

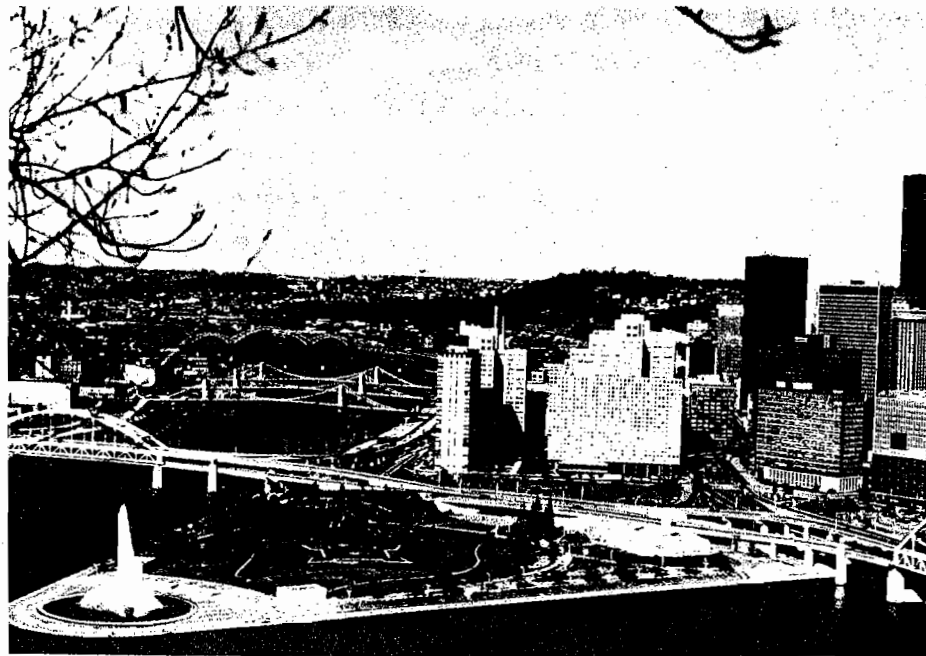


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EARTHQUAKE RESISTANT REINFORCED EARTH WALLS

Gregory N. Richardson, M.ASCE¹

INTRODUCTION

Present reinforced earth structures use bars or strips of galvanized steel embeded in the soil to provide a psuedo cohesion or tensile strength within the soil. Reinforced earth walls use layers of reinforcing placed between successive lifts of backfill. The reinforcing strips are attached to an outer skin that acts as the face of the wall. In theory this outer skin is assumed to be non-structural; in practice the outer skin is designed to resist localized lateral earth pressures.

Reinforcing strips are generally designed to resist applied static loads. Each strip is investigated for its tensile strength and its soil-strip bond resistance by a procedure analogous to the calculations of an anchored tie. Vertical and horizontal earth pressures adjacent to the skin, assumed to be principal stresses, may be found by finding the vertical stress and using Rankine's active lateral earth pressure coefficient to obtain the horizontal stress. For long walls, it is assumed that the design load for a given strip may be obtained by summing the lateral earth pressure acting over the tributary wall area of the strip. Details of the above described static design procedures, laboratory data, and field data have been previously presented in the literature (1,2).

During an earthquake these strips must resist additional forces which are produced by the relative acceleration of the inertial mass of the backfill. Thus the total force system within each strip is assumed to be the sum of the static forces acting before the seismic event plus the

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dynamic forces generated during the seismic activity. After the earthquake activity subsides, the forces in the strips are assumed to return to the static load pre-earthquake levels. The static forces in a reinforced earth wall can be calculated using referenced static design procedures (1,2).

A previous study by Richardson and Lee (3) suggested an empirical procedure for estimating the dynamic forces in each strip. This procedure was based on adapting spectral analysis techniques to data measure in harmonic shake-table tests of model reinforced earth walls. The total lateral dynamic force was shown to be proportional to a design spectral acceleration given by

$$A_{des} = \Gamma_1 S_{a1} + \Gamma_2 S_{a2} \quad (1)$$

where Γ_1 and Γ_2 are the first and second modal participation factors, and S_{a1} and S_{a2} are the first and second spectral accelerations. Note that S_{a1} and S_{a2} are functions of the first and second modal or natural frequencies and damping of the reinforced earth wall. The distribution of this total dynamic force was assumed to be independent of the reinforcement geometry.

To supplement the empirical data obtained from the laboratory tests, a full-scale test wall was constructed (4). Large mechanical vibrators were then used to determine the first and second low strain (0.001%) natural frequencies of the test wall. Additional forced vibration tests were conducted on four existing reinforced earth walls ranging in height from 7.5 ft. to 28 ft. The data showed that the resonant frequencies for the first two transverse modes, f_1 and f_2 , in hertz, are defined by

$$f_1 = \frac{125}{H} \quad (2)$$

$$f_2 = \frac{330}{H} \quad (3)$$

where H is the clear or unembedded height of the face of the wall in feet.

