ON THE ASSESSMENT OF THE DYNAMIC RESPONSE OF SOILS

Alemayehu Teferra
Civil Engineering Department
Addis Ababa University

ABSTRACT

The paper discusses two approaches for determining the dynamic response spectra of structures in which the effect of site conditions are taken into consideration. From the response spectra the estimation of maximum base shear is indicated. In light of the presented discussion, the earthquake loading as stipulated in the Ethiopian code of practice was examined. It has been found out that the loading criteria given in the code for structures founded on soft soils need modification and in consequence thereof a recommendation is proposed.

INTRODUCTION

Sites that are subjected to shaking or vibration either by man made means (blasting, underground explosion etc...) or by natural consequences (earthquakes) may suffer considerable damage. The effect of a man made vibration on a given site may be determined since the extent and intensity of the vibration is controlled at will. But this is not the case for earthquakes.

The extent and intensity of ground shaking caused by an earthquake are known only after the event. The occurance of earthquakes may lead to disaster unless appropriate counter measures are taken. To provide an adequate safety against disaster, it is necessary to have a high degree of expertise in earthquake engineering when in turn requires a deep knowledge in understanding the effect of ground motion on soil properties and that of structures. The study of the effect of ground motion on soil prosperity is generally known as geotechenical earthquake engineering.

Geotecbenical earthquake engineering is a relatively young decipline. It is in the early sixties that systematic investigation on the response of soils as a result of shaking and influence of site effects on the behavior of structures during earthquakes were undertaken [1]. It will be the task of this paper to present an overview of the different techniques used by scientists and engineers to assess the response of soils when subjected to earthquake shaking.

Soils respond differently to shaking according to their characteristic properties, thickness of deposit, pattern of stratification and site conditions. Loose granular deposits tend to compact due to ground vibration resulting in excessive settlements. In cases where the loose granular deposits are saturated, the ground shaking will induce the development of excess pore water pressure which would be sufficient to cause liquefaction (complete loss of bearing capacity) of soil. These types of soil instabilities would undoubtedly cause catastrophic damage to structures. However through appropriate ground investigation and design, it is possible to avoid such danger.

Apart from the above instabilities or permanent deformations soil respond to ground shaking and affect the structures that are founded on them. The assessment of this response is of equal importance to the safe design of structures. Considerable effort has been expended to develop methods for assessing the response of soils induced by ground shaking. Currently there are two methods for (statistical) relationships (in the form of curves) which have been deduced statistically from established patterns observed in actual earthquakes. The second method is based on theoretical analysis. Before delving into the discussion of the above two methods, it is appropriate at this stage to briefly discuss:

a) response spectra
b) the behavior of soil characteristics when subjected to vibration

RESPONSE SPECTRA

Seismic waves which are generated during an earthquake travel in all directions away from the source and be recorded by means of strong motion recording instruments. These instruments usually record the time history of accelerations. The data obtained directly from the recording instruments should be digitized and the necessary corrections for instrument, noise, base-line, etc. errors be conducted. A typical corrected accelerogram is shown in Fig.1 [3]. From the accelerogram one would be able to see the peak acceleration, the duration and the variation of the acceleration during the period of vibration. However, its effect on structures cannot be inspected directly. In order to study the response of buildings when subjected to acceleration, the following methodology has been introduced [4].

Consider a one-degree of freedom mechanical system having a mass m, a linear spring with a spring constant of k, and viscous damping c, acted upon by an acceleration $y(t)$ as shown in Fig.2. Let x denote the absolute displacement of the mass m, any y denote the absolute displacement of the ground. The extension of the spring will thus be $(x\text{-}y)$. The respective absolute accelerations of the ground and the mass are y and x. The response of the mass m, when acted upon by $y(t)$ is given by the following equation [4].

$$(x-y) = \frac{T}{2\Pi\sqrt{1-n^2}} \int_0^t \ddot{y}(\tau) e^{\frac{-2\pi n}{T}(t-\tau)} . \sin\frac{2\pi}{T}\sqrt{1-n^2}\,(t-\tau)d\tau \quad (1)$$

Where

T = undamped natural period of oscillation

$= 2\pi\sqrt{\frac{m}{k}}$

n = fraction of critical damping $= \frac{c}{2}\sqrt{\frac{k}{m}}$

τ = time parameter.

For small damping ($n < 0.2$), Eq. 1 may be simplified by equating $\sqrt{1-n^2} = 1$.

Hence:

$$(x-y) = \frac{T}{2\pi} \int_0^t \ddot{y}(\tau) e^{\frac{-2\pi n}{T}(t-\tau)} . \sin\frac{2\pi}{T}(t-\tau)d\tau \quad (2)$$

From Eq.2 , follow:

$$(\dot{x}-\dot{y}) = \int_0^t \ddot{y}(\tau) e^{\frac{-2\pi n}{T}(t-\tau)} . \sin\frac{2\pi}{T}(t-\tau)d\tau \quad (3)$$

$$\ddot{x} = \frac{2\pi}{T} \int_0^t \ddot{y}(\tau) e^{\frac{-2\pi n}{T}(t-\tau)} . \sin\frac{2\pi n}{T}(t-\tau)d\tau \quad (4)$$

The maximum values of $x\text{-}y$, $\dot{x}\text{-}\dot{y}$ and $\ddot{x}$ are important in engineering design work.

Since the same integral expression appears in all equations, the maximum value of the integral expression will be donated by S_v i.e.:

$$S_v = \left[\int_0^t \ddot{y}(\tau) e^{\frac{2\pi n}{T}(t-\tau)} \sin\frac{2\pi}{T}(t-\tau)d\tau \right]_{max} \quad (5)$$

Hence

$$(x-y)_{max} = \frac{T}{2\pi} S_v \quad (6)$$

$$(\dot{x}-\dot{y})_{max} = S_v \quad (7)$$

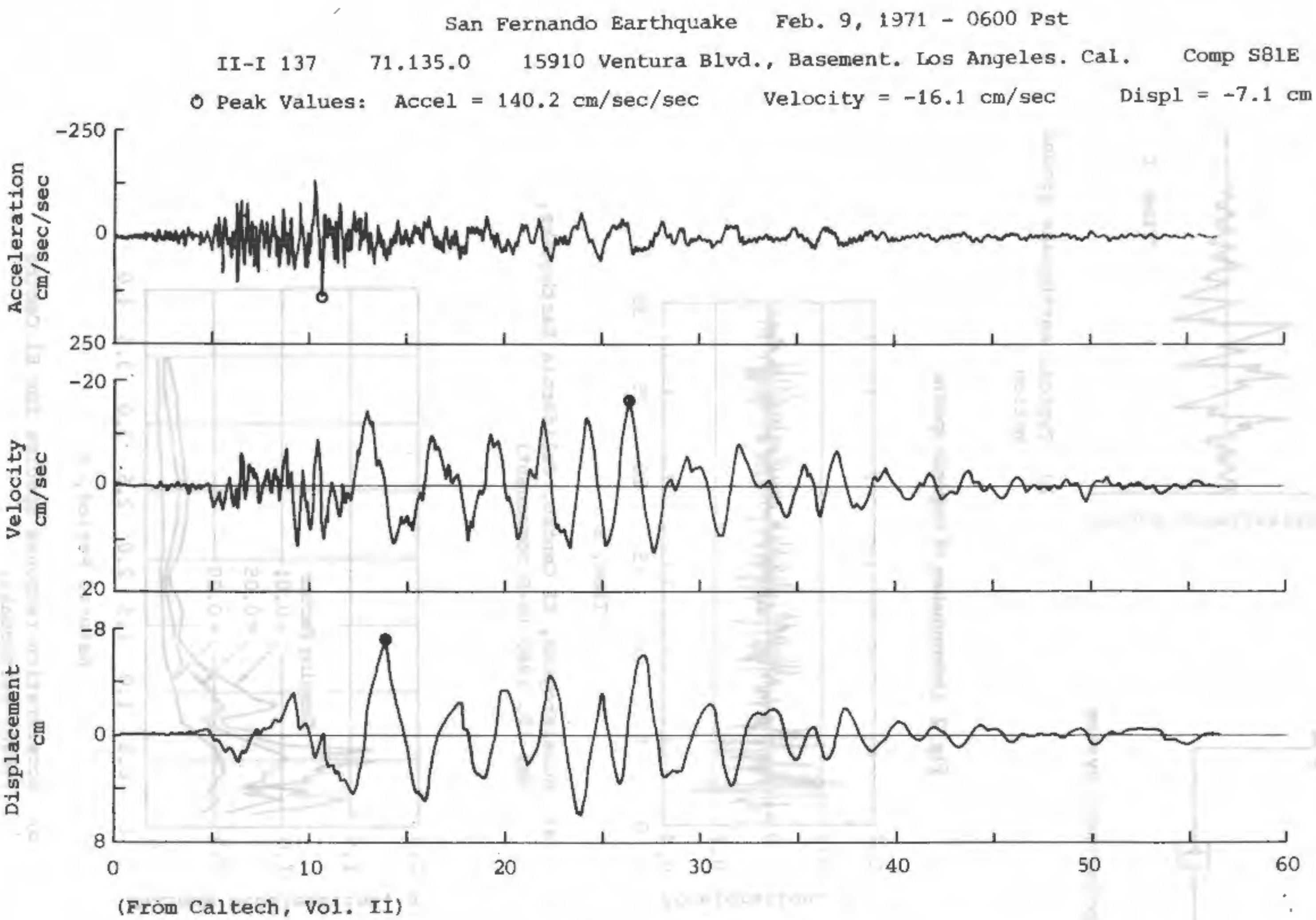
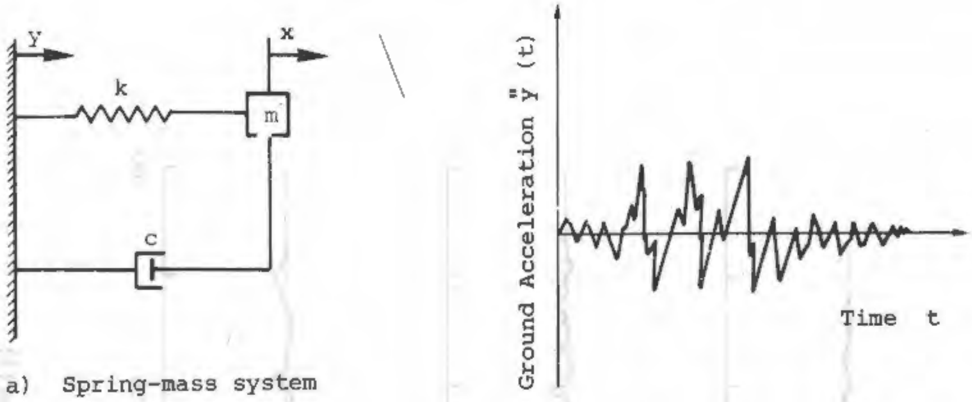


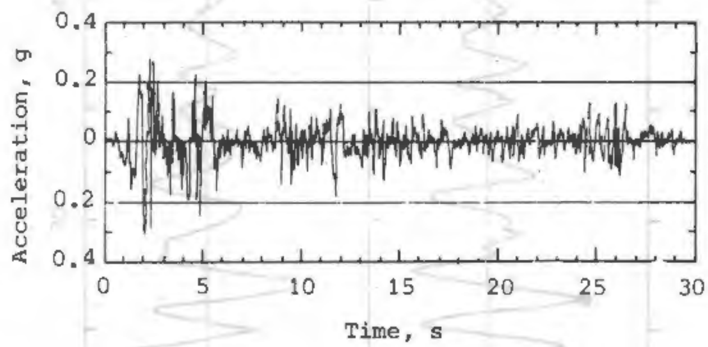
Fig. 1 Typical corrected accelerogram and integrated velocity and displacement time histories [3].



