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Earthquake Hazard Reduction in the Central and Eastern United States:
A Time for Examination and Action

CONSIDERATION OF LOCAL SOIL CONDITIONS FOR BUILDING SITING

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ABSTRACT

Depending upon the area seismicity, site conditions and the nature of the proposed construction, a geotechnical site investigation may be necessary to allow adequate definition of seismic hazard and related structural design criteria. In any event, the structural engineer and the geotechnical engineer must effectively cooperate in order to ensure that the building code provisions are adequately extended to effectively address existing geological hazards and potential for aggravated soil amplification of ground motion or ground failure. This paper provides a summary description of two of the problems introduced by the geotechnical site conditions (liquefaction and resonance) and presents evaluation methods. It is important that the structural engineer realize the importance of the local site conditions on structural response.

INTRODUCTION.

Seismic building code provisions have been prepared to provide minimum required resistance to typical earthquake ground shaking While current codes address soil conditions generally, special conditions may require seismic building code minimum requirements to be appropriately increased by the designer according to his professional judgement. Thus, in establishing structural design criteria to mitigate the damage potential of earthquakes, especially for schools and other essential facilities, it is important to understand special analysis methods for evaluation these design criteria. This paper examines two such soil effects - resonance and liquefaction.

The site period is that period at which the ground resonates. When the predominant frequency of earthquake motions matches the site period, resonance and consequent large ground motions occur. If, in addition, the natural period of the building matches the site period, "double resonance" can occur, resulting in large building deformations and severe damage. Much of the structural damage from the 1985 Mexico City earthquake was caused by double resonance (Rosenblueth, 1986). Mapping dynamic site periods for a city, and governing building construction accordingly, can greatly reduce the threat of damage from earthquakes.

Liquefaction is a phenomenon where a saturated, cohesionless soil is subjected to a loading that causes the pore pressures in the soil to increase, and the effective stresses to decrease to the point where the soil has very little or no strength. When liquefaction is combined with conditions of

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ground slope, surface loads, and the ejection of water and sediments, movements causing severe damage can result. Maps of liquefiable zones can be made to assist in siting structures.

METHODOLOGY FOR AREAL MAPPING OF SITE PERIODS

Purpose. This section describes microzone site period mapping This map gives the areal distribution of natural, small soil strain, site periods of the soil profile. Community engineers can compare this period with building periods to reduce the chance of double resonance.

Site Period Estimation Procedure. Manual calculation of the site period is possible using a method outlined by Dobry et al. (1976). While the method does not account for the nonlinear behavior of the soil under earthquake loadings or for a particular ground motion, the method is rapid to use, and provides sufficient accuracy for small earthquakes (Elton and Martin, 1989). If a soil deposit can be modeled as one layer with constant density and shear wave velocity, subjected to small strains, the site period can be calculated from

$$T = 4H / V_s \tag{1}$$

where T is the site period, H is the depth to bedrock, and V_s is the shear wave velocity of the soil.

Soils strain during earthquake shaking, increasing their flexibility and changing the site period. Hays (1980) proposed modifying equation (1) to account for the strain softening effect:

$$T = 6H / V_s \tag{2}$$

This correction improves the site period prediction. Elton and Martin (1989) compared the results from a non-linear computer program (which accounts for strain softening) with results from equation (1), and found that the differences in calculated site period ranged from 50% to 150% for the cases studied. Thus, Hays' modification appears to come close to the lower bound of site period increase that can be expected from strain softening.