Impulse loads (dynamite explosions) were used to generate large dynamic strains within the full-scale test wall. The first mode frequency of the wall during each explosion was obtained by performing a Fast Fourier Transform on the digitized record of accelerations measured at the top of the wall. The data showed a significant nonlinear decrease in the first mode frequency with increasing dynamic strain. A frequency correction factor, FCF, was used to fit these data based on the shape of the Seed-Idriss (5) nonlinear shear modulus vs. strain curves, as shown in Fig. 1a. The natural frequency of a reinforced earth wall experiencing a peak dynamic strain, ϵ , is equal to the low strain resonant frequency of the wall, as defined by Eq. 2, multiplied by FCF for a strain of ϵ on Fig. 1a.

The levels of damping developed during the explosive tests were measured using the log decrement method. The damping values and maximum dynamic shear strains are compared in Fig. 1b. These damping values represent both geometric damping and material hysteretic damping

Dynamic forces in the reinforcing strips were measured during the explosive tests and compared with those predicted using a seismic design procedure earlier proposed by Richardson and Lee (3). The spectral accelerations for Eq. 1 were calculated using natural frequencies from Eqs. 2 and 3 and damping and FCF given on Fig. 1. A comparison of the measured and predicted dynamic forces indicated that the seismic design

procedure earlier proposed by Richardson and Lee (3) grossly over-estimates the total dynamic force and does not accurately portray the dynamic force distribution within the reinforced earth system.

It will be the objective of this paper to propose a simple design procedure for evaluating dynamic force magnitudes in each strip. This new design procedure incorporates data obtained from subsequent laboratory shake-table tests performed to measure the influence of the reinforcement distribution on the dynamic response of reinforced earth walls. The simple seismic design procedure is postulated based on the observed influence of reinforcement distribution and correlation with data from the earlier full-scale test wall.

The proposed seismic design procedure also incorporates statistical response spectra to model earthquake ground motions for seismic magnitudes ranging from M4.0 to M8.5. Sufficient design aids are provided such that a complete seismic design can be accomplished using hand calculations. The proposed seismic design procedure is also applicable if an actual design acceleration time-history is available for the site.

INFLUENCE OF REINFORCEMENT DISTRIBUTION

During earlier laboratory shake-table tests on model reinforced earth walls, it was observed that the response of a wall is influenced by the amount of reinforcement used. Walls that are heavily reinforced undergo only moderate relative acceleration in the backfill, whereas lightly reinforced walls experience large relative accelerations. Since the dynamic lateral forces are proportional to the relative accelerations, the above observation implies that heavily reinforced walls will actually experience smaller seismic forces than lightly reinforced walls subjected

to equal base motions. The interdependence of dynamic forces and degree of reinforcement suggests that the design will require an iterative procedure.

Based on both field and laboratory data, it has been observed that the base of reinforced earth walls remain stationary and the walls rotate about the base, see Fig. 2. This outward rotation of the wall is resisted by tensile forces generated within the reinforcing strips. Figure 2 illustrates that these tensile forces are in turn limited by the soil-strip frictional pullout capacity or the tensile strength of the strip. An empirical stiffness factor was developed (6) to quantify the influence of these tensile forces on the dynamic response of the model walls. The stiffness coefficient, I , is defined as the second moment of the ultimate tensile forces about the base of the wall. In conventional short walls (<25 ft.), the ultimate tensile forces will be governed by pullout capacity. In conventional high walls, the ultimate tensile forces in the lower portion of the wall will be governed by the tensile strength of the strips.

The magnitude and distribution of the dynamic forces, and the peak dynamic strain were found to be a function of the stiffness coefficient. The empirical relationships developed using the stiffness concept showed that the relative stiffness of one wall with respect to another was more important than the absolute value of this stiffness parameter. Thus, if the response of a wall with a given strip configuration is known, then the response of a second wall having an arbitrary strip configuration can be determined.

An elastic dynamic strain, ϵ , defined as the maximum peak-to-peak cyclic displacement divided by the wall height, was measured during the shake-table tests. It is shown in Fig. 3a that the elastic dynamic strains at resonance are inversely proportional to I . Therefore, if the peak dynamic strain ϵ_n for a wall of stiffness I_n , is known then the peak dynamic strain for a wall of stiffness I_x will be

$$\epsilon_x = \epsilon_n \frac{I_n}{I_x} \quad (4)$$

assuming an uniform base motion.

The total dynamic lateral forces were found to be sensitive to both the excitation frequency and the wall stiffness. The total dynamic force is maximum near resonant frequencies of the wall and is inversely proportional to the stiffness of the wall.