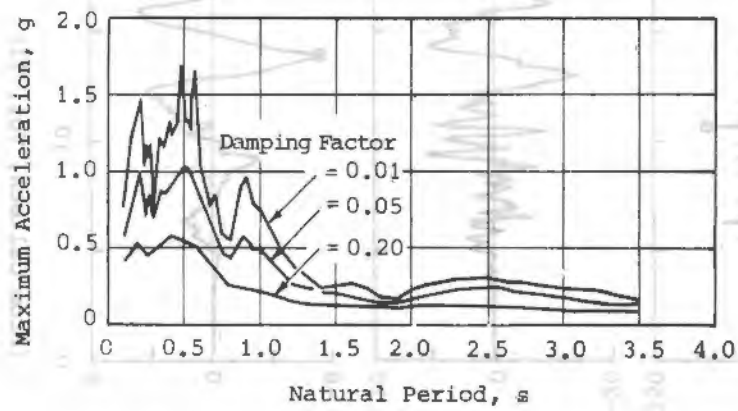
a) Spring-mass system

b) Typical earthquake ground motion

Fig. 2 Determination of response spectra



a) Accelerogram, El Centro, California Earthquake, May 18, 1940 (N-S component)



b) Acceleration response spectra for El Centro (1940) earthquake.

Fig. 3 Accelerogram and acceleration response spectra for El Centro earthquake [1].

$$\ddot{X}_{max} = \frac{2\pi}{T} S_v \qquad (8)$$

If one plots $\frac{T}{2\pi}S_v$ versus T, or $\frac{2\pi}{T}S_v$ versus T

for several values of the damping factor n, one obtains the displacement, or the velocity, or the acceleration response spectra. This operation is normally done by engineering seismologists.

A response spectrum is therefore, the maximum response induced by the ground motion in a single degree of freedom oscillators of different fundamental periods, but having the same degree of internal damping.

The acceleration response spectra for El Centro earthquake is reproduced in Fig.3. From the acceleration response spectra one should be able to determine the maximum acceleration of a building if its natural period of oscillation and damping are known (Fig.4).

BEHAVIOR OF SOIL CHARACTERISTICS WHEN SUBJECTED TO VIBRATION

The response of soils to vibration is determined mainly by their shear modulus and damping characteristics. The stress-strain relationships of soils under cyclic loading conditions are non-linear in form (Fig.5) This means, the shear modulus, which is the slope of the curve at given point, would vary. For ease of computation, however, the shear modulus may be expressed as the secant modulus determined by the extreme points of the curve say for the two conditions shown in the figure by G_1 and G_2 [5]

The dsmping factor is proportional to the area inside the hysteresis loop (Fig.5). These soil characteristics depend upon the magnitude of the strain for which the hysteresis loop is determined. Hence they are functions of the induced strain.

A wide variety of tests both in situ and in laboratory have been conducted to determine shear moduli and damping characteristics for a wide spectrum of strain values [5]. Seed and Idriss made a detailed parametric evaluation of a previous work done by Hardin and Drnevich [6] and proposed simplified equation for shear modulus of sand as follows:

$$G = 1000\ K_2\ (\overline{\sigma_o})^{0.5} \qquad (9)$$

Where

G = Shear modulus (lb/ft^2)

$\overline{\sigma_o}$ = effective stress (lb/ft^2)

K_2 = a parameter given in Fig. 6

One may use Eq. 9 also to estimate the shear modulus of gravelly soils with the appropriate value of the parameter K_2 which is given in Fig. 7. The corresponding value of the shear modulus in kN/m^2 may be obtained by multiplying the result of Eq. 9 by the factor 0.479.

For saturated clays, Seed and Idriss [5] have presented Fig. 8. In Figs. 9 and 10, the damping ratios for sand and for saturated clays are given.

GROUND RESPONSE AS A RESULT OF GROUND SHAKING

As stated earlier, the ground response may be studied using the analytical or the empirical methods, all of which have been developed at the University of California, Berkeley. It should be stated that due to the lack of sufficient earthquake records, the empirical method was introduced at a later stage. These two methods will be presented briefly.

ANALYTICAL METHOD

Closed-Form Solution for Homogeneous Soils

The vibration of soil layers induced by earthquakes are mainly attributed to the upward propagation of shear waves from the underlying rock of rocklike layer. If the soil layers and the rock or rocklike layer are essentially horizontal, the lateral dimension of the deposit does not affect the response and the deposit may be considered as a semi-infinite layer transmitting one dimensional shear beam in the vertical direction. It is using this approach that Idriss and Seed [7] developed their method. In the analysis, the materials are considered to be linear elastic and their properties (shear modulus and damping) may vary with depth following a simple mathematical expression.

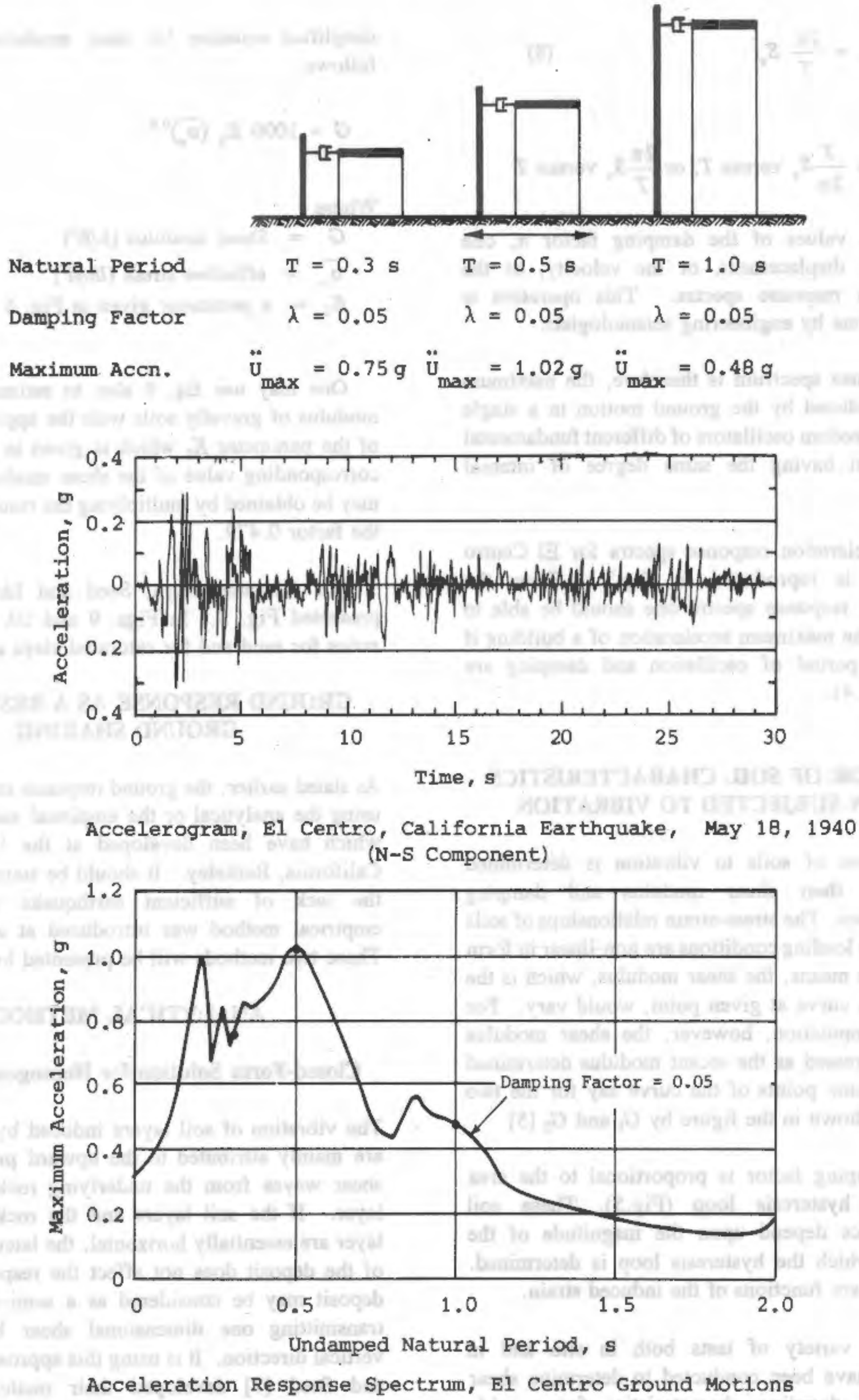


Fig. 4 Evaluation of acceleration response spectrum [1].

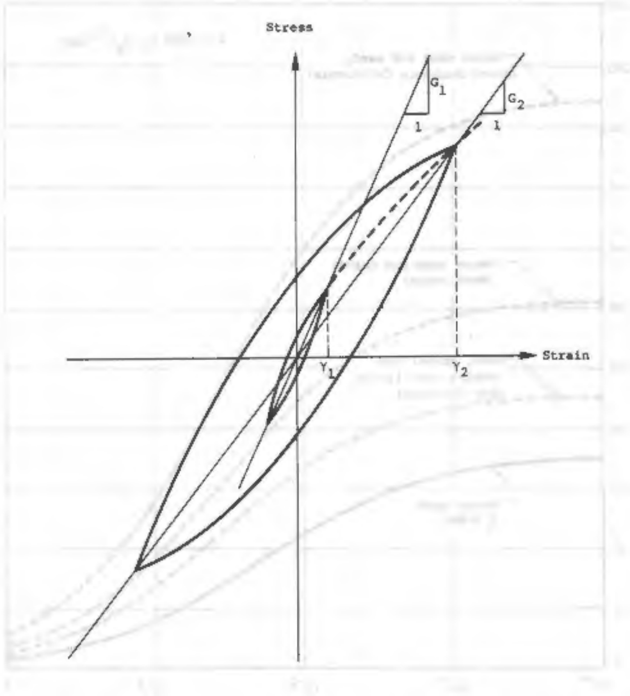


Fig. 5 Hysteretic Stress strain relationships at different strain amplitues

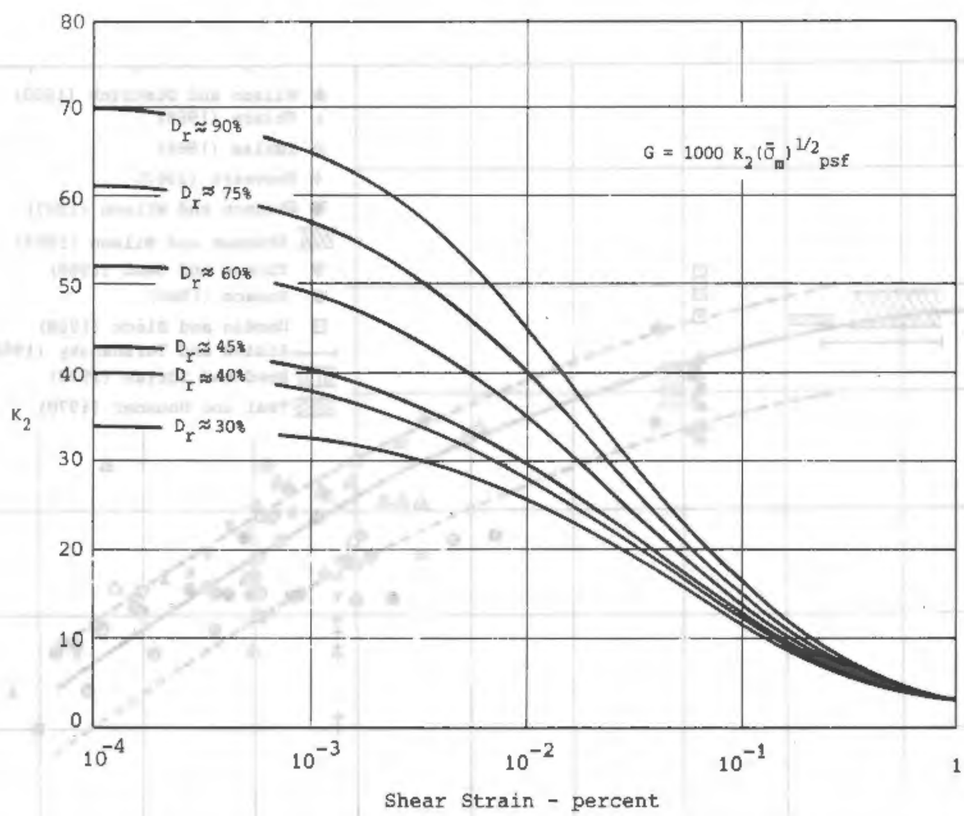


Fig. 6 Shear moduli of sands at different relative densities [5].

