SEISMIC TESTING OF REINFORCED EARTH WALLS

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INTRODUCTION

The term reinforced earth implies the use of bars or ties that are embedded in soil to provide additional load carrying strength. The bonding between the soil and the ties is developed through friction. Thus, cohesionless soil is preferred so that the frictional bonding is not affected by internal pore pressures. For retaining wall construction, an outer skin is also required to prevent the soil from raveling away at the face. Reinforced earth walls constructed to date have mainly used thin galvanized steel strips for the ties, and have used such materials as plastic, various fabrics, lightweight steel panels, and precast concrete blocks for the outer skin.

A satisfactory design for a reinforced earth wall must insure that applied loads do not lead to a tensile failure in the reinforcement and that the frictional bond is sufficient to carry the load transfer between the soil and the reinforcement. These criteria apply both to static loads as well as possible dynamic loads for structures in seismically active areas.

A previous study by two of the writers (2) suggested an empirical procedure for the seismic design of reinforced earth walls based largely on the results

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of shaking table tests on small-scale model walls using harmonic horizontal input base accelerations \(X\). A lateral earth pressure design envelope was proposed as shown in Fig. 1 for both static loading and for random dynamic loading. The empirical procedure was based on adapting spectral analysis techniques for

\[ A_{\text{DES}} = \Gamma_1 S_{A1} + \Gamma_2 S_{A2} \]  

(1)

The parameters \(\Gamma_1\) and \(\Gamma_2\) are the corresponding modal participation factors. Only the first two modes were used because the spectral-modal participation factor method is conservative, and because the effect of higher modes on the value of \(A_{\text{DES}}\) is negligible compared with the effect of the first two modes.

The modal participation factors were evaluated from a simplified lumped mass model to give values: \(\Gamma_1 = 1.25\) and \(\Gamma_2 = 0.5\). To determine the spectral accelerations it is necessary to estimate the first two natural frequencies of the reinforced earth wall and the respective modal damping at the cyclic strain level corresponding to the level of seismic excitation of the wall and soil backfill during the design earthquake. The design force in each tie is determined by summing up the total lateral earth pressure at each tie location as computed from Fig. 1, and multiplying by the tributary area for the particular tie. Conventional static design equations are then used to design the tie to resist the pseudostatic load (combined static plus seismic).

The Richardson-Lee laboratory tests demonstrated that if the ties broke during seismic loading, the wall would suddenly collapse in a complete and sudden failure. However, if the ties did not break, any undesirable behavior that might develop would be in the form of continual progressive deformations with time as the shaking continued. For this reason it was suggested that greater conservatism should be used in design for tie-breakage than for tie-pullout to ensure that if the wall were to become overstressed, the result would be in the form of a finite deformation rather than a major collapse.

To supplement the empirical data from laboratory model tests and to check the design procedure on a prototype structure, a full-scale test wall was constructed and tested during the summer of 1974 at the University of California at Los Angeles (UCLA) Engineering Field Station in Saugus, Calif., located some 30 miles (48 km) north of the UCLA campus. This test wall provided data on the static and seismic response, demonstrated the satisfactory performance of a conventional static design of a reinforced earth wall for moderately intense seismic loading, and illustrated the simplicity of construction for this new type of earth retaining structure.

A photograph of the completed test wall is shown in Fig. 2. The entire wall was constructed during a 10-week period of calendar time by the three graduate students (first three coauthors) standing in front of the wall, using the earth moving equipment parked on the top. In addition to performing the actual construction operations the three students also installed, calibrated, and checked all of the instrumentation that was buried in the backfill or mounted on the face of the wall. The concrete face panels and the steel reinforcing ties were donated by the Reinforced Earth Company and were delivered to the site by their forces. In fact, preliminary negotiations with precast concrete and steel organizations had shown that these materials could have been readily obtained and delivered to the site by local suppliers. This total accomplishment by the three graduate students surely represents a simplicity of construction not readily
found with other types of earth retaining walls.

A sketch showing the design dimensions of the UCLA test wall is presented in Fig. 3. The wall was 20 ft (6.1 m) high and 54 ft-113 ft (165 m-43.5 m) long. The face was composed of 79 precast concrete panels, weighing a maximum of 2,600 lb (1,200 kg) each. The reinforcing consisted of 302 galvanized steel ties 80 mm wide x 3 mm thick and 16 ft (4.9 m) long having a combined total weight of less than 3 tons. The ties were spaced 2.5 ft (0.76 m) horizontally and 2.5 ft (0.76 m) vertically making a tributary area of wall per tie of $T_A = 6.25 \text{ sq ft} (0.58 \text{ m}^2)$, except at the lowest two levels where the vertical tie spacing and the tributary areas were half of these values.