Plotting the total resonant dynamic lateral force data on Fig. 3b, the total resonant lateral force, F_I , can be defined as:

$$I < 2.0 \quad F_I = \frac{F}{I}, \text{ and} \quad (5)$$

$$I > 2.0 \quad F_I = \frac{F}{I} = 2 \quad (6)$$

where F is the total dynamic force measured in a wall defined to have a stiffness of 1.0. The reference wall having $I=1.0$ was arbitrarily defined as the wall having a minimum static factor of safety against strip failure. As more reinforcing is added to the wall, the total dynamic lateral force decreases. The increased reinforcement stiffens the wall and reduces the level of relative acceleration in the backfill. This reduction in acceleration

correspondingly reduces the dynamic forces. Apparently for a wall having a stiffness greater than 2 the reinforced mass acts as a shear block and the additional reinforcement does not reduce the accelerations or the total dynamic forces.

The distribution of dynamic forces within the reinforcement system as a function of wall stiffness I is shown in Fig. 4. The average dynamic lateral earth pressure is assumed to act at the midheight of the wall and is defined as:

$$F_{0.5H} = \frac{F}{H} \times \frac{1}{I} \quad [I < 2.0] \quad (7)$$

where H is the height of the wall. The dynamic lateral earth pressure at the base of the wall is given by

$$F_{0.0H} = \frac{F_{0.5H}}{I} \quad (8)$$

where I may be greater than 2. Thus, as the wall is made stiffer, the dynamic earth pressure distribution approaches an inverted triangular distribution commonly assumed in the Psuedo-Static design of conventional retaining walls.

DESIGN RESPONSE SPECTRUM

The total lateral dynamic force was shown (3) to be proportional to Eq. 1 which is in terms of the first and second modal spectral accelerations. These spectral accelerations are determined for each mode using a response spectra generated using a design earthquake time history. A response spectrum being simply a plot of the maximum response of a 1-degree-of-freedom oscillator to the design earthquake motion versus the natural frequency of the

oscillator, $\omega = \sqrt{k/m}$.

Evaluation of a design response spectrum is dependent upon the selection of a design earthquake time history. The seismic design procedure developed herein uses statistically defined response spectra to eliminate the need for defining an arbitrary design earthquake for each structure designed.

Newmark and Blume (7) analyzed 61 earthquake records and defined statistical response spectra for a range of earthquake magnitudes based on this data set. This early work was updated by Newmark (8) for use on the Alaskan pipeline. In this work, two levels of design earthquakes are defined; a low level "operating earthquake" associated with a return period for the design earthquake of approximately 50 years and a "contingency plan earthquake" associated with a return period of about 100 to 200 years. The analysis developed herein uses the more severe contingency spectra.

A procedure for defining a design spectrum was developed based on the statistical procedures presented above and is shown in Fig. 5. To define a design spectrum it is necessary to know the magnitude of the design earthquake and the damping in the reinforced earth wall during the earthquake.

PROPOSED SEISMIC DESIGN PROCEDURE

A seismic design procedure was developed using 1) data from a full-scale field test (4), 2) the stiffness relationships observed in the shake-table tests (6), and 3) the statistical response spectra concept (7,8). Empirical relationships developed using the stiffness concept showed that it is possible to predict the performance of a wall having arbitrary reinforcing if the response and reinforcement geometry for a reference wall are known. The full-scale test wall (4), with its known strip configuration

and measured response, served as a basis for this reference.

Based on the reinforcement configuration of the full-scale test wall, a reference reinforcement geometry was defined;

- 1) having a uniform strip length equal to $0.8H$, where H is the height of the reinforced zone of fill,
- 2) having a uniform horizontal and vertical spacing of the reinforcing strips, and
- 3) having a minimum factor of safety against static failure (F.S.=1.0).

The above criteria results in a reference stiffness, I_{ref} , given by

$$I_{ref} = \sum_{i=1}^N d_i^2 F_i \quad (9)$$

where d_i is the height above the base to the i^{th} strip, N is the total number of strips in a tributary width, and F_i is the minimum yield strength of the strip as defined by either yielding of the soil-strip friction bond or of the strip itself.

The stiffness, I' , for a design wall of arbitrary strip configuration is then defined as

$$I' = \sum_{j=1}^M d_j^2 F_j \left[\frac{S}{S_j} \right] \quad (10)$$

where S is the horizontal spacing of the strips in the reference wall, S_j is the spacing of the j^{th} strip layer in the design wall, and M is the number of strips in the design wall.

The normalized stiffness of the design wall, I , may be obtained by combining Eqs. 9 and 10 to give

$$I = \frac{I'}{I_{ref}} \quad (11)$$

For low walls (<25 ft.) this relationship is simply a comparison of the relative stiffness of the design wall to that of the full-scale test wall. Using this criterion, the full-scale test wall has a normalized stiffness $I = 1.0$. For high walls these relationships are based on the additional assumption that the stiffness would be limited by the tensile strength of the reinforcing as well as soil-strip frictional strength.