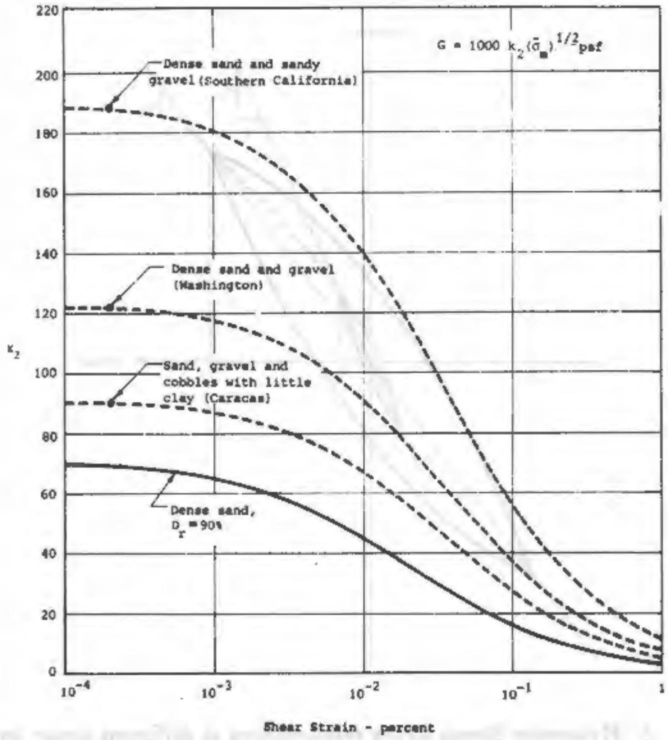


Fig. 7 Moduli determination for gravelly soils [5].

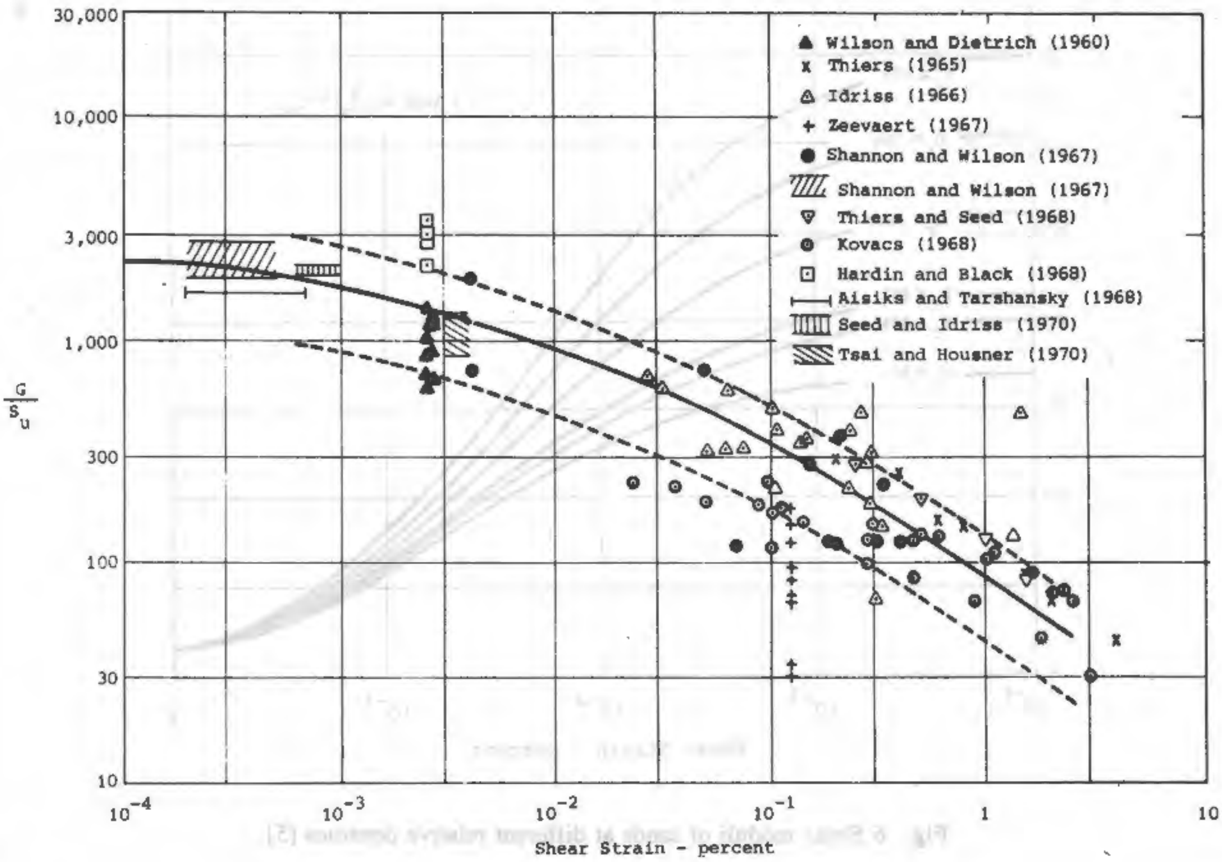


Fig. 8 In-situ shear moduli for saturated clays [5]

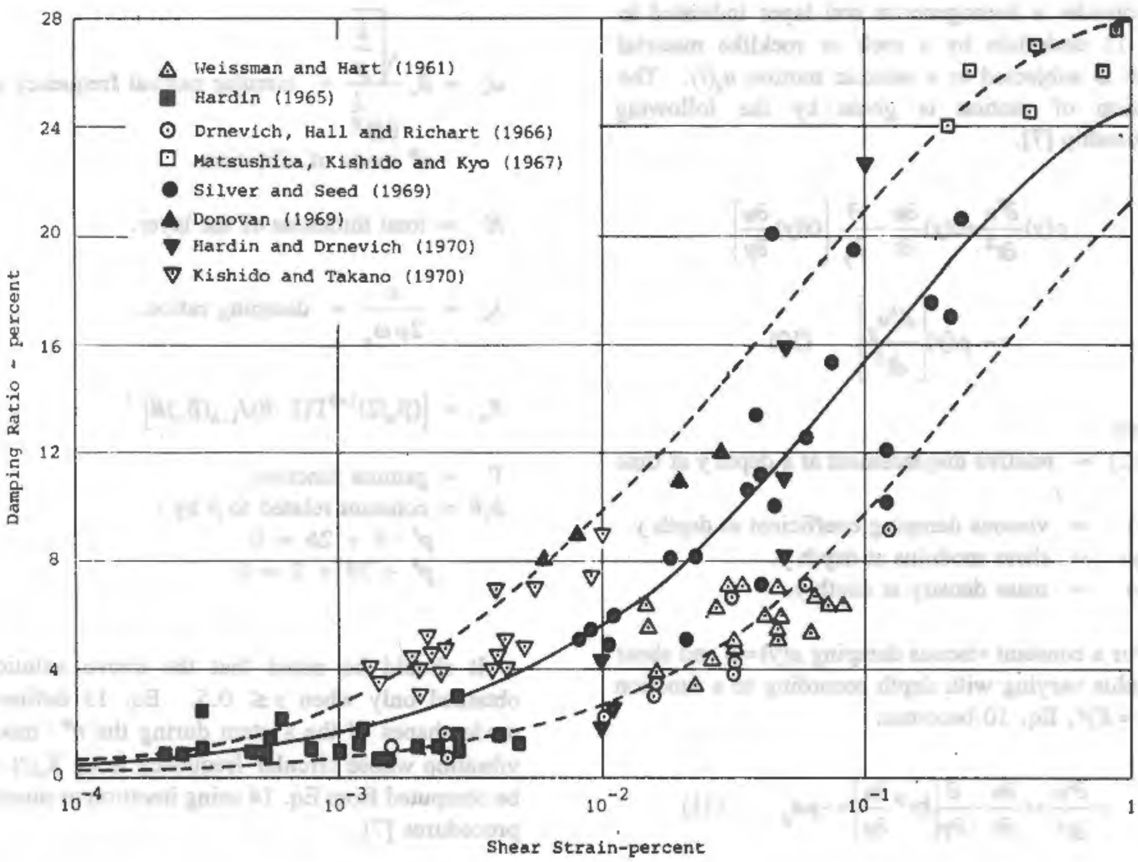


Fig. 9 Damping ratios for sand [5]

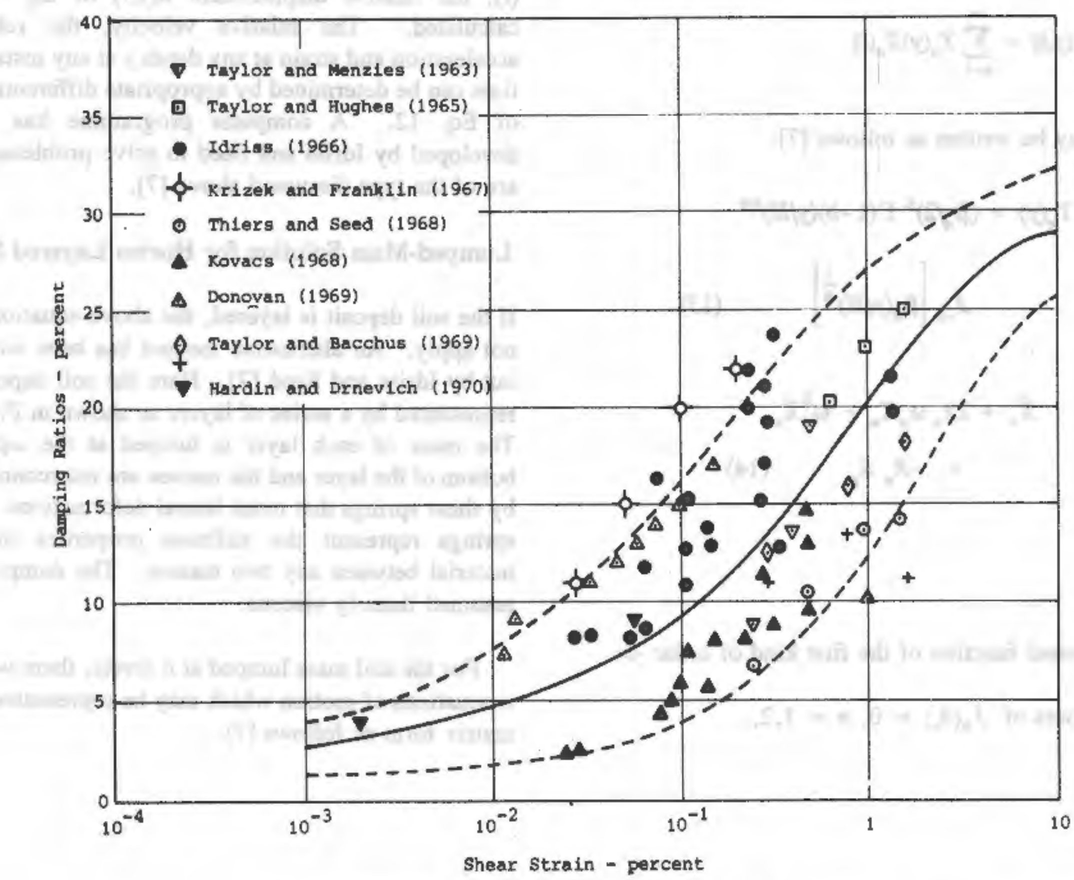


Fig. 10 Damping ratios for saturated clays [5]

Consider a homogeneous soil layer indicated in Fig. 11 underlain by a rock or rocklike material which is subjected to a seismic motion $u_g(t)$. The equation of motion is given by the following relationship [7].

$$\rho(y)\frac{\partial^2 u}{\partial t^2}+c(y)\frac{\partial u}{\partial t}-\frac{\partial}{\partial_y}\left[G(y)\frac{\partial u}{\partial y}\right].$$

$$.=-\rho(y)\left[\frac{d^2 u_g}{dt^2}\right] \qquad (10)$$

Where

$u(y,t)$ = relative displacement at a depth y at time t.

