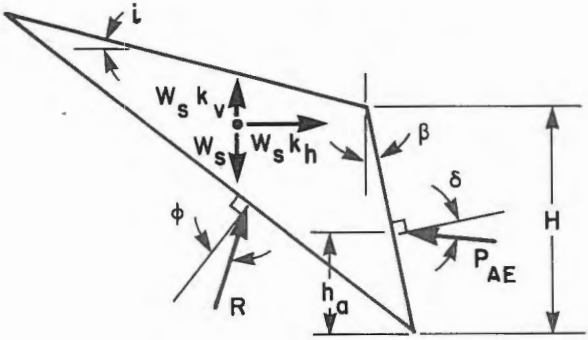
(30) Zarrabi-Kashani, K., "Sliding of Gravity Retaining Walls During Earthquakes Considering Vertical Acceleration and Changing Inclination of Failure Surface", M.S. Thesis, Department of Civil Engineering, Massachusetts Institute of Technology, 1979.



a. GRAVITY RETAINING WALL



b. CANTILEVER RETAINING WALL



c. ACTIVE SOIL WEDGE

Figure 1. FORCE DIAGRAMS FOR ACTIVE LIMITING EQUILIBRIUM CONDITION

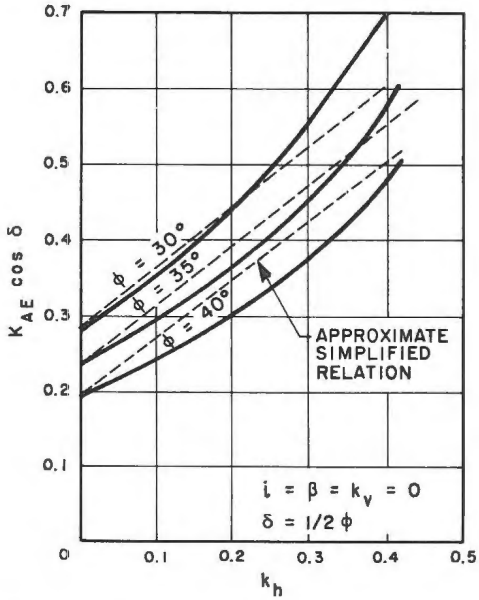
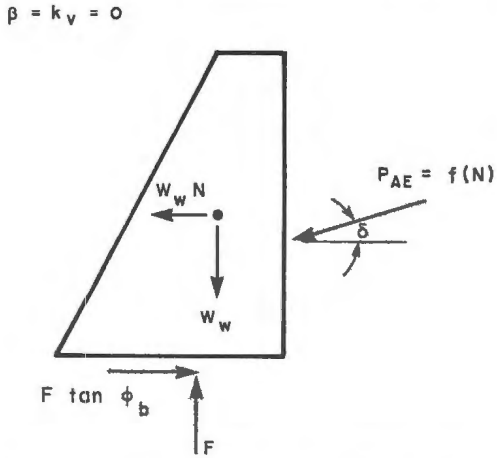


Figure 2. INFLUENCE OF ANGLE OF FRICTION OF BACKFILL ON DYNAMIC LATERAL PRESSURES DETERMINED BY MONONOBE-OKABE ANALYSIS (AFTER WHITMAN, 26)



N = LIMITING ACCELERATION COEFFICIENT (EQUIVALENT TO k_h IN MONONOBE-OKABE ANALYSIS)

ϕ_b = ANGLE OF FRICTION ALONG THE WALL-BASE GROUND INTERFACE

Figure 3. FORCES ON GRAVITY RETAINING WALL FOR LIMITING ACCELERATION (AFTER WHITMAN, 27)

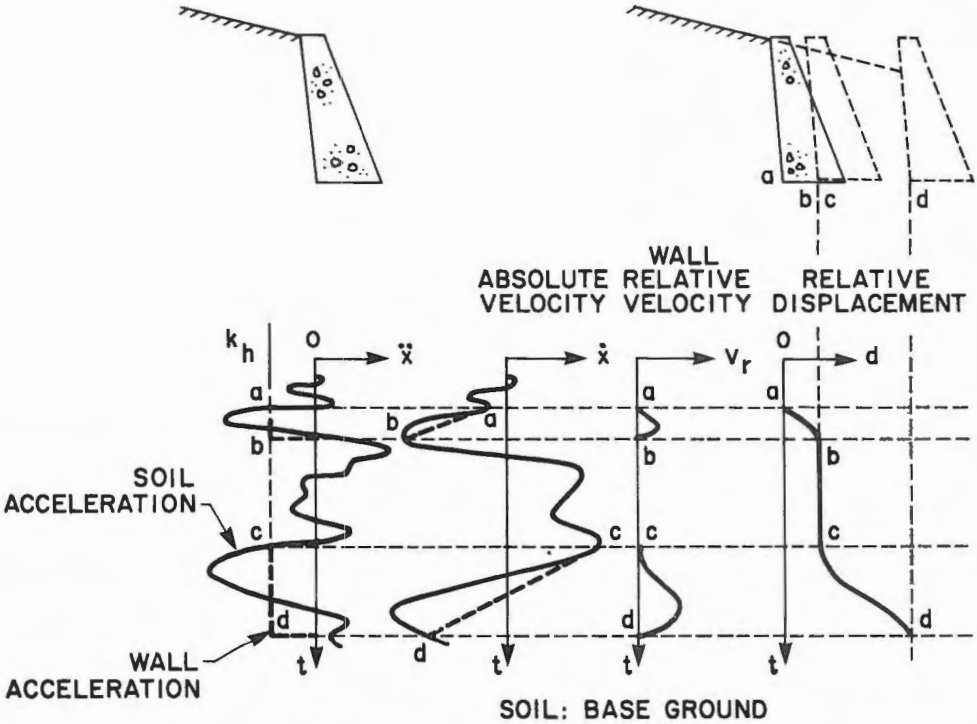


Figure 4. RELATION BETWEEN RELATIVE DISPLACEMENT AND ACCELERATION, VELOCITY TIME HISTORIES OF SOIL AND WALL (AFTER RICHARDS AND ELMS, 21)