Since it is not likely that the shear modulus or density remain constant with depth, or that the site profile consists of one soil type, a more complicated solution is needed. Dobry et al (1976) present an approximate manual solution that calculates the site period. The inputs are the shear wave velocity, thickness and density of each soil layer. The procedure calculates the period of successive two layer systems. Referring to Figure 1, this is accomplished by

1. dividing the soil profile into layers, and calculating the periods of the top two soil layers A and B using equation (2) applied separately to each layer,

2. entering Figure 1 on the abscissa, and (using the proper curve) selecting the ratio of the two-layer period to the top layer period (T/T_A) from the ordinate, calculate T by multiplying the value read from the ordinate by T_A,

3. consider the combination of the top two soil layers as one layer with period T, and repeat the above process for layer C and the new top layer, again using Figure 1. The soil density (ρ) of new top layer is now the weighted average of the soil densities of the original top two layers. The shear wave velocity of new top layer is now the weighted average of the shear wave velocities of the original top two layers. The weighting calculation is done according to

equation (3) below. The weighting factors are the thicknesses of the respective layers. For layers A and B,

$$\rho_{AVG} = \frac{(H_A)(\rho_A) + (H_B)(\rho_B)}{(H_A) + (H_B)} \tag{3}$$

where H_A and H_B are the thicknesses of layers A and B, respectively (see Figure 1). A similar calculation is made for the average shear wave velocity, substituting shear wave velocity for soil density in equation (3),

4. repeat until the bottom-most soil layer is combined with all the upper soil layers The final period T found from Figure 1 is the period for the entire soil profile.

This approximate procedure is recommended as a rapid method of estimating site specific periods when shear wave velocities applicable to the strain range under consideration are known or can be estimated. Since soil moduli may approach linear behavior at small strains, the site period calculated by this procedure may be applicable to small earthquakes.

The above method does not account for the time history of the earthquake. While it is known that the frequency of the input motion affects the dynamic site period, it does not appear to have a great influence (Elton and Martin, 1989). Their examination of the influence of input earthquake motion reveals that the dynamic site periods are not overly sensitive to frequency content of input motion, provided hard site records are used. The dynamic site period increases with increasing peak ground acceleration.

The shear wave velocity needed above can be determined by either direct measurement using cross-hole tests, or by correlation. If the shear modulus, G, is known or can be estimated by correlation, the shear wave velocity can be calculated from

$$V_s = \left(\frac{G}{\rho}\right)^{0.5} \tag{4}$$

G for clays can be estimated as $2000 \times S_u$, where S_u is the undrained shear strength of the clay. The undrained shear strength can be estimated from SPT data (Bowles, 1982) or determined from laboratory tests on samples. For sands, G can be estimated from

$$G = 1000\ K_2(\bar{\sigma}_m)^{0.5} \tag{5}$$

where K_2 is a dimensionless parameter which is a function of relative density, and $\bar{\sigma}_m$ is the mean effective stress.

Seed and Idriss (1970) have correlated K_2 to relative density and shear strain level. Small earthquakes correspond to the low shear strain value at 10^{-4}% strain (Figure 2). The relative density can be determined from the SPT using the relations presented in by Gibbs and Holtz (1957). In this case, the calculated value of G represents the maximum modulus which is reduced as the shear strain increases. Seed and Idriss (1970) outline modulus reduction curves for sands if high strains are expected.

Use of the Site Period, T_s. There are two uses of site periods. The first is to make a map of site periods to aid in zonation. This is done by applying the above procedures to enough boring logs to define the soil profile throughout the study area. The site periods for each location are calculated and plotted on a map of the area and then divided into zones of ranges of site periods.

The second use of site periods is in conjunction with natural building period to determine an appropriate site-structure resonance factor for base shear evaluation. With knowledge of the building period and site period, the S factor in the Uniform Building Code (1976) base shear equation can be determined and used in the base shear equation:

$$V = Z\,I\,K\,C\,S\,W \tag{6}$$

where V is the design base shear for a structure, and S is the site-structure interaction factor. The other coefficients are defined in the code (UBC, 1976). This base shear should be compared with the base shear calculated using the newer UBC (1988). The larger of the two values should be used.

LIQUEFACTION SUSCEPTIBILITY METHODOLOGY

Current technology allows the geotechnical engineer to determine soil liquefaction susceptibility at a given site. This section describes a methodology for determining the liquefaction susceptibility at a site, and creating a liquefaction susceptibility map, which is useful in planning studies.