$c(y)$ = viscous damping coefficient at depth y.

$G(y)$ = shear modulus at depth y.

$\rho(y)$ = mass density at depth y.

For a constant viscous damping $\rho(y)=\rho$ and shear modulus varying with depth according to a function $G(y)=Ky^p$, Eq. 10 becomes:

$$\rho\frac{\partial^2 u}{\partial t^2}+c\frac{\partial u}{\partial t}-\frac{\partial}{\partial y}\left[ky^p\frac{\partial u}{\partial y}\right]=-\rho u_g \qquad (11)$$

By letting

$$u\ (y,t)\ =\ \sum_{n=1}^{\infty} Y_n(y)X_n(t)$$

Eq. 11 may be written as follows [7]:

$$Y_n(y)\ =\ (\beta_n/2)^b\ \Gamma(1-b)(y/H)^{b/\theta}$$

$$J_{-b}\left[\beta_n(y/H)^{\frac{1}{\theta}}\right] \qquad (13)$$

$$\ddot{X}_n\ +\ 2\lambda_n\omega_n X_n\ +\ \omega_n^2\dot{X}_n.$$

$$.=\ -R_n\ \ddot{u}_g \qquad (14)$$

Where

J_{-b} = Bessel function of the first kind of order $-b$.

β_n = roots of $J_{-b}(\beta_n) = 0$, $n = 1,2,\ldots$

$\omega_n = \beta_n \dfrac{\sqrt{\frac{k}{p}}}{\theta H^{\frac{1}{\theta}}}$ = circular natural frequency of n^{th} mode of vibration

H = total thickness of the layer.

$\lambda_n = \dfrac{c}{2\rho\omega_n}$ = damping ration.

$R_n = \left[(\beta_n/2)^{1+b}\Gamma(1-b)J_{1-b}(\beta_n)R\right]^{-1}$

Γ = gamma function.

b,θ = constant related to p by :

$p^{\theta} - \theta + 2b = 0$

$p^{\theta} + 2\theta + 2 = 0$

It should be noted that the above solution is obtained only when $p \leq 0.5$. Eq. 13 defines the mode shapes of the system during the n^{th} mode of vibration whose circular frequency is ω_n $X_n(t)$, may be computed from Eq. 14 using iteration or numerical procedures [7].

After having obtained the values of $Y_n(y)$ and $X_n(t)$, the relative displacement $u(y,t)$ of Eq. 12 is calculated. The relative velocity, the relative acceleration and strain at any depth y at any instant of time can be determined by appropriate differentiation of Eq. 12. A computer programme has been developed by Idriss and Seed to solve problems that are of the type discussed above [7].

Lumped-Mass Solution for Horizo Layered Soils

If the soil deposit is layered, the above equations do not apply. An alternative method has been worked out by Idriss and Seed [7]. Here the soil deposit is represented by a series of layers as shown in Fig 12. The mass of each layer is lumped at the top and bottom of the layer and the masses are interconnected by shear springs that resist lateral deformations. The springs represent the stiffness properties of the material between any two masses. The damping is assumed linearly viscous.

For the soil mass lumped at n levels, there will be n equations of motion which may be represented in a matrix form as follows [7].

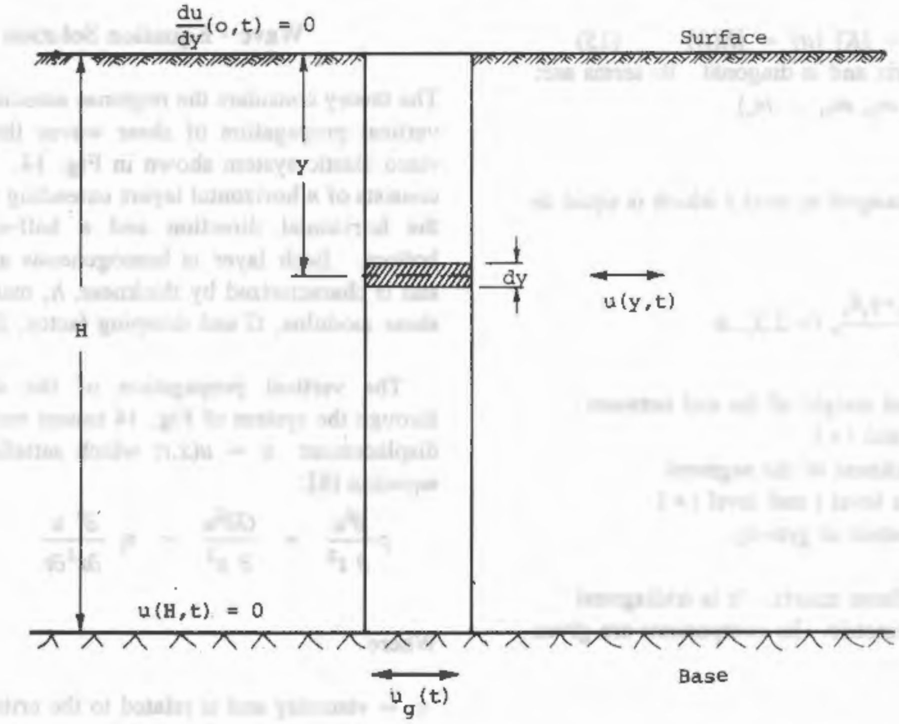


Fig. 11 Semi-infinite soil layer subjected to a horizontal seismic motion at the base, according to Idriss and Seed [7].

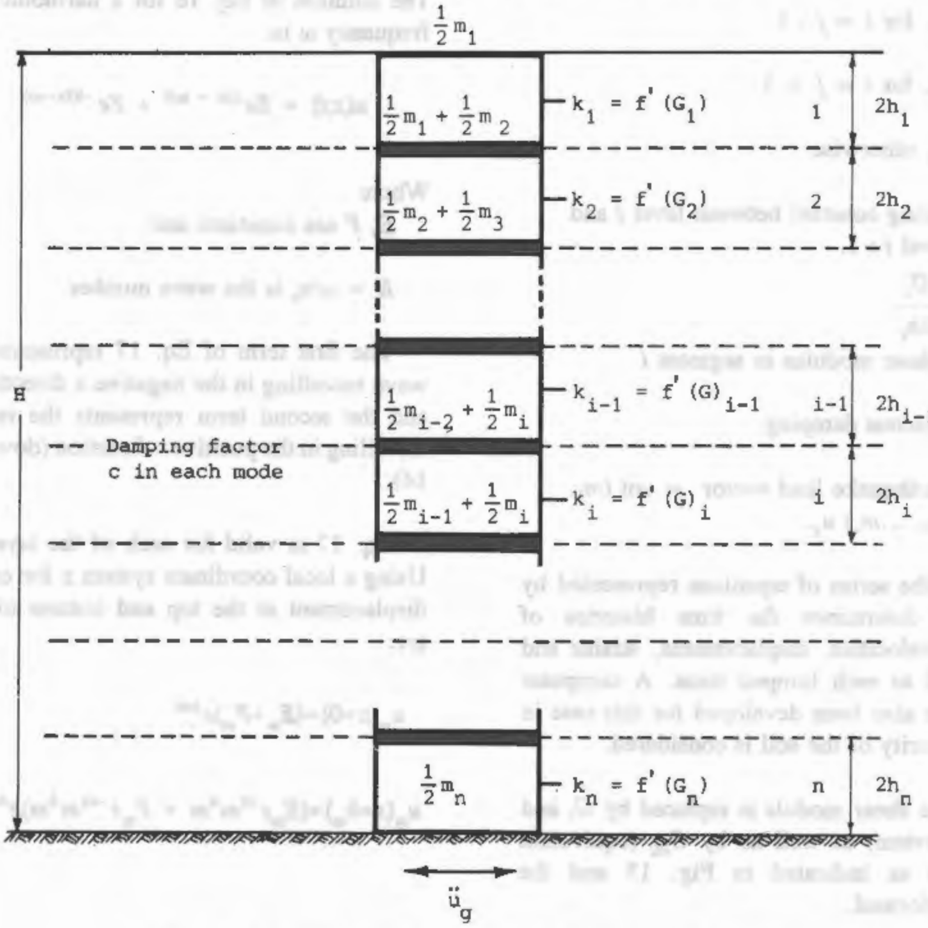


Fig. 12 Lumped-mass representation of layered system [7].

$[M]\{\ddot{u}\} + [C]\dot{u} + [K]\{u\} = \{R(t)\}$ (15)

$[M]$ = mass matrix and is diagonal. Its terms are: diag $[M] = (m_1, m_2, m_3, .. \; m_n)$

Where

m_i is the mass lumped at level i which is equal to

$$m_i = \frac{\gamma_{i-1}h_{i-1}+\gamma_i h_i}{g}, \; i = 2,3,...n$$

γ_i = total unit weight of the soil between level i and $i+1$.

h_i = half thickness of the segment between level i and level $i+1$

g = acceleration of gravity.

$[K]$ = the stiffness matrix. It is tridiagonal and symmetric. Its components are given by:

$K_{11} = k_1$

$K_{ij} = k_{i-1} + k_i$, for $i=j$

$= -k_i$, for $i = j - 1$

$= -k_j$, for $i = j + 1$

$= 0$, otherwise

k_i = spring constant between level i and level $i+1$.

$$= \frac{G_i}{2h_i}$$

G_i = shear modulus in segment i

$[C]$ = viscous damping.

$(R(t))$ = earthquake load vector = col $(m_1, m_2, ...m_n)\, u_g$.

By solving the series of equations represented by Eq. 15 one determines the time histories of accelerations, velocities, displacements, strains and stresses related to each lumped mass. A computer programme has also been developed for this case in which non-linearity of the soil is considered.

The variable shear moduls is replaced by G_1 and G_2 (bilinear system) as well as by G_{EQ} (equivalent linear system) as indicated in Fig. 13 and the calculation performed.

Wave - Equation Solution

The theory considers the response associated with the vertical propagation of shear waves through linear visco elastic system shown in Fig. 14. The system consists of n horizontal layers extending to infinity in the horizontal direction and a half-space as its bottom. Each layer is homogeneous and isotropic and is characterized by thickness, h, mass density, ρ shear modulus, G and damping factor, β.

The vertical propagation of the shear waves through the system of Fig. 14 causes only horizontal displacement $u = u(x,t)$ which satisfies the wave equation [8].

$$\rho\frac{\partial^2 u}{\partial t^2} = \frac{G\partial^2 u}{\partial x^2} = \eta\,\frac{\partial^3 u}{\partial x^2 \partial t} \qquad (16)$$

Where

η = viscosity and is related to the critical damping ratio, β, by relationship $\omega.\eta = 2G\beta$.

The solution of Eq. 16 for a harmonic motion with frequency ω is:

$$u(x,t) = Ee^{i(kx+\omega t)} + Fe^{-i(kx-\omega t)} \qquad (17)$$

Where

E, F are constants and

$K = \omega/v$, is the wave number.