The design of the wall was done by the Reinforced Earth Company, who donated the reinforcing materials for the project. It followed their standard design procedure for static loading, with no special allowance for an additional seismic loading effect. This design was checked and found to be more than adequate to meet the static loading design procedure suggested by previous UCLA studies (2,3). Thus, the performance of this test wall is illustrative of the performance of a standard design Reinforced Earth wall.

The total volume of backfill was about 6,500 cu yd (5,500 m$^3$) of which 1,500 cu yd (1,150 m$^3$) were contained within the zone of reinforcing. The earth moving equipment used to construct the wall consisted of the following: one Massey-Ferguson 3366, 1.62-cu yd (1.25 m$^3$) front end loader; one Massey-Ferguson 200 bulldozer; and two 6-cu yd (4.5-m$^3$) dump trucks. Of the 10 weeks total construction time, nearly half of the man hours were spent in placing, calibrating, and monitoring the instrumentation equipment.

The total cost for construction and testing of the wall was about $44,000.

FIG. 4.—Buried Instrumentation

FIG. 5.—Static Tie Forces Measured During Construction of UCLA Test Wall

This figure does not include time and money spent during the design stages prior to construction or to the subsequent reduction of the data and preparation of reports.

TEST WALL INSTRUMENTATION

Many instruments were used within the backfill and on the wall to measure its performance and response under both static and dynamic loading conditions. A sketch showing the location of the buried instruments is presented in Fig. 4. Most of the instruments were buried within the backfill and were thus inaccessible for recalibration or repair during the testing. However, most of the accelerometers were mounted on the surface of the wall or backfill. These were recovered and recalibrated following the dynamic tests. About 10% of the instruments were found to be inoperative at the end of construction and about 20% of the instruments were damaged during the high level blasting tests.

Most of the transducers, except the accelerometers, were monitored statically after each construction lift and after each blasting test. During the blasting tests, time-history records were taken for 12 accelerometers and eight tie force gages. All time-history recordings were made simultaneously on Sanborn strip chart recorders and on a Kinematics DDS 1103 digital tape recorder. Thus, the dynamic test records were immediately available for visual inspection in the field and were later accessible for analysis with a computer.

DESCRIPTION OF SITE AND BACKFILL MATERIAL

The test wall was constructed at the UCLA field station near the town of Saugus, Calif., at the edge of a small alluvial plain in foothills of the San Gabriel mountains. To minimize the amount of backfill required, the wall was constructed close to a small steeply sloping hill. The backfill material consisted of dusty sandy gravel excavated from the bed of a dry stream adjacent to the site. The $D_60$ and $D_80$ grain sizes were 1.0 mm and 0.15 mm, respectively. No water was used during placement of the backfill and no special compaction procedure was employed other than driving the trucks over the fill during the construction operations. Field densities were taken for each 1.5-ft (0.46-m) high construction lift. The average field dry density was about 125pcf (2,000 kg/m$^3$) at a water content of about 1%. This corresponded to about 85% relative compaction based on the modified Proctor compaction test or about 65% relative density. The thickness of construction lifts varied between about 1 ft (0.3 m) and 2.5 ft. The tie reinforcing steel was always placed on the top of a construction lift, although not necessarily on the top of each consecutive lift.

STATIC MEASUREMENTS

Static tie forces were measured at the end of each construction lift. These data are shown in Fig. 5. The data show steadily increasing tie forces for each successive construction lift with increasing height of fill.

In addition, theoretical force values $F$ are shown for the final full height of backfill, computed by the procedure implied in Fig. 1, $F = K_n \cdot yd \cdot T_A$. 

$K_n$ is the load factor for a typical earth retaining wall.
In this equation \( d \) is the depth below the fill surface and a value of \( K_s = 0.4 \) was taken which corresponds to the value computed from Jaky equation \( K_s = 1 - \sin \phi \) using the \( \phi = 38^\circ \) as measured in triaxial tests. Comparison of the measured data with the calculated values shows that at the completion of the wall, the final static tie forces were approximately equivalent to the theoretical tie forces computed for the assumed at-rest pressure conditions. The trend shown by these field data agrees with trends obtained from previous studies on small models (2,3) and on field walls (4). The decrease in \( F \) for the lowest lifts reflects the decrease in vertical tie spacing and tributary wall area \( T_s \) at these locations.