Determination at a Single Site. The method used develop ground failure susceptibility makes use of the empirical method developed by Seed and his co-workers (1981, 1983, 1986). The method produces accelerations used as the measure of liquefaction susceptibility. The method characterizes the stress state of the soil by the ratio of the average earthquake-induced shear stress to the effective confining pressure, the cyclic stress ratio (CSR), which is obtained from:

$$CSR = \frac{\tau}{\bar{\sigma}_0} = 0.65\ \frac{\sigma_0}{\bar{\sigma}_0}\ a_{\max}\ r_d \tag{7}$$

where τ is the cyclic shear stress, σ is the total overburden pressure, $\bar{\sigma}$ the effective overburden pressure, a_{max} the maximum peak ground acceleration as a percentage of the acceleration due to gravity, and r_d the reduction factor for soil flexibility for depths d below the ground surface in meters. Iwasaki et al (1981) calculate r_d using:

$$r_d = 1 - 0.015d \tag{8}$$

Liquefaction is likely to occur if the CSR exceeds the ratio of the cyclic *strength* to effective overburden pressure - the critical cyclic stress ratio (CCSR). The CCSR required to cause liquefaction can be evaluated from laboratory tests on soil samples or by empirical relationships with in situ tests, such as the Standard Penetration Test (SPT). In situ tests are preferred because of sampling difficulties for laboratory tests.

Seed and his co-workers plotted SPT data versus CSR for sites that did and did not liquefy during earthquakes of magnitude 7.5. A curve was drawn on the plot separating sites that liquefied (left of the curve) and sites that did not liquefy (right of the curve). Figure 3 shows this data (Seed and De Alba, 1986). The fines content (percentage of particles passing through a no. 200 sieve) affects liquefaction susceptibility, and can be accounted for using the appropriate curve in Figure 3. The resulting CSR represents the cyclic strength of the soil. Different magnitude earthquakes can be accounted for using the correction factors presented in Table 1. The CSR increases with decreasing magnitude.

TABLE 1. Correction Factor for Influence of Earthquake Magnitude on Liquefaction Resistance (After Seed and De Alba, 1986)

Earthquake Magnitude	Correction Factor	Number of Representative cycles at 0.65 S_{max}
5.25	1.50	2 - 3
6.00	1.32	5 - 6
6.75	1.13	10
7.50	1.00	15
8.50	0.89	26

The SPT N-values used in Figure 3 are the field values corrected for overburden pressure as described by Marcuson and Bieganousky (1977) and for the energy ratio of the SPT hammer as described by Seed and De Alba (1986).

The curves shown in Figure 3 may be interpreted as the critical state for a given site and magnitude. This critical cyclic stress ratio is read from the graph as a function of magnitude M, fines content f, and N-value:

$$CCSR = h(N,M,f) \tag{9}$$

Equations (7) and (9) can then be and solved to yield the critical acceleration (a_c) required to cause liquefaction for a given magnitude, N-value and vertical stresses:

$$a_c = \frac{1}{0.65 r_d} \frac{\bar{\sigma}_0}{\sigma_0} h(N,M,f) \tag{10}$$

These a_c can be interpreted as a measure of the liquefaction susceptibility.

Soil properties. Several soil properties are needed to evaluate equation (10). Soil boring logs can provide the soil type, N-value, and the water table location. The soil type is needed to determine if the soil is cohesionless, and thus potentially susceptible to liquefaction. While the GSD is desirable, the soil classification may be sufficient to make this determination. The N-value is needed to be able to use Figure 3, and to estimate the density of the soil, which is used in the calculation of σ_0 and $\bar{\sigma}_0$. Table 2 shows a correlation between N-value and soil unit weight. Soil density is unit weight divided by the acceleration due to gravity. The water table location is used in calculating $\bar{\sigma}_0$.