A procedure to evaluate the peak dynamic strain was developed based on the measured response of the full-scale test wall (4) during blast tests. The procedure is based on an empirical correlation between the displacements measured in the blast tests and that predicted by the computer program LEVSFC. The program LEVSFC was developed by Idriss and Seed (9) to provide an initial estimate of the acceleration amplification and displacement due to horizontal shearing of earth structures. The correlation of measured and predicted peak dynamic strains led to the following expression (11):

$$\epsilon = \frac{1.1}{I} \epsilon_{\text{LEVSFC}} \quad (12)$$

where ϵ is the peak dynamic strain of the design wall, I is the normalized stiffness of the design wall, and ϵ_{LEVSFC} is the peak dynamic strain calculated by the program LEVSFC.

The program LEVSFC requires an actual earthquake to calculate the peak dynamic strain. To remove the need for an actual earthquake time history, a large number of LEVSFC analyses were conducted on wall configurations ranging in height from 15 to 55 feet. Earthquake time histories have original magnitudes ranging from 4.5 to 8.5 were used in these analyses. Relationships developed by Richter (10) and incorporating later modifications by Newmark were used to scale the amplitude of the earthquake records to a

range of magnitudes. This relationship and the peak dynamic strains versus earthquake magnitude from these analyses are shown in Fig. 6.

The total dynamic force calculated using the design acceleration given in Eq. 1 is a function of the modal spectral accelerations and modal participation factors. The modal spectral accelerations can be calculated for a given wall height, H, and magnitude of earthquake, M, in the following manner:

- 1) M: estimate the peak dynamic strain, ϵ , from Fig. 6b,
- 2) ϵ : estimate the design damping, β , from Fig. 1b,
- 3) M, β : develop the design response spectra from Fig. 5,
- 4) H: calculate the undamped low-strain natural frequencies, f_1 and f_2 from Eqs. 2 and 3,
- 5) ϵ : obtain the natural frequency strain reduction factor, FCF, from Fig. 1a,
- 6) FCF, f_1 , f_2 : estimate the strained natural frequencies, \bar{f}_1 and \bar{f}_2 , of the design wall, and
- 7) \bar{f}_1 , \bar{f}_2 : obtain the spectral accelerations from the design response spectra calculated in 3).

If an actual design acceleration time history is available for the site, then steps 1) and 3) could be performed using the time history and the appropriate computer analysis.

The modal participation factors for Eq. 1 had been estimated using a simple lumped mass model (3) such that

$$A_{des} = 1.25 S_{a1} + 0.5 S_{a2} \quad (13)$$

Comparison of the dynamic forces calculated using Eq. 13 to those measured in the full scale field test (4) indicated that Eq. 13 grossly overestimated the magnitude of the total dynamic force acting on the system.

Corrected modal participation factors were empirically calculated from

the wall data of the full-scale tests by redefining the total dynamic force as

$$\Sigma F = [C_1 S_{a1} + C_2 S_{a2}] M_{eff} \quad (14)$$

where C_1 and C_2 are functionally the same as the participation factors and M_{eff} is the effective mass of the reinforced earth wall. The effective mass was arbitrarily define as

$$M_{eff} = \frac{3}{4} K_o \gamma H^2 / g \quad (15)$$

to be consistent with the earlier procedure proposed by Richardson and Lee (3). Based on an analysis (11) of dynamic forces measured during blast tests on the full-scale test wall, Eq. 14 was found to equal

$$\Sigma F = [S_{a1} + 0.2 S_{a2}] M_{eff} \quad (16)$$

This empirical total dynamic force is based on ground motions that included significant vertical accelerations.

The amplitude and distribution of the total dynamic force calculated using Eq. 16 is also a function of the relative stiffness of the design wall. The distribution of dynamic forces in the full-scale test wall were found to be well defined by the relationship shown in Fig. 4. This relationship is therefore assumed to be applicable to all reinforced earth walls having a reasonably uniform reinforcement density. The actual dynamic earth pressures for a design wall are obtained by first calculating the magnitude of the total force for a wall of stiffness $I=1$ from Eq. 16 and then using Fig. 4 to obtain the correct amplitude and distribution of the dynamic earth pressures for a wall of arbitrary stiffness.