Static horizontal displacements were monitored at three vertical locations using a system of cables attached to the face of the wall and anchored well within the backfill. The measured wall movement from the time the fill was placed at the level of the gage until the end of construction when the fill was 18.1 ft (5.5 m) above the ground in front of the toe was as follows: 0.50 in. (13 mm) at the lowest gage at lift No. 5 (Fig. 5) situated 4.35 ft (1.3 m) above the toe; 0.34 in. (9 mm) measured at the uppermost gage lift No. 8, which was some 6.25 ft (1.9 m) below the top of the wall or 11.85 ft (3.6 m) above the ground in front of the toe. These lateral deformations correspond to an average outward rotational tilt of about 0.9% at the lowest gage and 0.25% near the top of the wall. Recall for comparison that about 0.5% outward movement is generally assumed to be required to develop the minimum active \( K_a \) earth pressure condition from an initial \( K_a \) condition. Thus, it might be expected that end-of-construction static lateral earth pressure behind most of the wall might be close to \( K_a \), whereas near the top the static lateral earth pressure would be greater than \( K_a \) and close to \( K_s \). This is more or less the trend of the static end-of-construction earth pressures measured for this test wall (Fig. 5).

Vertical settlements measured within the backfill during construction ranged between about 0.65% to 2.8% of the thickness of the fill below the settlement point. These fairly large vertical settlements reflect the relatively loose (\( D_c = 65\% \)) nature of the dry granular backfill material.

Static earth pressure measurements were made to determine the distribution of vertical pressure within the backfill at the elevation of the base of the wall and at various locations behind the face. The reason for measuring vertical earth pressures is that this is one of the key parameters in the Reinforced Earth Company design method which assumes a trapezoidal earth pressure below the reinforced backfill, much as would be found below the base of a conventional retaining wall. The data obtained from the UCLA test wall indicated a fairly uniform vertical pressure distribution equal to the overburden weight at all locations extending from near the wall to the far end of the reinforcing ties.

**FORCED VIBRATION TESTING**

The theoretical analysis governing forced vibration testing of a complex structure involves the assumption that the various modes of vibration are uncoupled. The governing equations are those that pertain to a single degree-of-freedom structure which may be found in most elementary textbooks on vibration (compare with Ref. 6) and will not be detailed herein. The maximum displacement, \( X_{max} \), due to a harmonic forced vibration can be expressed as

\[
X_{max} = A \omega^2 MF
\]

in which \( A \) is a constant that is a function of the stiffness of the oscillator and amplitude of the forcing function; \( \omega \) is the angular frequency of the harmonic forcing function; and \( MF = \beta \) is the magnification factor which is a function of the damping \( \beta \), the location of the forcing function, and the ratio of the forcing frequency, \( \omega \), to the fundamental frequency of the undamped system, \( \omega_y \).

Near resonance (\( \omega / \omega_y = 1.0 \)), the values of MF are large and at resonance increase they are inversely proportional to \( \beta \), for \( \beta \) less than about 20%. A value of damping may be calculated from the shape or "bandwidth" of the response curve near the resonant frequency.

Because accelerations are generally easier to monitor than dynamic displacements, the response measurements from forced vibration tests are usually obtained with accelerometers mounted at key locations on the structure being tested. The maximum displacement, \( X_{max} \), is computed from the maximum recorded acceleration \( \dot{X}_{max} \) at each frequency using the harmonic motion relationship

\[
X_{max} = \frac{\dot{X}_{max}}{\omega^2}
\]

A typical test consists of operating the forced vibration generator at a constant frequency, \( \omega \), and measuring the corresponding new value of \( \dot{X}_{max} \). This process is continued using small frequency steps from low values of \( \omega \) up to the limit of the vibration equipment. For convenience, one entire sweep of frequencies is performed using the same amount of eccentric rotating mass. Sometimes the procedure is repeated using a different amount of eccentric mass to produce a different range of dynamic forces and strains.

If the structure being tested is a multidegree-of-freedom system, then a test sweep covering a large range of frequencies will in general give a response curve having several peaks; one corresponding to each separate mode of vibration. If the modes are uncoupled, as assumed, the response of a structure at a particular modal resonant frequency is essentially the response of that only particular mode with perhaps a minor contribution from other modes. Then follows that the experimental forced vibration data for a multidegree-of-freedom structure can be analyzed separately by mode.