The first term of Eq. 17 represents the incident wave travelling in the negative x direction (upwards) and the second term represents the reflected wave travelling in the positive x direction (downwards)(Fig. 14).

Eq. 17 is valid for each of the layers in Fig.14. Using a local coordinate system x for each layer, the displacement at the top and bottom of the layer m are:

$$u_m(x=0)=(E_m+F_m)e^{i\omega t} \qquad (18)$$

$$u_m(x=h_m)=(E_m e^{ik_m h_m} + F_m e^{-ik_m h_m})e^{i\omega t} \qquad (19)$$

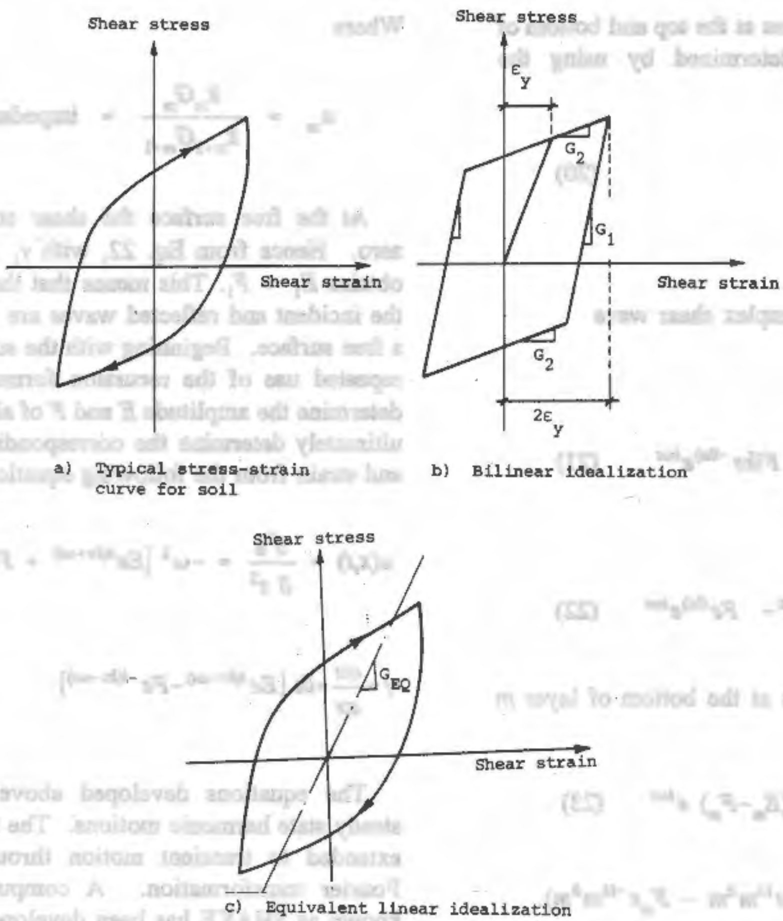


Fig. 13 Idealization of stress-strain behavior of soils

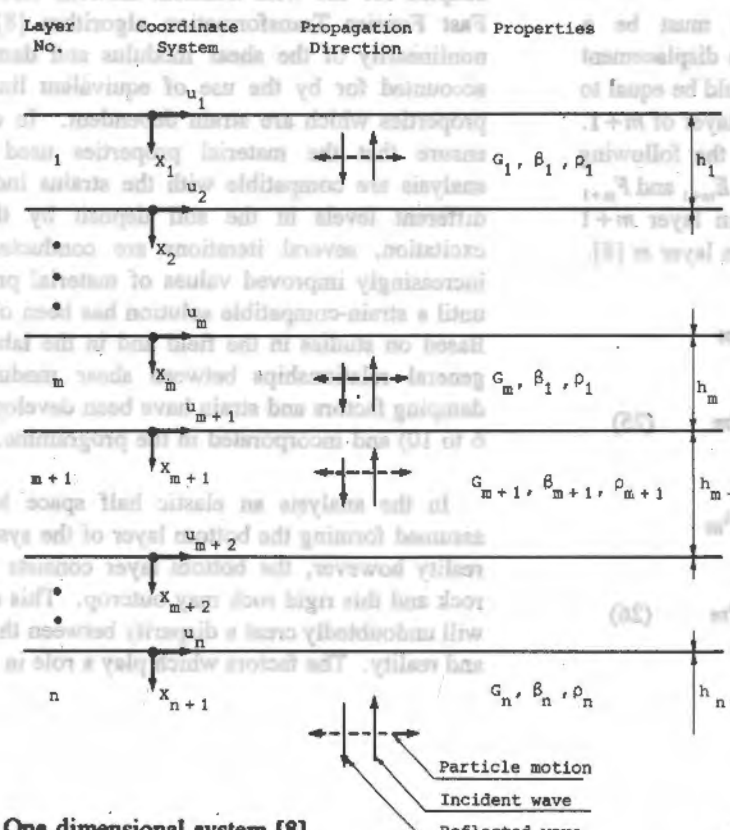


Fig. 14 One dimensional system [8]

Similarly the shear stress at the top and bottom of the layer m may be determined by using the definition:

$$\tau = G^* \frac{\partial u}{\partial x} \qquad (20)$$

Where

$G^* = G(1+2i\beta)$ = complex shear wave

From Eq. 17.

$$\frac{\partial u}{\partial x} = (Eike^{ikx} - Fike^{-ikx})e^{i\omega t} \qquad (21)$$

Hence

$$\tau(x,t) = ikG^*(Ee^{ikx} - Fe^{ikx})e^{i\omega t} \qquad (22)$$

Hence the shear stress at the bottom of layer m are respectively:

$$\tau_m(x=0) = ikG^*(E_m - F_m)\,e^{i\omega t} \qquad (23)$$

$$\tau_m(x=h_m) = ikG^*(E_m e^{ik_m h_m} - F_m e^{-ik_m h_m}).e^{i\omega t} \qquad (24)$$

Since between each layer there must be a displacement and shear continuity, the displacement and shear at the bottom layer of m should be equal to the displacement and shear of the top layer of $m+1$. From this consideration, one obtains the following recursion formulas for the amplitudes. E_{m+1} and F_{m+1} of the incident and reflected wave in layer $m+1$ expressed in terms of the amplitudes in layer m [8].

$$E_{m+1} = \frac{1}{2}E_m(1+\alpha_m)e^{ik_m h_m} + \frac{1}{2}F_m(1+\alpha)e^{-ik_m h_m} \qquad (25)$$

$$F_{m+1} = \frac{1}{2}E_m(1-\alpha_m)e^{ik_m h_m} + \frac{1}{2}F_m(1+\alpha_m)e^{-ik_m h_m} \qquad (26)$$

Where

$$\alpha_m = \frac{k_m G_m}{k_{m+1} G_{m+1}} = \text{impedance ratio.}$$

At the free surface the shear stresses must be zero. Hence from Eq. 22, with $\tau_1 = x_1 = 0$, one obtains $E_1 = F_1$. This means that the amplitudes of the incident and reflected waves are always equal at a free surface. Beginning with the surface layer and repeated use of the recursion formulas one would determine the amplitude E and F of all the layers and ultimately determine the corresponding acceleration and strain from the following equations [8].

$$u(x,t) = \frac{\partial^2 u}{\partial t^2} = -\omega^2\left[Ee^{i(kx+\omega t)} + Fe^{-i(kx-\omega t)}\right] \qquad (27)$$

$$\gamma = \frac{\partial u}{\partial x} = ik\left[Ee^{i(kx+\omega t)} - Fe^{-i(kx-\omega t)}\right] \qquad (28)$$

The equations developed above are valid for steady state harmonic motions. The theory has been extended to transient motion through the use of Fourier transformation. A computer programme known as SHAKE has been developed by a team at the university of California, Berkeley, which is based on the continuous solution of the wave equation adapted for use with transient motions through the Fast Fourier Transformation algorithm [8]. The nonlinearity of the shear modulus and damping is accounted for by the use of equivalent linear soil properties which are strain dependent. In order to ensure that the material properties used in the analysis are compatible with the strains induced at different levels in the soil deposit by the base excitation, several iterations are conducted using increasingly improved values of material properties until a strain-compatible solution has been obtained. Based on studies in the field and in the laboratory, general relationships between shear modulus and damping factors and strain have been developed (Fig 6 to 10) and incorporated in the programme.

In the analysis an elastic half space has been assumed forming the bottom layer of the system. In reality however, the bottom layer consists of rigid rock and this rigid rock may outcrop. This situation will undoubtedly creat a disparity between the theory and reality. The factors which play a role in creating this disparity are discussed by the authors and their effects is taken into consideration [8].

The programme SHAKE is the one that is widely used to assess ground responses induced by earthquake exitations.

The programme performs the following set of operations:

1. Read the input motion, find the maximum acceleration, scale the values up or down, and compute the predominant period.

2. Read data for the soil deposit and compute the fundamental period of the deposit.

3. compute the maximum stresses and strains in the middle of each sublayer and obtain new values for modulus and damping compatible with a specified percentage of the maximum strain.

4. Compute new motions at the top of any sublayer inside the system or outcropping from the system.

5. Print, plot and punch the motions developed at the top of any sublayer.

6. Plot Fourier Spectra for the motions.

7. Compute, print and plot response spectra for motions.

8. Compute print and plot the amplification function between any two sublayers.

9. Increase or decrease the time interval without changing the predominant period or duration of the record.

10. Set a computed motion as a new object motion. Change the acceleration level and predominant period of the object motion.

11. Compute, print and plot the stress or strain time-history in the middle of any sublayer.

These operations are performed by exercising the various available options in the programme.

EMPERICAL METHOD

As indicated earlier, the response spectrum is the most significant characteristics of an earthquake motion and plays an important role in the assessment of the influence of local soil conditions. The shape of the response spectrum is highly influenced by the soil conditions.

The design spectral shape of a site is obtained by first determining the normalized acceleration response spectrum. The normalized acceleration response spectrum is obtained by expressing the ordinate as aproportion of the maximum ground acceleration for the motion for which the spectrum was derived or the zero period spectral values as shown in Fig. 15.

As an alternative to the analytical approach of assessing the response of soils to earthquake shaking, a systematic study was conducted on different spectral response shapes of various site conditions for which complete records of actual earthquake were available, with the objective of establishing a relationship between spectral shape and local site conditions. A total of 104 horizontal ground motion records out of which: 28 for rock sites, 31 for stiff soil sites, 30 for deep cohesionless sites and 15 for sites with soft to medium clay and sand were statistically analized [9]. The study was limited to spectra determined for 5% damping.

The analysis revealed that the spectral shapes were different for the 4 categories of site conditions. The mean spectra of different site conditions are compared in Fig. 16 and the 84-percentile spectra are compared in Fig. 17. It is desirable that additional records be included to all categories especially to "soft to medium clay and sand sites" in order to increase the releability of the statistical analysis. The validity of the statistically deduced curves was checked with other spectral curves proposed hy researchers for the four categories of site conditions. A reasonable degree of agreement was observed for all cases [9].