Sufficient reinforcement must be provided such that the combined static and dynamic earth pressures do not cause a tensile or soil-strip frictional

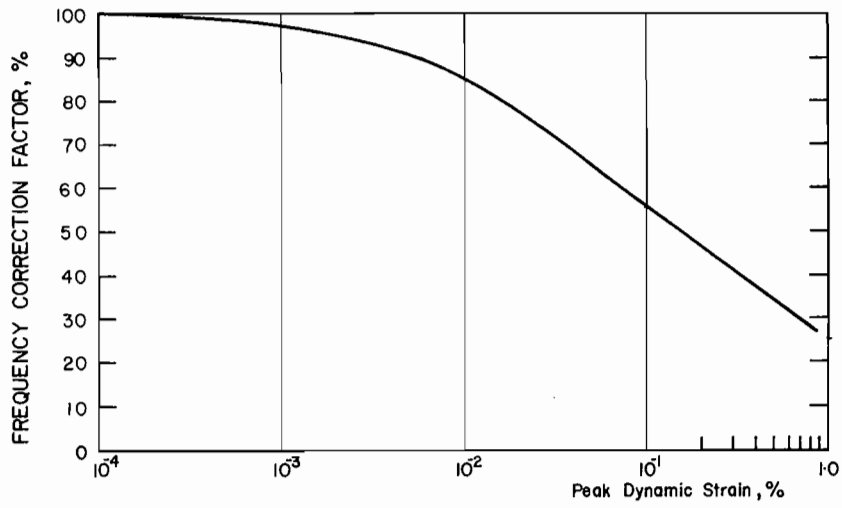
failure of the reinforcement. This is verified by applying the combined earth pressures over the tributary area of the strips to give design forces. These design forces must not exceed the ultimate tensile or soil-strip friction capacity of the strips. If reinforcement must be added to removed from the design wall, then the wall stiffness I will change and the entire analysis must be repeated. A flow chart of the design process is given on Fig. 7.

ACKNOWLEDGMENTS

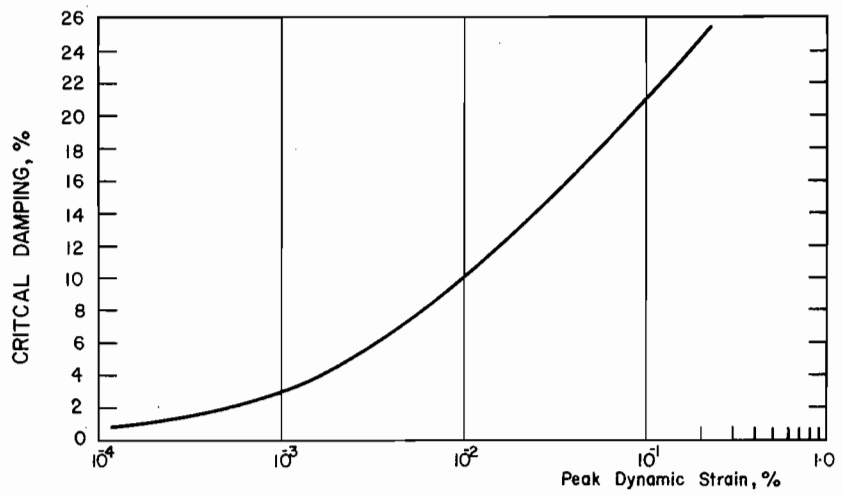
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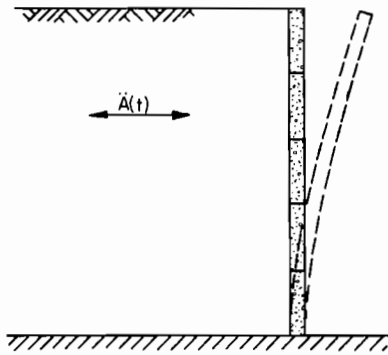