The maximum dynamic force produced by the vibrator increases with the square of the excitation frequency. Since the force increases, the corresponding dynamic deformations will also increase with the square of the vibrating frequency. However, when working with actual forced vibration data, it is desirable to normalize the measured accelerations to a constant amplitude forcing function by dividing them by \( \omega^2 \). The validity of this normalization with respect to theory assumes a linear system with stiffness and damping that are independent of the amount of deformation produced by different levels of dynamic force.

Systems involving soil, such as the reinforced earth test wall and backfill described herein, can be expected to have nonlinear strain-dependent stiffness and damping properties (5). This means that the stiffness or fundamental frequency and the damping will vary throughout the sweep of vibration frequencies used.
in a test according to different levels of strain produced by the different dynamic forces at the different frequencies. Nevertheless, to a first approximation, it seems reasonable to use the simple linear theory concept for analyzing the data from forced vibration tests on soil structures, but at the same time noting the absolute amount of deformation or strain corresponding to the resonant frequency and damping. This approach should be accurate for resonant frequency determinations, and should be satisfactory, although approximate for the damping calculated from the response curve, provided that the bandwidth between the two frequencies used in the damping equation is not large.

The forced vibration tests, and to a lesser extent the blasting tests, provided the basic data needed in order to estimate the first two modes of vibration of the test wall. The forced vibration tests were performed using either of two vibrators. Each vibrator employed counterrotating eccentric weights to produce a sinusoidal force proportional to both the rotating eccentric weights and to the square of the rotating frequency. The smaller of the two vibrators operated within a frequency range of 0 Hz-30 Hz and could produce a maximum force of 5,000 lb (2,300 kg) at 30 Hz. The larger vibrator operated within a frequency of 1 Hz-10 Hz and could produce a maximum force of 5,000 lb (2,300 kg) at 10 Hz. The vibrators were mounted on large concrete pads located either in front of the wall near the toe or alternatively on the top of the backfill near the wall. Only one vibrator was used at a time. The predominant direction of vibration was normal to the face of the wall, although a few special purpose tests were performed using a vertical and also transverse alignments.

The results of two forced vibration tests on the UCLA wall are shown in Fig. 6. The absolute values of the system responses depend on the level of applied dynamic force, which is a function of the weight of the rotating mass. However, since only the frequency locations of the peak responses were of interest in these tests, the measured responses for these tests were normalized for a constant eccentric mass by multiplying the measured accelerations by the ratio of the actual eccentric mass used in the respective tests to some arbitrary constant mass.

The forced vibration test data from Test T1R2, which used the large vibrator, define the first transverse mode at a frequency of 6.5 Hz. Data from Test T8R1, which used the small vibrator, defines several other modes, including the second and third horizontal translational modes, and some twisting or rocking modes. Only the translational modes were of interest in this study.

In general, different weights were used in different tests so that the entire program generated a large number of frequency response curves, such as shown in Fig. 6, from which natural frequencies and damping for each mode could be determined. Since a reinforced earth system is nonlinear with respect to the modulus and damping of the backfill soil, different values of soil properties were involved for different tests, depending upon the level of the forced excitation used.

To supplement the information obtained from the UCLA test wall similar but less extensive forced vibration tests were made during December 1974 on four standard reinforced earth walls belonging to other owners and built according to Reinforced Earth Co. specifications. These walls were located in other parts of the United States. Because of difficult access, only the small vibrator was used with these additional tests and it was mounted at the top of the backfill. Because of various limitations, the amount of testing done on the commercial walls was only sufficient to determine the frequencies of the first two modes of vibration and was not sufficient to determine the damping in any mode.

The frequencies and the damping for the first three transverse modes as determined from the forced vibration tests on the UCLA wall are listed in Fig. 6. A summary of the pertinent data and results for all walls tested in the program is presented in Table 1. These data involve only low levels of dynamic excitation. From the measured values of maximum acceleration and Eq. 3, the maximum shear strains in the soil behind the walls were estimated to be less than 0.001%.

In constructing a reinforced earth wall it is common practice to place a berm of fill on the outside at the toe of the wall so that in effect the lowest soil elements are buried at some distance below the ground surface. This arrangement is shown in Fig. 5 for the UCLA test wall. The effective height of the walls listed in Table 1 refers to the height of the skin and backfill extending above the soil berm at the toe at the time of the testing.