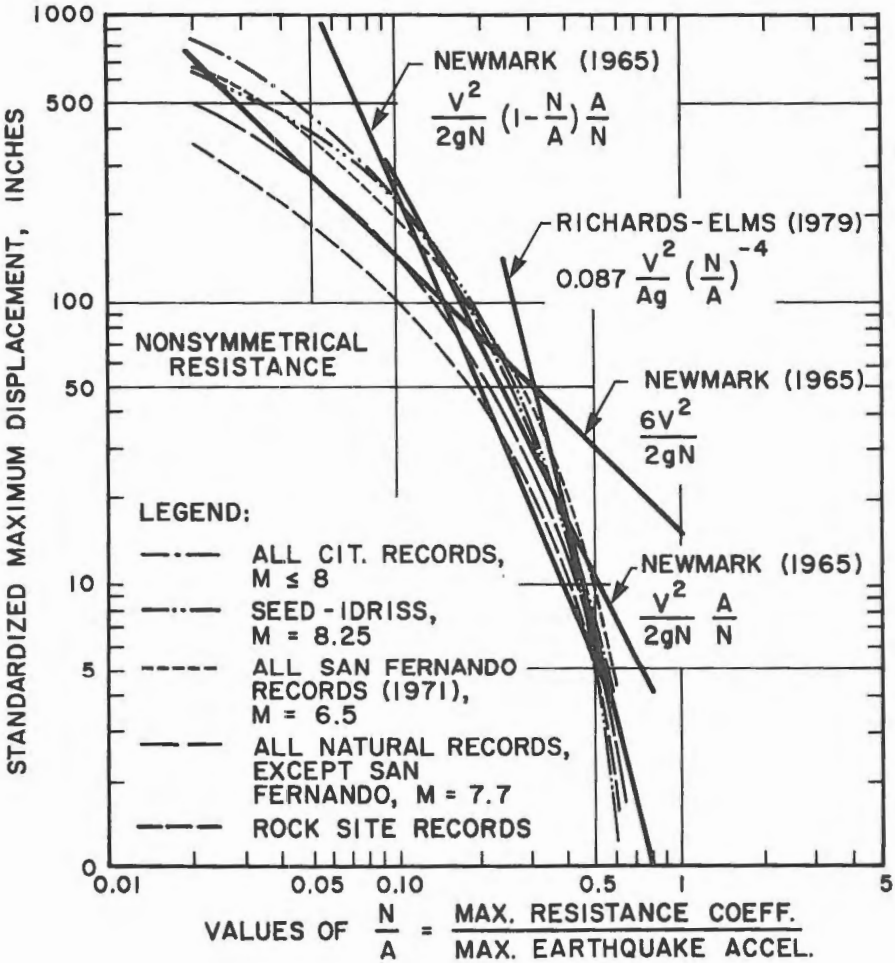


Figure 5. UPPER BOUND ENVELOPES (AFTER NEWMARK, 18, FRANKLIN AND CHANG, 8, RICHARD AND ELMS, 21)

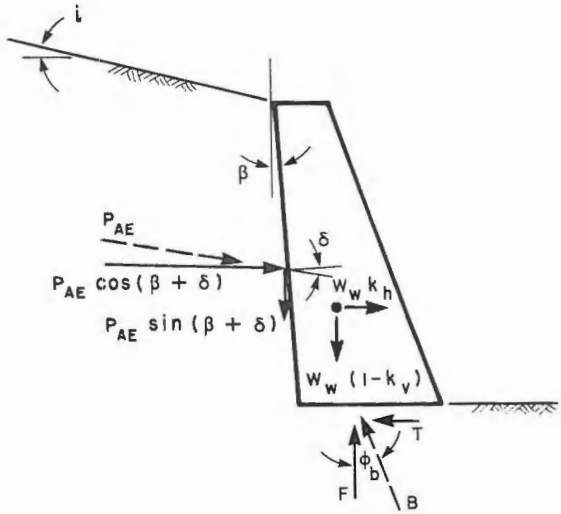


Figure 6. FORCES ON GRAVITY RETAINING WALL AT SEISMIC-ACTIVE LIMITING EQUILIBRIUM

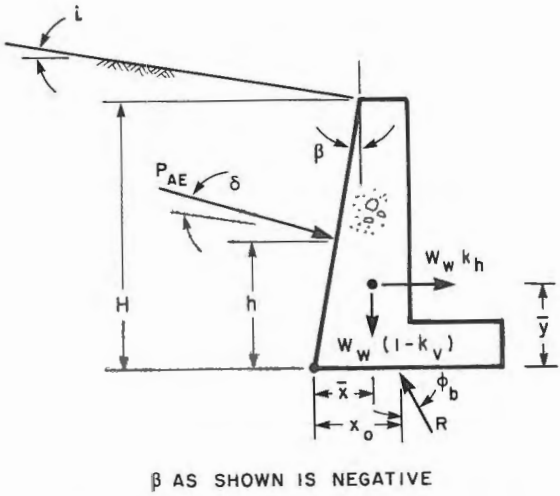


Figure 7. FREE BODY DIAGRAM OF GRAVITY RETAINING WALL AT ACTIVE LIMITING EQUILIBRIUM (AFTER ELMS AND RICHARDS, 4)