When performing these calculations for an area mapping exercise, borings relatively close to one another can be combined into composite boring logs based on similarity of soil types and soil location in the profile. In any given borehole, potentially liquefiable soil layers should be identified and analyzed separately. Typically, only layers within 30 ft. (10m) of the ground surface are analyzed, since the overburden stress makes liquefaction unlikely beyond this depth.

Boring logs may be obtained from local building owners, government offices and utilities.

Once the magnitude of earthquake is selected, the corrected N-value determined, the total and effective overburden stresses calculated and the flexibility correction (r_d) found, the acceleration required to cause liquefaction can be calculated from equation 10.

TABLE 2. Relationship Between SPT Values and Density (after Bowles, 1982)

Cohesionless soils		Cohesive soils	
N-value	unit weight (pcf)	N-value	unit weight (pcf)
5 - 10	80 - 110	2	100 - 120
8 - 15	90 - 130	4 - 8	110 - 130
10 - 40	130 - 140	16 - 32	120 - 140
20 - 70	140 - 150		

Alternatively, equation (7) can be used. This is done when it is desired to know if a given site will liquefy in a given ground acceleration. In this instance, the expected acceleration is estimated from seismic data, the total and effective overburden stresses calculated and the flexibility correction (r_d) is found. Equation (7) is then used to calculate the cyclic stress ratio that the given acceleration will produce and then compared with the CSR from Figure 3. If the latter value is greater than that from equation (7), the soil will not liquefy.

Next the corrected N-value $(N_1)_{60}$ determined, as noted above. Figure 3 is entered from the abscissa for the appropriate percentage of fines in the soil, and the cyclic stress ratio that the soil can withstand without liquefaction is read from the ordinate. This ratio is compared with the ratio determined from equation (7). If the ratio from equation (7) is larger than that from Figure 3, liquefaction is likely.

Areal determination of liquefaction susceptibility. Liquefaction susceptibility mapping quantifies the soil's ability to liquefy for a given earthquake excitation. The susceptibility is a function of the soil type, soil density, and location of the water table, and expected earthquake magnitude.

The methodology for mapping areal liquefaction susceptibility begins with defining the areal soil conditions. This can be done with either a sufficient number of soil borings, or with a combination of soil borings and geologic maps that show near-surface soil formations.

Use of borings exclusively. If enough soil borings are available, these borings can be analyzed individually, plotted, and contoured to create a liquefaction susceptibility map.

The procedure for determining the liquefaction susceptibility of the individual boreholes (or composite boreholes) is the same as that given above for local liquefaction determination - a magnitude earthquake is selected, the cyclic stress ratio corresponding to that earthquake and N-value is selected, and that cyclic stress ratio is used in equation (10) to estimate the acceleration needed to cause liquefaction.

Once the liquefaction susceptibility of each borehole is known, in terms of the acceleration needed to cause liquefaction, these accelerations can be plotted and contoured on a map. The different zones of accelerations required to cause liquefaction indicate different liquefaction susceptibilities (higher acceleration zones indicates a reduced susceptibility to liquefaction, and vice versa). Elton and Hadj-Hamou (1990) present such a map of Charleston, SC.

Use of geologic maps in combination with borings. If an insufficient number of borings to define the soil conditions are available, it is desirable to make the liquefaction susceptibility map based on a combination of data from soil borings and geologic maps. The intention is to characterize the various geologic formations comprising the study area with geotechnical borings. The borings should indicate water table information, and give good indication of the soil type. The N-values of the soil are needed in order to determine the acceleration needed to cause the soil liquefy for a given magnitude earthquake. Youd and Perkins (1978) provide a table that can assist in the use of geologic maps. They categorize the liquefaction susceptibility of a wide variety of soil deposits by age and type of deposit. As expected, loose, very young deposits with potentially high water tables are most susceptible. This general type of categorization can greatly assist the engineer in zoning the study area, by matching the type of soils in the study area with the characteristics listed in Youd and Perkins (1978). Nonetheless, the more distinct characterization by N-value is still needed to calculate the acceleration needed to cause liquefaction.