For the deep cohesionless site, it was felt that if exposed to high intesity of shaking, the spectra may be influenced. To check this, the authors conducted a separate analysis by considering only those records (6 records) that had maximum acceleration greater than 0.3g. The mean spectrum shape for these six records was compared with mean spectrum shape for all the records (Fig. 18). From the figure it appears

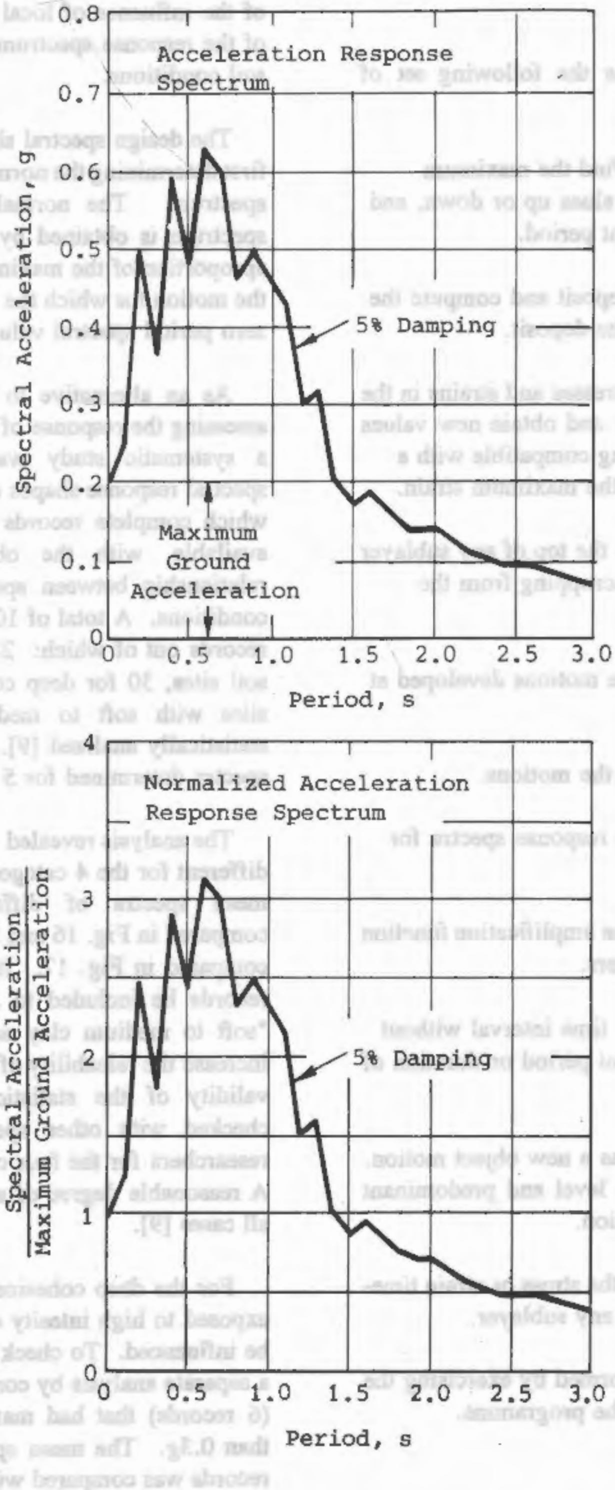


Fig. 15 Determination of normalized acceleration response spectrum [1]

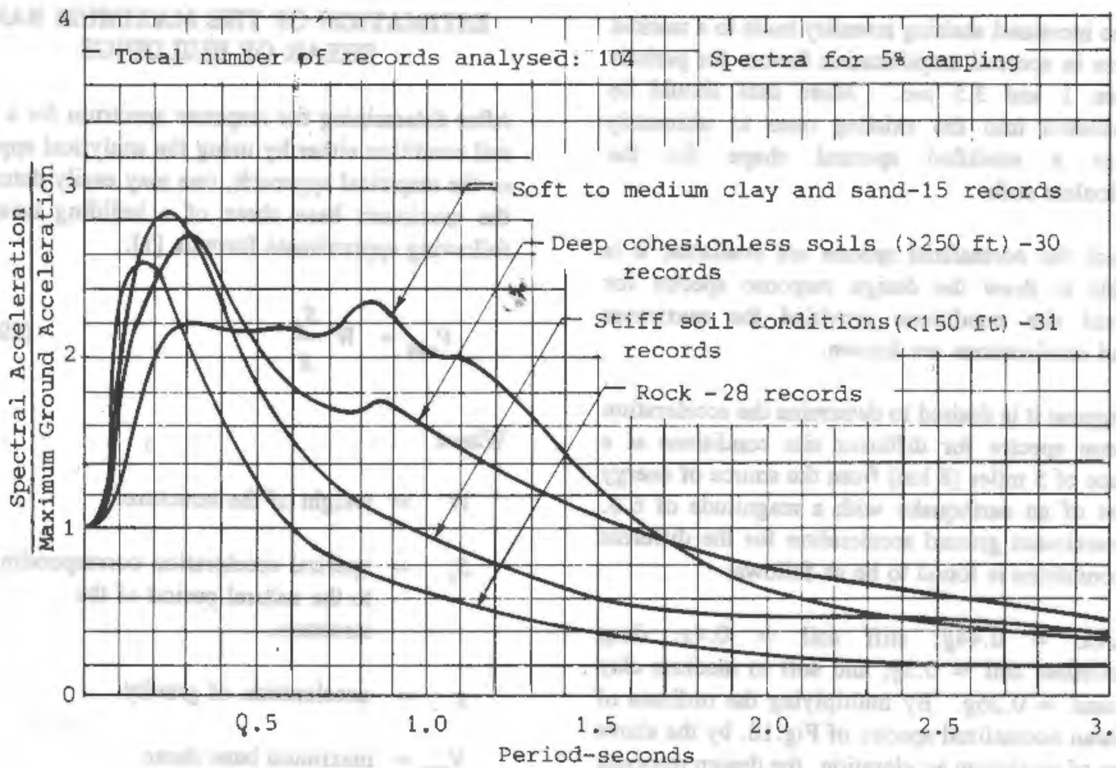


Fig. 16 Average acceleration spectra for different site conditions [9]

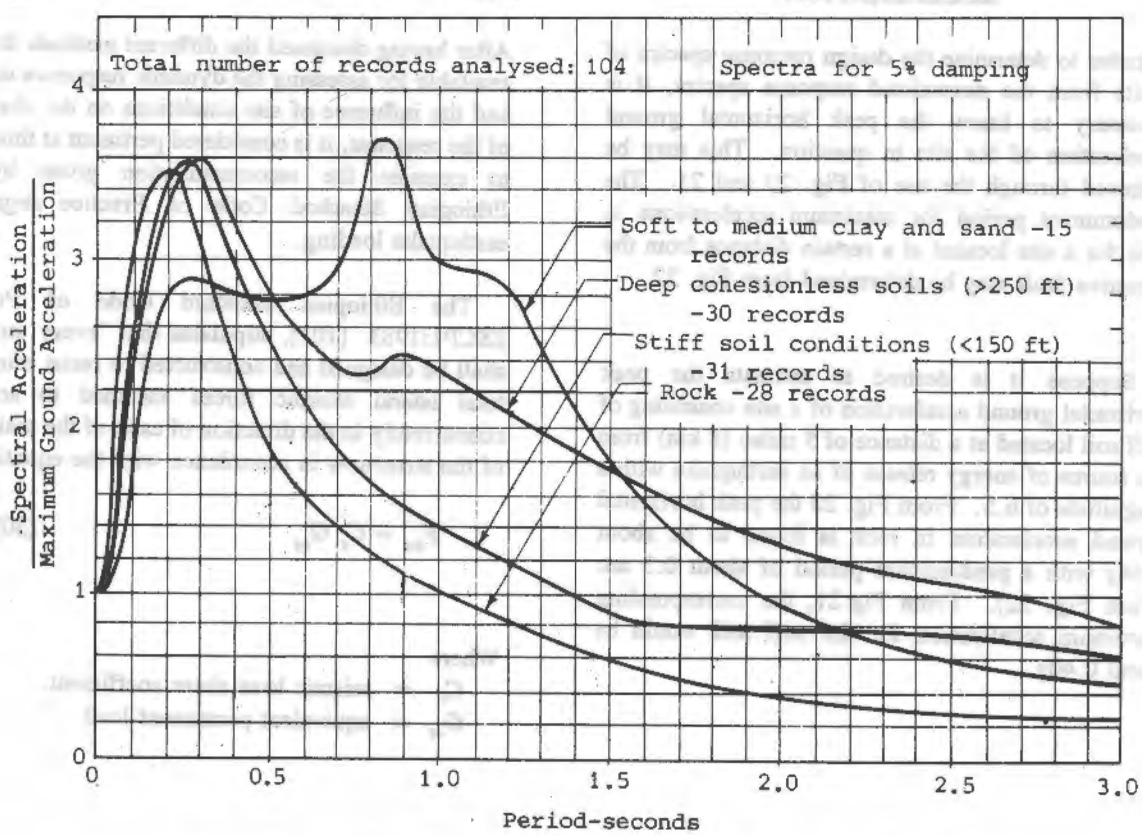


Fig. 17 Percentile acceleration spectra for different site conditions [9]

that the increased shaking intensity leads to a marked increase in spectral amplification factors for periods between 1 and 3.5 sec. More data should be incoprorated into the existing ones to ultimately propose a modified spectral shape for the cohesionless soils.

Once the normalized spectra are available, it is possible to draw the design response spectra for different site conditions provided the maximum ground accelerations are known.

Suppose it is desired to determine the acceleration response spectra for different site conditions at a distance of 5 miles (8 km) from the source of energy release of an earthquake with a magnitude of 6.5. The maximum ground acceleration for the different site conditions is found to be as follows:

Rock = $0.44g$; stiff soil = $0.4g$; deep cohesionless soil = $0.3g$; and soft to medium clay and sand = $0.26g$. By multiplying the ordinate of the mean normalized spectra of Fig.16. by the above values of maximum acceleration, the design response spectra are obtained (Fig.19).

ESTIMATION OF PEAK HORIZONTAL ACCELERATION

In order to determine the design response spectra of a site from the normalized response spectra, it is necessary to know the peak horizontal ground acceleration of the site in question. This may be achieved through the use of Fig. 20 and 21. The predominant period for maximum accelerations in rock for a site located at a certain distance from the causative fault may be determined from Fig. 22.

Suppose it is desired to estimate the peak horizontal ground acceleration of a site consisting of stiff soil located at a distance of 5 miles (8 km) from the source of energy release of an earthquake with a magnitude of 6.5. From Fig. 20 the peak horizontal ground acceleration in rock is found to be about $0.44g$ with a predominant period of about 0.3 sec (from Fig. 22). From Fig.21, the corresponding maximum acceleration for the stiff soil would be about $0.40g$.

ESTIMATION OF THE MAXIMUM BASE SHEAR OF BUILDINGS

After determining the response spectrum for a given soil condition either by using the analytical approach or the empirical approach, one may easily determine the maximum base shear of a building using the following approximate formula [1].

$$V_{max} = W \frac{S_A}{g} \qquad (29)$$

Where

W = weight of the structure.

S_a = spectral acceleration corresponding to the natural period of the structure.

g = acceleration of gravity.

V_{max} = maximum base shear.

EARTHQUAKE LOADING ACCORDING TO ETHIOPIAN STANDARD CODE OF PRACTICE

After having discussed the different methods that are available for assessing the dynamic responses of soils and the influence of site conditions on the character of the response, it is considered pertinent at this stage to examine the recommendation given by the Ethiopian Standard Code of Practice regrading earthquake loading.

The Ethiopian Standard Code of Practice ESCP1:1983 [101], stipulates that "every structure shall be designed and constructed to resist minimum total lateral seismic forces assumed to act non concurrently in the direction of each of the main axes of the structures in accordance with the equation".

$$F_{tot} = C_s\, G_{eq} \qquad (30)$$

Where

C_s = seismic base shear coefficient.