The resonant frequencies for the first two transverse modes, \( f_1 \) and \( f_2 \), in hertz, of each of the five walls tested are shown in Fig. 7 plotted versus the effective height of the wall, \( H \), in feet. Note that the data are fairly well defined by the empirical equations

\[
f_1 = \frac{125}{H}
\]

\[
f_2 = \frac{330}{H}
\]

As an ideal approximation, the reinforced backfill soil behind the wall might be considered to respond to dynamic excitation as if it were a layer of level ground above a rigid base. From wave propagation theory (1), the resonant frequency, \( f_n \), of the \( n \)th mode of vibration in an elastic solid layer of thickness, \( H \), above a rigid base is given by

\[
f_n = \frac{(2n - 1) V_s}{4H}
\]
TABLE 1.—Results of Forced Vibration Tests on Reinforced Earth Walls

<table>
<thead>
<tr>
<th>Location</th>
<th>Purpose</th>
<th>Effective height, in feet (meters)</th>
<th>Backfill material and properties.*</th>
<th>Natural Frequency, in hertz</th>
</tr>
</thead>
<tbody>
<tr>
<td>UCLA Test Wall,</td>
<td>Dynamic test wall</td>
<td>18.1 (5.5)</td>
<td>Sand, φ = 40°, V&lt;sub&gt;n&lt;/sub&gt; = 20</td>
<td>6.2 (6)</td>
</tr>
<tr>
<td>Saugus, Calif.</td>
<td>Sea wall and retaining</td>
<td>7.5 (2.3)</td>
<td>Sand, 40% clay, φ = 32°, V&lt;sub&gt;n&lt;/sub&gt; = 110</td>
<td>20.0 (6)</td>
</tr>
<tr>
<td>Brunswick, Ga.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fighting Creek Gap</td>
<td>Slide correction</td>
<td>12 (3.6)</td>
<td>Quarry stone</td>
<td>4.5 (14.0)</td>
</tr>
<tr>
<td>Gatlinburg, Tenn.</td>
<td></td>
<td>28 (8.5)</td>
<td>Sand, V&lt;sub&gt;n&lt;/sub&gt; = 115</td>
<td>4.5 (14.0)</td>
</tr>
<tr>
<td>140, Rockwood,</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tenn.</td>
<td>Slide correction</td>
<td>28 (8.5)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>180, Lovelock, Nev.</td>
<td>Double H-way</td>
<td>19 (5.8)</td>
<td>Sand, V&lt;sub&gt;n&lt;/sub&gt; = 115</td>
<td>4.5 (14.0)</td>
</tr>
<tr>
<td></td>
<td>bridge abut</td>
<td></td>
<td></td>
<td>5.0 (12.0)</td>
</tr>
</tbody>
</table>

*Given in pounds per cubic foot (grams per cubic centimeter).

FIG. 7.—Measured and Calculated First and Second Mode Natural Frequencies for Different Heights of Reinforced Earth Walls (1 ft = 0.305 m)

FIG. 8.—Visually Spectacular Explosive Test T8R1 on UCLA Wall

in which V<sub>n</sub> = the shear wave velocity within the elastic material. The shaded zones shown in Fig. 7 represent the theoretical frequencies calculated by Eq. 6 for V<sub>n</sub> in the range of 400 fps–600 fps (120 m/s–180 m/s).

For reference purposes it is noted that Seed and Idriss (5) have shown that the shear modulus, G, of a granular soil can be expressed by the equation

\[ G = 1,000 \times K_2 (\sigma_m)^{0.5} \]

in which \( \sigma_m \) = the mean normal stress, in pounds per square foot. It is also noted that the shear modulus \( G = \rho V_n^2 \), in which \( \rho \) = the mass density of the soil. The values of \( V_n = 400 \text{ fps} – 600 \text{ fps} \) (120 m/s–280 m/s) correspond to values of \( K_2 = 55 \), and a mean normal stress at a location about the mid-depth of the walls. The value \( K_2 = 55 \) corresponds to a sandy soil at \( D_r = 60\% \), at very low levels of dynamic excitation (5) which would be appropriate for the forced vibration testing. Thus, insofar as the theory is appropriate, there is good agreement between the experimental and theoretical values for natural frequencies of the first two modes of vibration at low strain levels for the five walls tested.