(a) NATURAL FREQUENCY PEAK STRAIN REDUCTION FACTOR

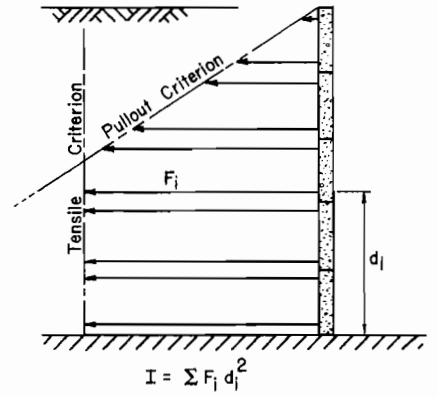


(b) DESIGN DAMPING versus PEAK STRAIN

FIGURE I MODAL DESIGN PARAMETERS

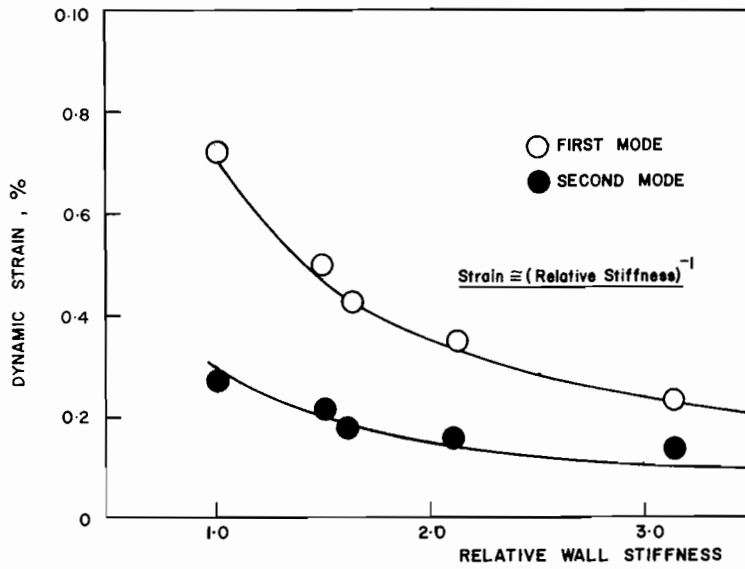


OBSERVED SEISMIC
LATERAL DEFORMATIONS

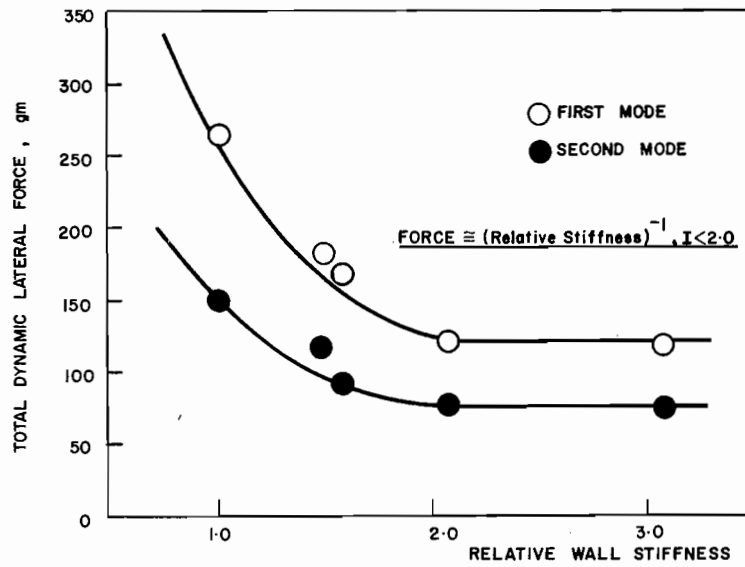


REINFORCEMENT FORCES
RESISTING DEFORMATIONS