G_{eq} = equivalent permanent load

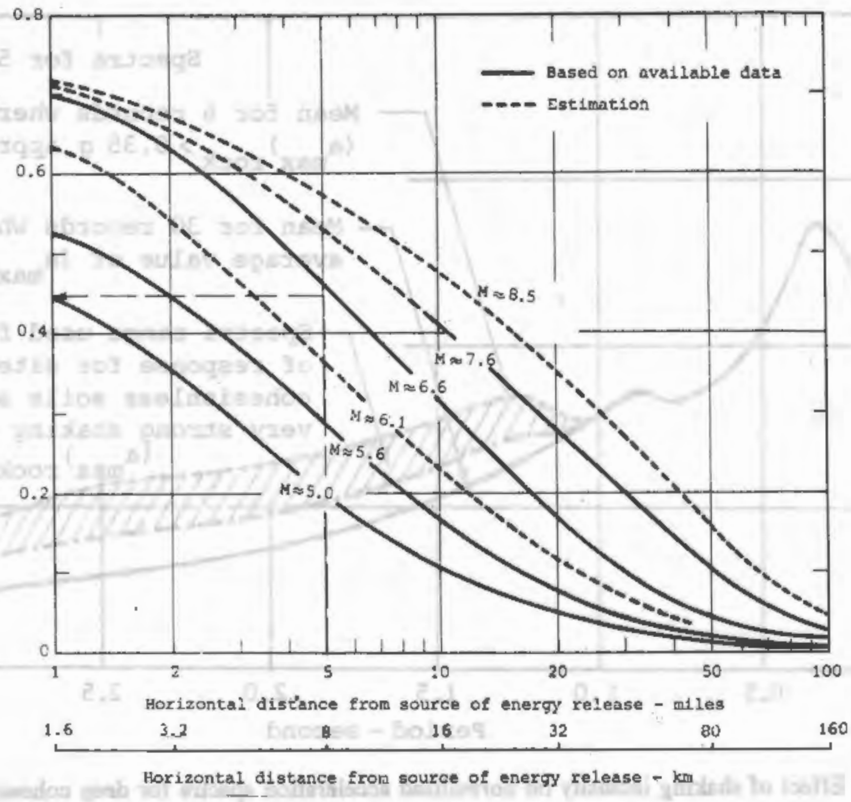


Fig. 20 Average values of maximum accelerations in rock [1]

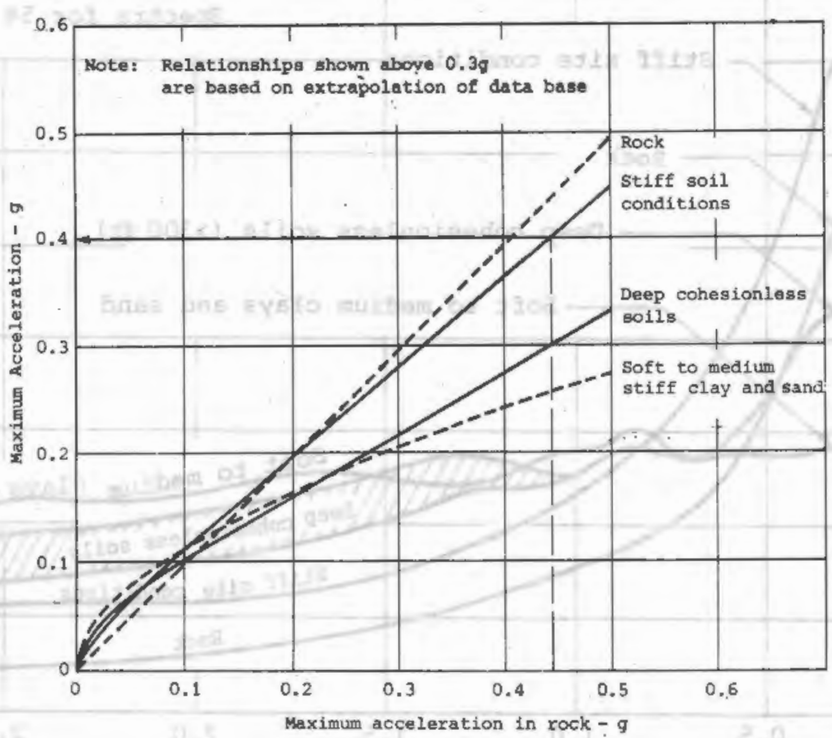


Fig. 21 Approximate relationships between maximum accelerations on rock and other local site conditions [1]

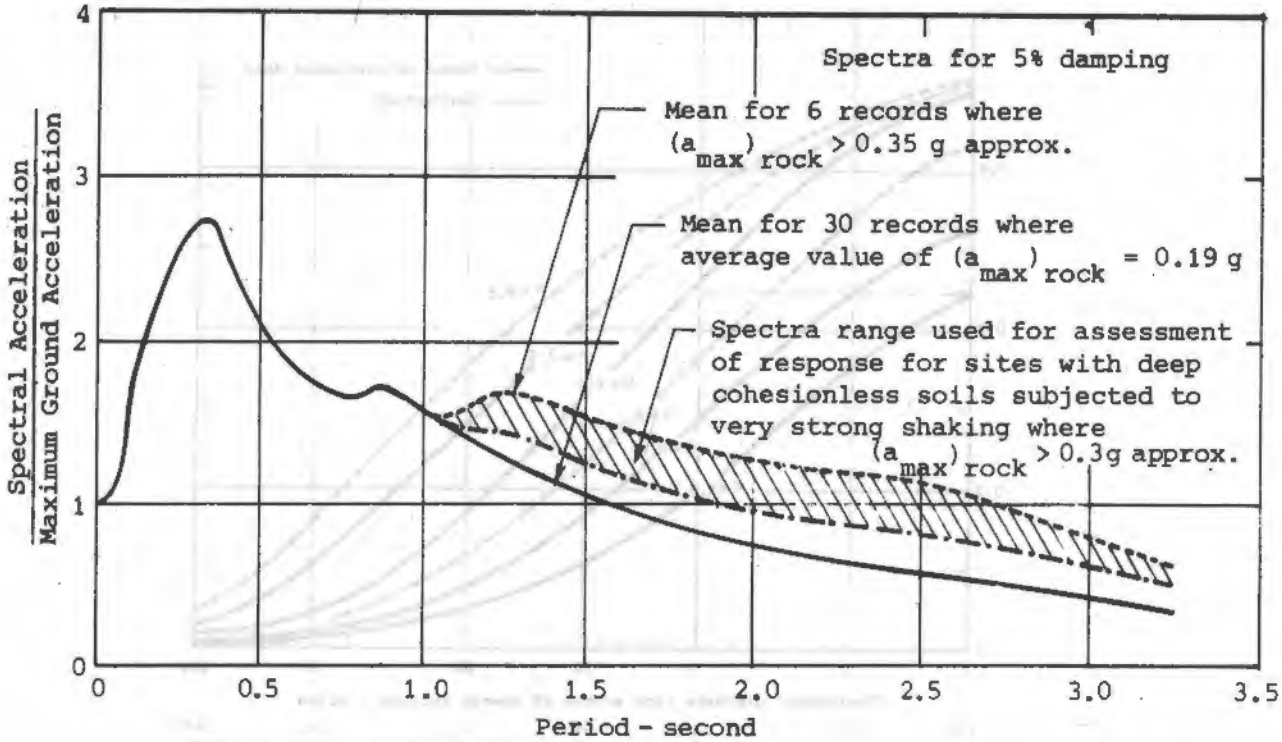


Fig. 18 Effect of shaking intensity on normalised acceleration spectra for deep cohesionless soil sites [9]

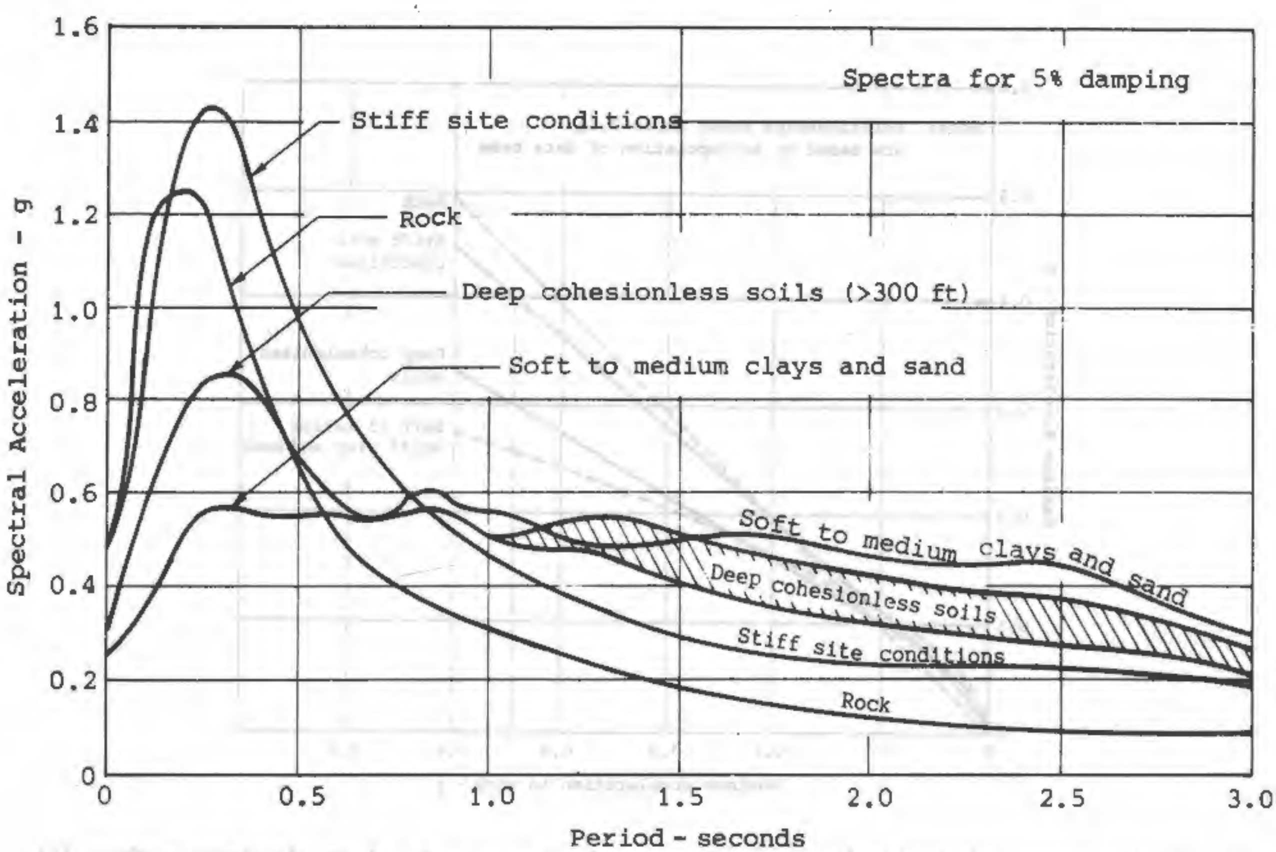


Fig. 19 Anticipated mean spectra for magnitude 6 ½ earthquake at distance of 5 miles [9]

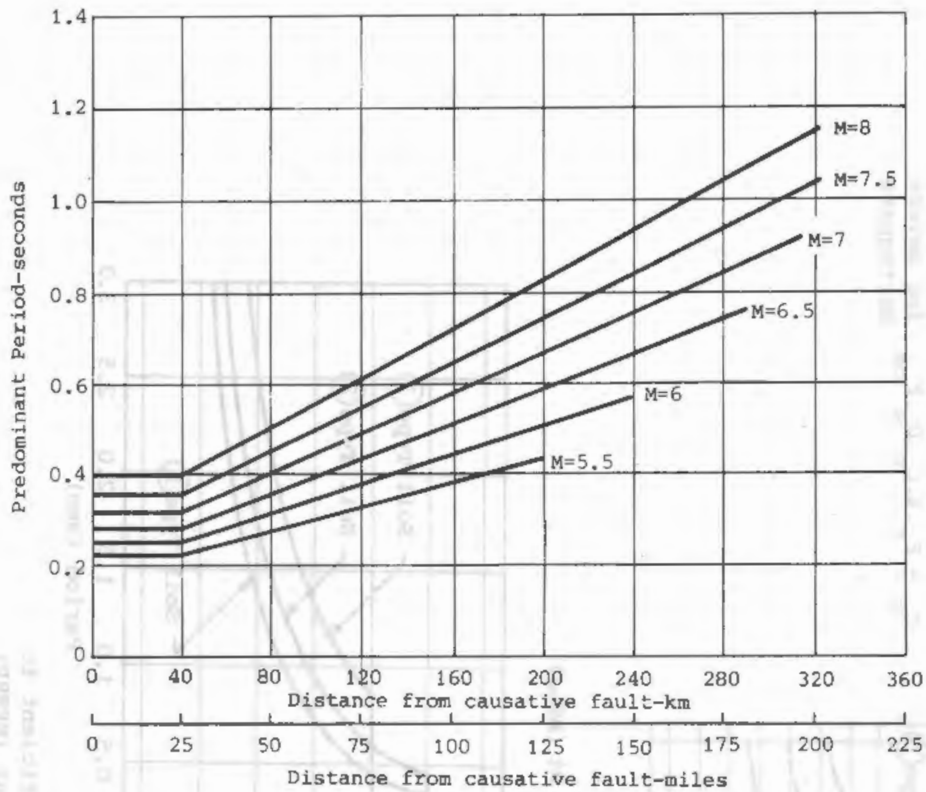


Fig. 22 Predominant periods for maximum accelerations in rock [1]