EXPLOSIVE VIBRATION TESTS

Because the forced vibration tests described previously were limited to rather low levels of dynamic excitation (shear strains less than 0.001%), because previous studies have shown that the dynamic ground response is highly nonlinear with respect to vibration strain levels, and to obtain data from random type dynamic loading, it was decided to use dynamite explosions to produce high levels of excitation at the UCLA test wall. It was also hoped that the irregular dynamic loading produced from multiple explosions would provide a better insight to the actual seismic response of this wall than was possible from the sinusoidal forced vibration testing. However, it was realized that the time history of accelerations from nearby blasts would have a higher frequency content than an earthquake accelerogram.

A photograph of one of the more visually exciting test blasts is shown in Fig. 8. This particular single blast used 70 sticks [21 lb (10 kg)] of dynamite in a vertical hole between depths of 2 ft–10 ft (0.6 m–3 m) below the surface and some 30 ft (10 m) behind the face of the wall. Although shallow blasts within the backfill created a spectacular dust cloud and thrilled the many spectators who gathered for the occasion, the more significant data from response evaluation point of view were obtained from deeper blasts and especially from the blasts detonated in front of the wall.

A total of 20 explosive tests were performed using a wide variety of explosive types, detonation rates, locations, and depths of the explosives. In the first day of 2 days of testing, small charges were placed at varying distances from the wall to determine the attenuation properties of the foundation soil, 10 ft–50 ft (3 m–15 m) in front of the tie and 25 ft–50 ft (8 m–15 m) behind the wall. In addition, the response of the wall to both fast and slow detonating powders was also measured. The amount of dynamite varied between 0.3 lb–2.7 lb (150 g–1,200 g) per blast, giving peak ground accelerations of 0.01 g–0.8 g horizontal and up to 0.38 g vertical. These initial tests were useful in designing the subsequent stronger explosive tests.

During the first day of testing it was observed that the slow detonating powder, Extra Gel, produced a greater response of the test wall than an equal amount
of the fast detonating explosive, Hercomite. Based on these initial observations, the second and final day of testing used larger amounts of an even slower burning explosive, HP-90A. These tests used from 5 lb–20 lb (2 kg–9 kg) of dynamite and produced peak accelerations at the base of the wall ranging up to 1.46 g horizontal and 1.25 g vertical. In both days the high accelerations were associated with very low vibration periods less than 0.1 sec. For the single short blasts there was usually only one initial peak or full cycle followed

![Graph](image)

**FIG. 9.**—Typical Record from Short Delay Multiple Blast Test

by several cycles of longer period and lower amplitude accelerations.

Four of the test blasts used multiple delayed detonations. In general, the multiple delay test results look a little more like an earthquake than the single shots. Unfortunately, the available delay caps were either too short to reproduce realistic earthquake frequencies or so long that the results were like separate single shots. Time and budgetary constraints in the project limited the amount of research that could be done on determining the best blasting procedures for simulating realistic earthquake motions. In retrospect, it is now felt that much larger amounts of explosives located at much greater distances from the wall and the use of many multiple delay charges would have been necessary to simulate realistic strong earthquake motions. Nevertheless, the results obtained from the blasting were useful in extending the response of the wall to higher levels of excitation than could be obtained from the forced vibration devices.

An example of a time-history record from a typical short delay multiple blast test is shown in Fig. 9. The data show time histories of accelerations recorded at the top of the wall and forces produced at the wall in an embedded tie located at El. 13.75 ft (4.2 m). The acceleration time history was integrated twice to obtain the displacement time history. This data were obtained during the first day of the testing using charges. The dynamic deformations of less than 0.2 mm were small compared to the 6-m high test wall, and the permanent lateral deformations following each blast were virtually zero. Note that the permanent deformations were measured at the end of many of the blasting sequences so that the conclusion regarding negligibly small deformations for this series of tests was based both on the results of the double integration technique and the directly measured residual end-of-test displacements.

An overall summary of the complete loading time history of the test wall is presented in Fig. 10, including construction, forced vibration, and dynamite blasting. The waiting time between the aforementioned three major stages and the waiting time between individual blasts has been removed to shorten the time scale for convenient presentation. Note that the outward wall movements were measured at level 8 (Fig. 5) located at about the upper third point on the wall. During construction of the wall and backfill above level 8 this point on the wall moved out 0.34 in. (8.6 mm). The several series of forced vibration testing created an additional 0.1 in. (2.5 mm) to give a total of 0.44-in. (11-mm) outward movement.