FIGURE 2 CONCEPTUALIZED STIFFNESS



(a) PEAK DYNAMIC STRAIN



(b) TOTAL DYNAMIC LATERAL FORCE

FIGURE 3 DYNAMIC RESPONSE versus WALL STIFFNESS

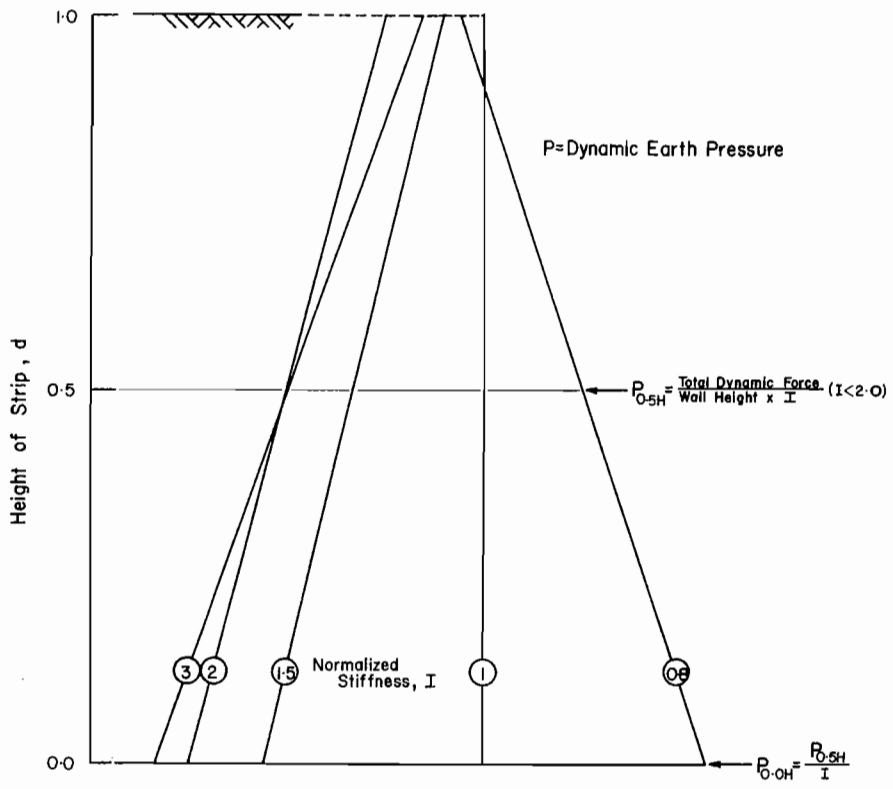


FIGURE 4 STIFFNESS versus DYNAMIC EARTH PRESSURE

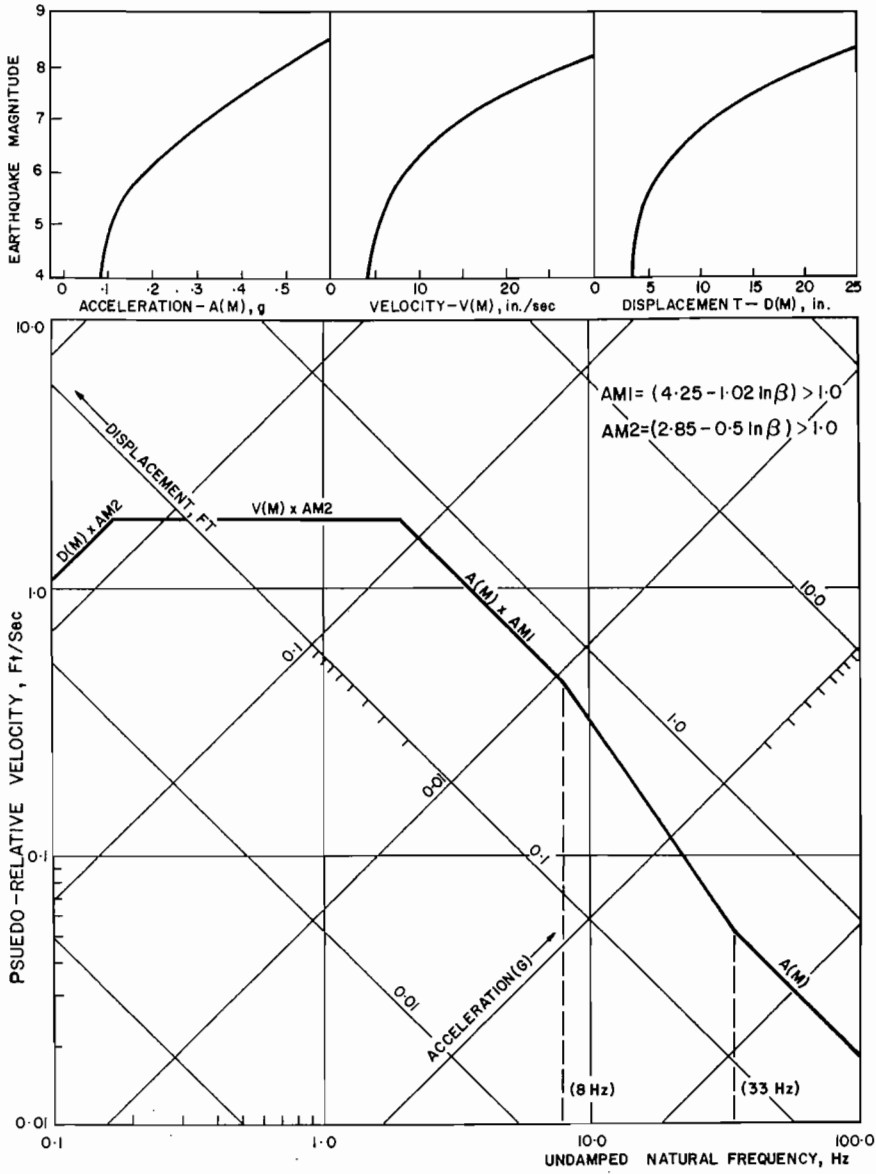
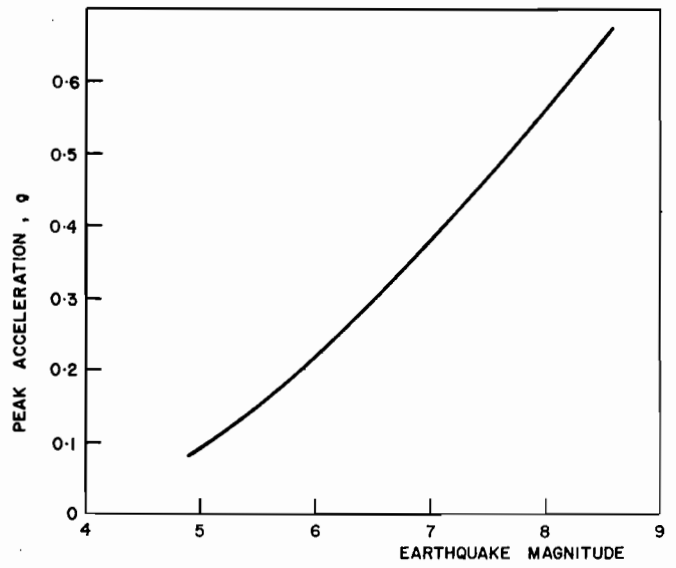
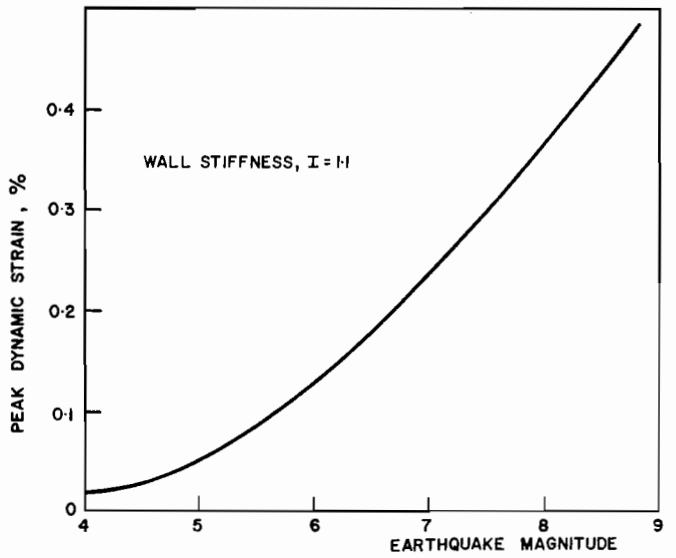