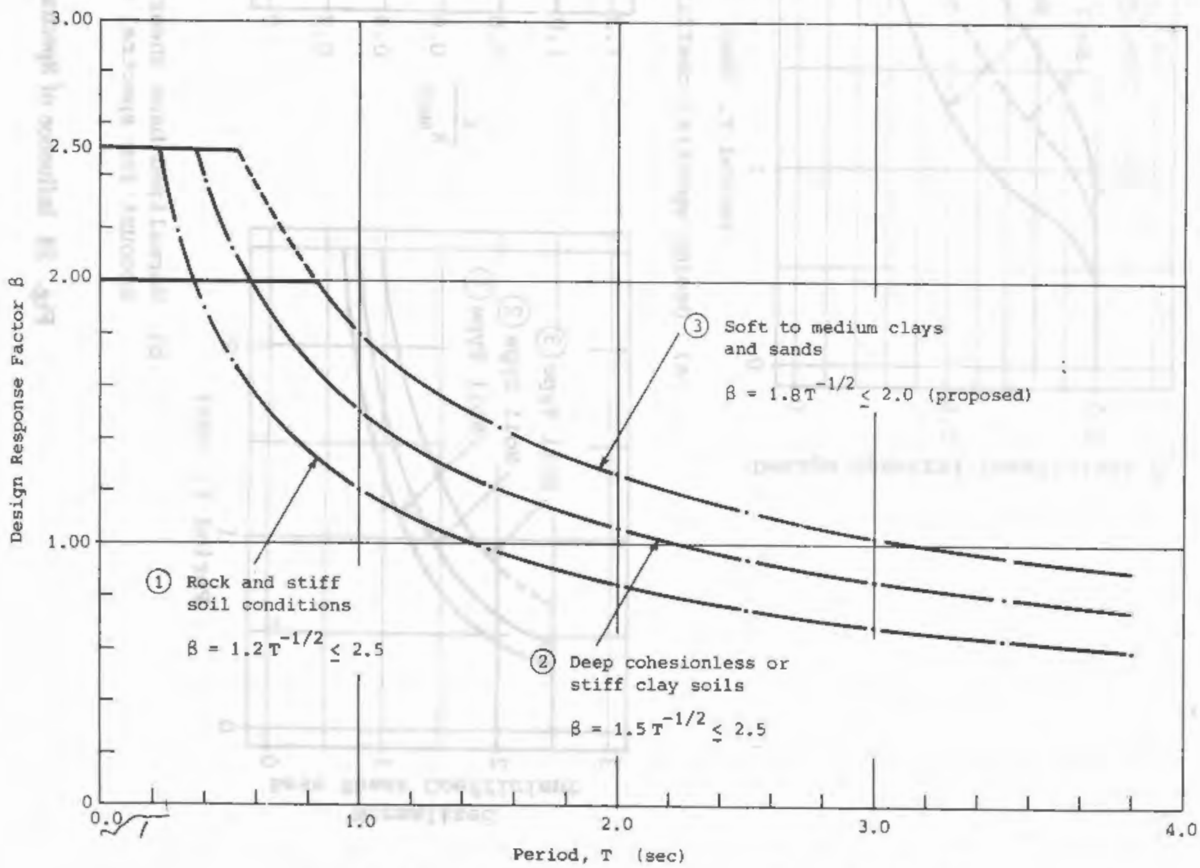
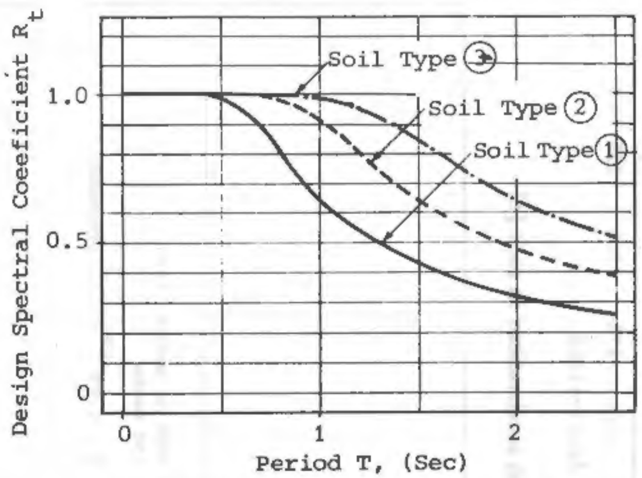


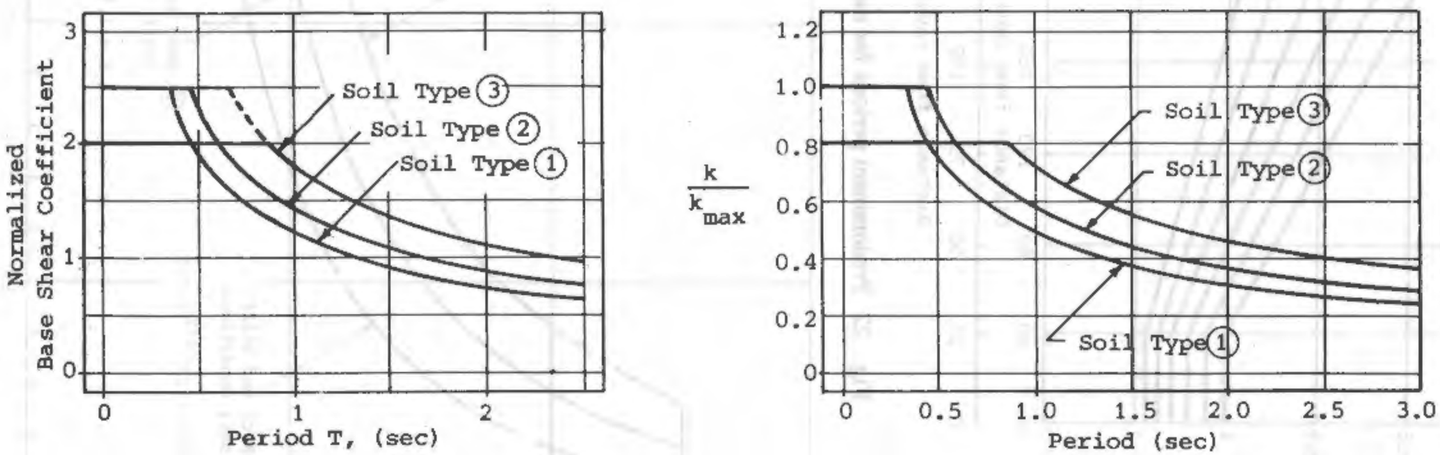
Fig. 23 Design responses factor according to ESCP 1:83



a) Design spectral coefficient Rt (BSLJ)

$C_s = R_t.Z.C_o$; for moderate earthquake

$C_s = F_t.Z.C_o.D_s.F_{cs}$; for severe Earthquake



b) Normalized base shear coefficient to account for spectral content (NEHRP)

Fig. 24 Influence of Spectral Content

The seismic base shear coefficient is given by

$$C_s = \alpha \beta \gamma \quad (31)$$

Where

$\alpha = \alpha_o I$

$\beta = \beta_o S$

α_o = bedrock acceleration ratio for the site and depends on the seismic zoning.

I = importance factor.

β_o = elastic design response spectrum factor for bedrock A foundstion and standard damping of 5% and determined from:

$\beta_o = (1.2)\ T^{0.5}$, where T is the period.

S = soil classification and site condition factor.

γ = structural system type factor.

Within the framework of the present discussion, attention is given exclusively to the parameter β even though the seismic zoning map that is currently being used in design leaves much to be desired. The design response factor may be expressed by the following equations for the given three soil types.

(i) For rock and stiff conditions (soil Type 1)

$$\beta = 1.2(T)^{-1/2} \leq 2.5 \quad (32)$$

(ii) For deep cohesioniess or stiff clay soils (soil Type 2)

$$\beta = 1.5(T)^{-1/2} \leq 2.5 \quad (34)$$

(iii) For soft to medium clays and sands (soil Type 3)

$$\beta = 1.8(T)^{-1/2} \leq 2.5 \quad (34)$$

A plot of β versus T (Fig. 23) for the three soil types indicates a general similarity with the Building Standard Law of Japan (BSLJ) and the National Earthquake Hazard Reduction Project (NEHRP) of the United States of America [11], who have had many years of experience in asesmic design of structures.

According to BSLJ, the coefficient R_t is assigned a constant value of 1.0 for a period up to 0.4 sec and 0.8 sec for soil Types 1, 2 and 3 respectively (Fig. 24 a). NEHRP on the other hand assigns a constant normalized base shear coefficient of 2.0 for a period up to 0.9 sec for soil Type 3 and a constant normalized base shear coefficient of 2.5. for a period up to 0.6 sec for soil Type 2 and up to 0.5 sec for soil Type 1 (Fig. 24 b).

The spectral curves of Fig.24 are derived from the statistically deduced average spectra of different soils presented in Fig. 16. From Fig. 16 it is apparent that the variation of soil Type 3 is different from the other soil types. Its normalized peak acceleration is lower than the other soil types. The normalized peak accelerations of the two soil types drop quickly in comparison with soil Type 3. In fact the normalized peak acceleration of soil Type 3 remains constant till about 1 sec.

Hence the proposal of ESCP1:83 of assigning an equal maximum β-value for all the soil types does not conform with the real situation. It would therefore be appropriate to assign a different β- value for soil Type 3 in conformity with NRHRP. It is recommended to use $\beta = 2$ up to a period of 0.9 sec for this soil as indicated in Fig. 23. With this modification the ESCP1:83 recommendstion regarding earthquake loading is comparable with the other codes that have been discussed above.

CONCLUDING REMARKS

From the preceding discussions, it has been possible to present two approaches (one analytical and one empirical) for determining the response spectra of a given site.

Even though the analytical method simplifies the complex wave patterns that develop during an earthquake to one-degree of freedom, experience with this method has demonstrated its usefulness. In fact the dynamic response analysis highlights details of ground response characteristics of a given site more than the empirical method, where these characteristics are defused by the statistical averaging techniques.

Due to its simplicity, there is a tendency to use exclusively the empirical curves rather than performing the dynamic response analysis. It should be stated however that before the analytical method is completely replaced by the empirical curves, additional earthquake records should be incorporated in the existing data and new evaluation should be made. The new spectral curves should of course be incorporated in the codes. As things stand at present, there is still a need to use both methods in order to arrive at reliable design spectra.

In order to estimate the base shear force of a building, one may either use the spectral acceleration derived from the analytical method (e.g SHAKE) and apply Eq. 29, or directly use the appropriate Curves of Fig. 23 or the equivalent equations described in ESCP1:83. The result thus obtained should be compared.

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