The small blasts in front of the wall caused a very small amount of additional outward movement. Only when large blasts were detonated behind the wall did the outward movement become appreciable and then only for the very high intensity of seismic loading. The largest single outward movement was for the single short blast No. T6R2 when the wall moved from 4.48 to 8.28 = 3.8 in. (95 mm). This blast detonated 64 sticks [19.2 lb (8.8 kg)] of dynamite in one shot in a single hole located in the backfill a distance of 20 ft (6 m) behind the wall. The charge was located at about mid-depth in the backfill. The peak ground surface accelerations at the wall were 1.46 g at the toe and 1.25 g at the top. This seismic movement was concentrated in a single cycle with a period of less than 0.1 sec.

Although not like the usual large magnitude earthquake, nevertheless, the combined blasting effect was a severe seismic loading condition to the test wall. The total accumulative measured outward movement during all of the blast loading amounted to an outward tilt of only about 5.5%. This amount of movement is visible by sighting along the wall, but would not be detrimental to many applications. The wall remained standing after all the blast loading with no indication of any permanent reduction in the overall static stability.

**STRAIN DEPENDENT DYNAMIC EXCITATION**

Values of the first mode frequency and the maximum dynamic peak to peak shear strain calculated from all of the blast and forced vibration tests are shown
plotted in Fig. 11(a). The first mode frequency of the wall during each explosive test was obtained by performing a Fast Fourier Transform on the digitized record of the accelerations obtained at the top of the wall. The maximum deformation of the top of the wall was obtained by double-integrating the acceleration time history. The maximum dynamic strain of the soil in the backfill during each blasting test was defined as the ratio of maximum dynamic deformation to the height of the wall.

The data show a significant nonlinear decrease in the first mode frequency with increasing dynamic strain excitation. This type of behavior was also noted from previous tests on small model walls (3) and follows from the well-known nonlinear dynamic properties of soil (5). For reference, a calculated curve is also shown in Fig. 11(a). The calculated first mode frequency at various strains was based on the shape of the Seed-Idriss (5) nonlinear shear modulus strain curves for sand at \( D_s = 65\% \) and evaluating \( f_1 \) from the shear modulus using Eq. 6. This calculated reference curve is in good agreement with the measured data points over the entire range of dynamic strains for less than 0.001% to almost 1.0%.

Note that the Seed-Idriss curves are based on single amplitude shear strains. Therefore single amplitude strains were also used in evaluating the data from the wall vibration tests. The cyclic movements of the reinforced earth wall generally were outward and back to the at-rest condition, which was taken as equivalent to a single amplitude strain.

The values of damping developed during the explosive excitation tests were estimated using the log decrement method for the wall response following the blast excitation. The damping values and maximum dynamic shear strains calculated from the blast and forced vibration tests are shown in Fig. 11(b).

For reference, the range of damping values for sands recommended by Seed-Idriss (5) are also shown. The explosive test damping values approximate the upper limit of the Seed-Idriss range of damping values; the forced vibration damping values are significantly greater than both the explosive test damping values and the Seed-Idriss damping values. This high level of forced vibration damping may be due in part to errors in analyzing the low amplitude and poorly defined modal resonance peaks that are typically obtained from the forced vibration test data. However, it is felt that the high damping, especially for the forced vibration tests, represents a considerable amount of geometric damping or energy loss radiating away from the small footing zone where the vibrating energy was being introduced into the system. The Seed-Idriss curves apply only to material hysteretic damping. It should be expected that the forced vibration test results to a major extent and the blast tests to a lesser extent, which involve point source excitation near the measurement location, should involve some geometric energy loss as well as hysteretic damping. Therefore, these field test data should show higher damping than indicated by the Seed-Idriss curves.

The dynamic tie forces were of considerable interest during the explosive tests because the stability of the wall requires that the ties do not break. The peak tie forces measured during the dynamic tests consisted of an initial static tie force component plus an additional dynamic deviator tie force component caused by the wall response during the blast. The measured peak dynamic deviator tie forces, measured at the wall during one of the tests, are shown in Fig. 12. Also shown are the calculated peak tie forces obtained using the design procedure shown in Fig. 1. The response spectra values used for calculating the design dynamic deviator tie force curve were obtained using the actual acceleration record made during the test and damping \( \beta = 6\% \), which was estimated from Fig. 11(b) using the appropriate dynamic strain level. The data show that the measured dynamic deviator tie forces are considerably less than the calculated values.