FIGURE 5 DESIGN RESPONSE SPECTRA



(a) PEAK ACCELERATION SCALING FACTOR



(b) PEAK DYNAMIC STRAIN, %

FIGURE 6 ESTIMATING PEAK DYNAMIC STRAINS

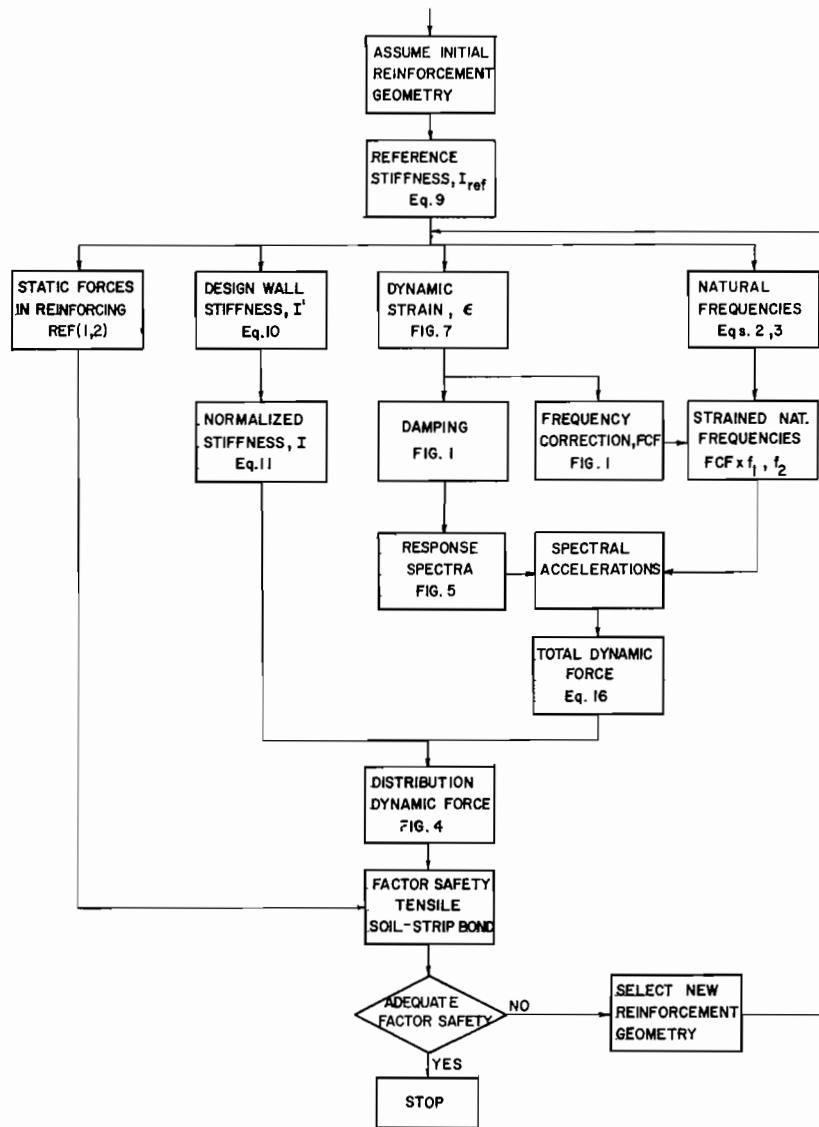


FIGURE 7 SEISMIC DESIGN PROCEDURE