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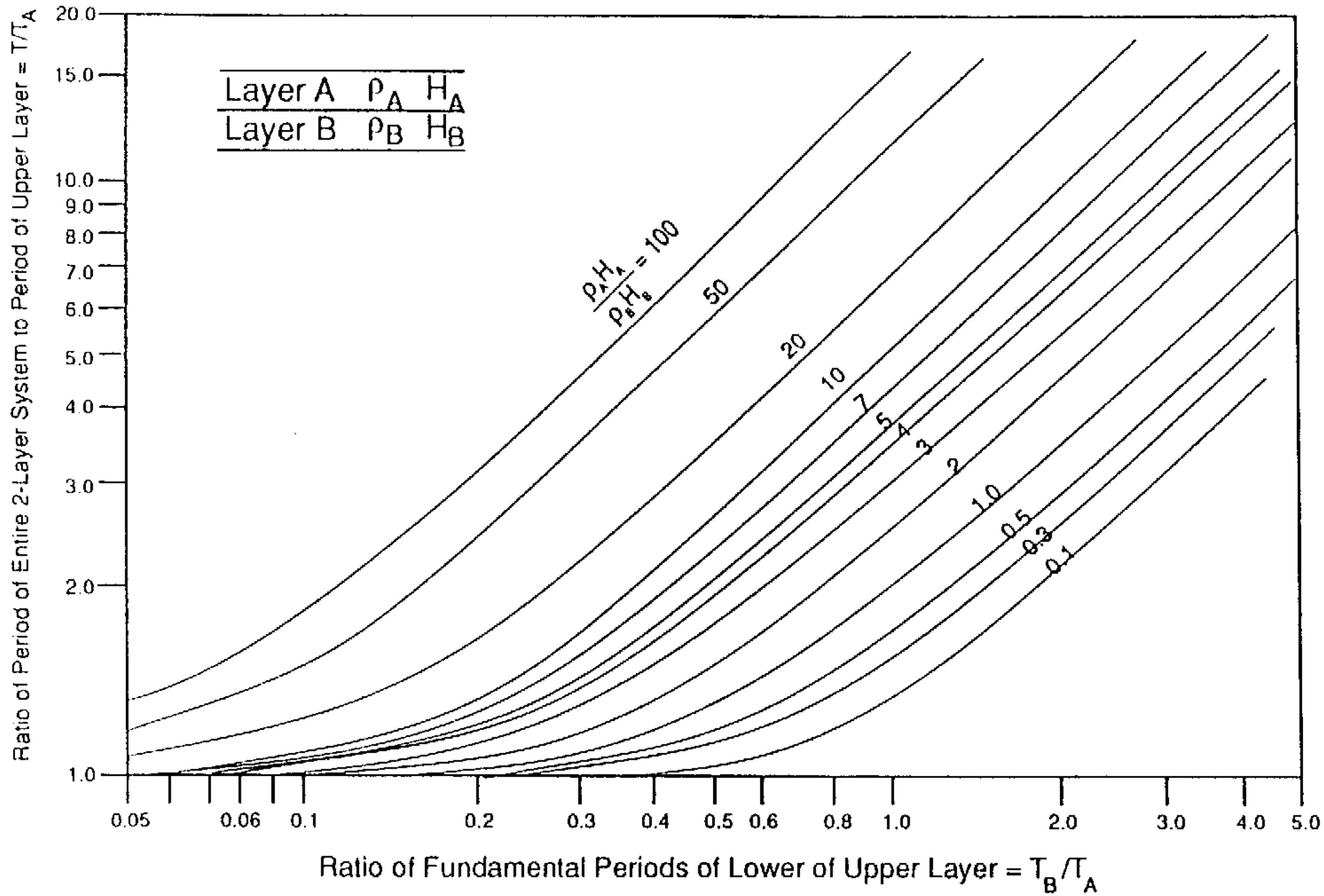

Figure 1 - Chart for use in calculating approximate site periods (from Dobry et al., 1976).