The calculation method suggested by Fig. 1 was based on a series of small model tests using one arrangement of tie distribution in the backfill. Based on data from ongoing model tests, it is felt that the discrepancy between the calculated and the measured dynamic deviator tie forces shown in Fig. 12 is primarily due to the influence of the length, arrangement, and density of the reinforcing ties in the backfill. In general, the longer the ties, the higher the maximum dynamic tie forces. The models used to derive the Fig. 1 procedure involved relatively longer ties than those used in the field prototype tests. Thus, the field test results suggest the need for a modification to the seismic design procedure of Fig. 1, which is the subject of continuing studies.

**Permanent Deformations**

After satisfying stability requirements to ensure that the wall will not completely collapse, the second major concern to a user or to an owner of a reinforced earth wall is the amount of deformation that may be expected under both static and dynamic loading.

The amount of permanent deformations caused by the several stages of construction and testing of this field prototype wall have been described in
previous pages. In all stages of the program the deformations were small and probably within tolerable limits for most practical applications despite some rather severe dynamic loading conditions that were imposed. The wall was designed according to standard procedures for static loading. Thus on a qualitative basis it would appear that a standard designed Reinforced Earth wall of moderate height should show small deformations resulting from strong seismic shaking. The data from this study and supplementary model tests are currently being analyzed for the purpose of developing a more quantitative procedure for predicting displacements caused by earthquake shaking.

CONCLUSIONS

This paper has presented some preliminary results of field studies undertaken to develop a logical method of seismic design for reinforced earth retaining walls. The study involved construction and testing of one 20-ft (6-m) high wall plus a limited amount of forced vibration testing on four other existing commercial walls. The special test wall and the other four walls were all designed according to the standard methods used by the Reinforced Earth Co. for static conditions only. Thus, the results provide an indication of the performance of a standard Reinforced Earth wall.

The simplicity and ease of construction of reinforced earth walls were demonstrated with the construction of this test wall solely by three previously inexperience persons during a 10-week period, including attention to extensive instrumentation.

Good agreement was found between the predicted and the measured tie forces under static loading during and immediately after construction.

Fundamental to the proposed seismic design method is knowledge of the damping and the resonant frequencies in the first and second modes of vibration. The test data led to a consistent empirical relation between these frequencies and the wall height for all walls tested. This empirical relation was in good agreement with theoretical predictions based on simple wave propagation theory formula (Eq. 6) and the Seed-Idriss (5) nonlinear shear modulus of dynamic strain.

The measured dynamic deviator tie forces in the prototype walls were considerably less than calculated from the technique developed on the basis of tests with small model walls of one geometric design (Fig. 1).

The general satisfactory behavior of the test wall, designed only for static loading, was well illustrated by its overall performance at the end of construction and throughout a total duration of over 13 sec of strong shaking with several peak accelerations in excess of 0.5 g. This shaking caused a permanent outward tilt of only about 5%, but no other observable damage or ill effects.

Studies are currently under way to improve the seismic design methods for reinforced earth walls subjected to seismic shaking. The intended improvements include a better definition of the components of the tie forces caused by shaking and a quantitative evaluation of the amount of deformation resulting from seismic loading. Preliminary data obtained thus far indicate that the tie forces and the deformations pertaining to seismic loading are interrelated with the entire geometry of the tie layout. For example, relatively short ties which provide minimum required stability for only static loading may slip somewhat through the soil backfill during seismic shaking, whereas longer ties would show less slippage. On the other hand, short ties that show high slippage will develop lower seismic forces than long ties that show a relatively small amount of slippage. The problem is further complicated by the tie spacing, tie width, surface texture, and distribution of tie lengths in the backfill. These complicated interrelations are believed responsible for the high seismic tie force components predicted by Fig. 1 as compared with the measured relatively low tie forces in the prototype tests.

Although other tie arrangements would probably lead to different measured tie force response to seismic loading, it is believed that design methods by the current Reinforced Earth Co. procedure or other comparable procedures for static loading should be sufficiently conservative for moderately high walls to insure stability during a moderate earthquake. Repeating an important conclusion from the previous Richardson and Lee (3) study for seismic designs, it is prudent to be more conservative with respect to tie breaking than tie slipping, because a broken tie could lead to a total collapse, whereas inadequate seismic slipping resistance would result only in some excess deformation.

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APPENDIX—REFERENCES