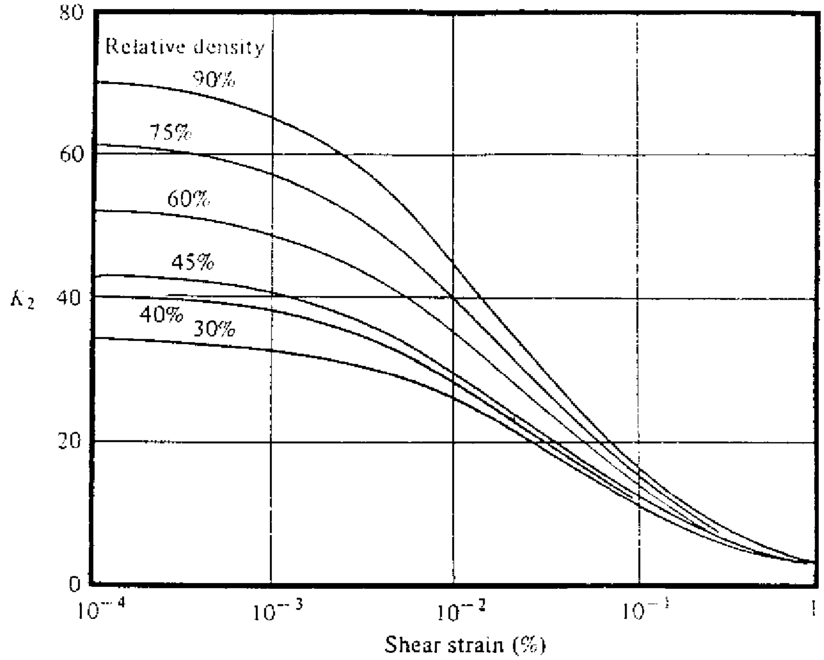

FIGURE 2. Values of K_2 for sand at different relative densities (from Seed and Idriss 1970, cited by Das 1983)

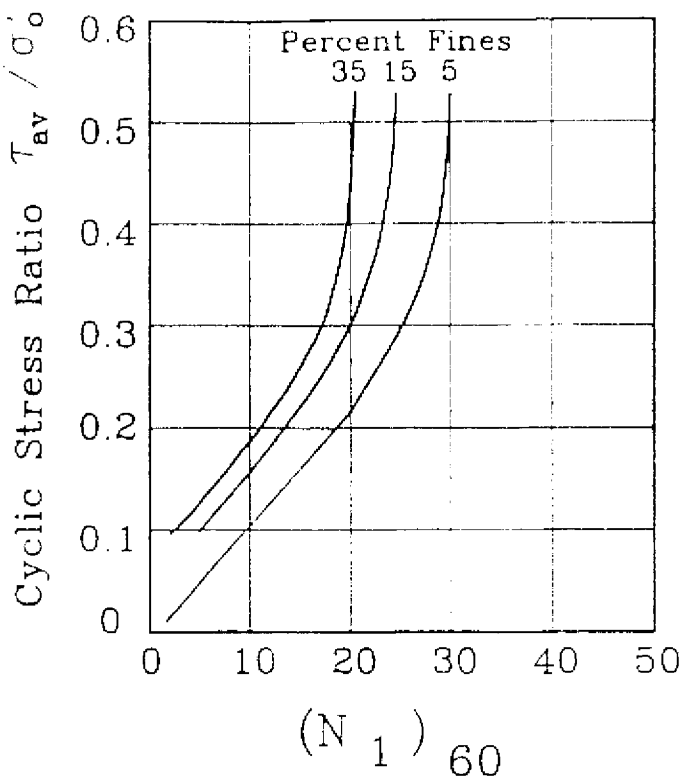

FIGURE 3. Empirical relationship for shear stress ratio and SPT (after Seed and De Alba 1986).