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Soil properties and seismic response

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ABSTRACT: Fundamental relations between basic soil properties and seismic response of soil deposits are discussed. The results of cyclic strain-controlled tests are used in conjunction with a cyclic soil characterization method which assumes that the cyclic shear strain, γ_c, is the governing loading parameter and that the type of soil is characterized by the value of its Plasticity Index, PI. The test results indicate that deposits of clay with high PI are capable of strongly amplifying the incoming earthquake motion. Such amplification is less likely through clays which have small or medium PI. Through saturated sand deposits, for which PI = 0, such amplification is unlikely. However, stiffness and strength of saturated sands may significantly degrade under cyclic earthquake loads, leading to a lengthening of the original predominant period of the deposit. The cyclic degradation is considerably smaller in clays having low to medium PI, and it is the smallest in clays with high PI. Consequently, a shifting of the predominant period of plastic clay deposits should not be expected.

1. INTRODUCTION

The response of soil deposits to earthquake loads is a complex three-dimensional phenomenon. It involves random three-directional excitation, nonlinear, nonhomogeneous and anisotropic material, and usually a complicated geometry. However, its fundamental aspects can be explained on a simple model of a homogeneous horizontally layered soil profile excited at its base by one-directional horizontal shaking, which involves only vertically propagating shear waves. An element of soil, which, prior to an earthquake, is subjected to vertical and horizontal geostatic effective stresses (σ_v' and σ_h' respectively), is during an earthquake subjected under such ideal conditions to cyclic shear stresses, τ, and strains, γ. This is sketched in Fig. 1. In the geotechnical laboratory, the corresponding cyclic loading response can be most effectively studied by the cyclic strain-controlled undrained direct simple shear tests (CyUDSS), the results of which are sketched in Fig. 2. In Figs. 1 and 2, N = number of cycles, γ_{cN} = cyclic shear strain amplitude in cycle N, τ_{cN} = cyclic shear stress amplitude in cycle N, G_{mN} = maximum shear modulus corresponding to cycle N, G_{sN} = secant shear modulus in cycle N, and u_N = residual (or permanent) cyclic pore pressure in cycle N.

Cyclic laboratory studies have shown, that the loading parameters which govern the cyclic response of saturated soils are those that govern the distortion (deformation) of the soil skeleton. The main component of such distortion is relative displacement between soil particles, which can be expressed in terms of the shear strain, γ. Such displacements are directly responsible for the breakage of particle bonds, slippage at the particle contacts, corresponding change of microstructural repulsion forces, and the tendency towards volume change which causes pore pressure variation. The most important cyclic loading parameters are therefore the cyclic shear strain amplitude, γ_c (measures the relative magnitude of displacements between soil particles in a single loading cycle), and the number of cycles N (related to the cumulative distortion of the soil skeleton).

In this paper, the cyclic shear strain-dependent soil behavior, which was measured and analyzed by different researchers under the conditions presented in Figs. 1 and 2, is synthesized. The main objective of the synthesis is to explain in a simple form: (i) the fundamental aspects of the cyclic responses of different saturated soils, and (ii) how different types of soils may affect the seismic response of soil deposit. For this purpose, the *in-situ* soil properties are characterized by (i) the Plasticity Index, PI, which is related to the size,

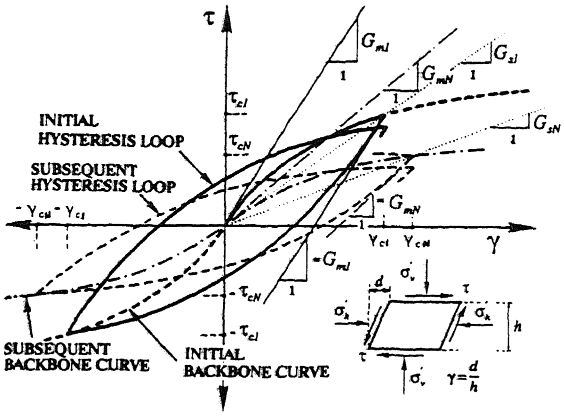

Fig. 1. Idealized behavior of soil element during earthquake

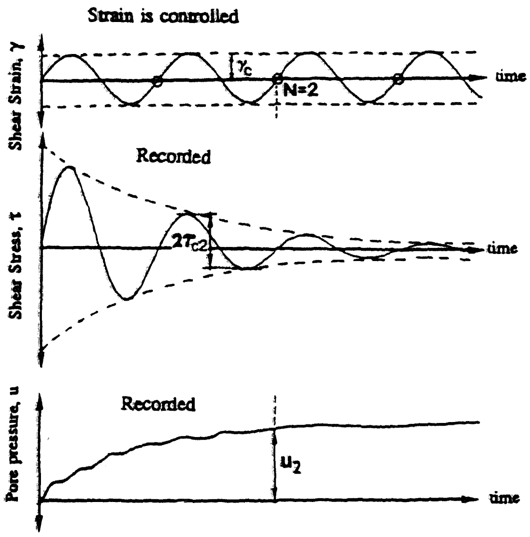

Fig. 2. Sketch of the strain-controlled CyUDSS test results

shape and mineralogy of soil particles; (ii) the effective vertical and horizontal consolidation stress, σ'_v and σ'_h respectively, which characterize the *in-situ* initial loading conditions; and (iii) the overconsolidation ratio, OCR, the magnitude of which reflects the loading history of soil deposit.

2. CYCLIC PARAMETERS AND DESIGN CURVES

From Figs. 1 and 2, the cyclic response parameters which are commonly used in engineering practice can be derived. These are: (i) maximum initial shear modulus at small strains, G_{mN}, corresponding to cycle N; (ii) secant shear modulus at cycle N

$$G_{sN} = \frac{\tau_{cN}}{\gamma_{cN}}; \qquad (1)$$

(iii) equivalent damping ratio at cycle N

$$\lambda_N = \frac{1}{2\pi} \cdot \frac{\Delta E}{G_{sN}\gamma_{cN}^2}, \qquad (2)$$

where ΔE is the area enclosed by the cyclic loop; (iv) degradation parameter

$$t = -\frac{\log \delta}{\log N}, \qquad (3)$$

which describes the rate of the degradation of G_{sN} with N in a cyclic strain-controlled test (in which case γ_{cN} = const.) via the degradation index (Idriss et al. 1978)

$$\delta = \frac{G_{sN}}{G_{s1}} = \frac{\tau_{cN}/\gamma_{cN}}{\tau_{c1}/\gamma_{cN}} = \frac{\tau_{cN}}{\tau_{c1}}, \qquad (4)$$

and (v) the cyclic residual (permanent) pore pressure at cycle N, u_N.

Since both G_{s1} and λ_1 depend on the cyclic strain amplitude γ_{c1}, it is customary to represent the initial loading stress-strain relationship (initial loading backbone curve shown in Fig. 1) by the G_{s1}/G_{m1} versus γ_{c1} and λ_1 versus γ_{c1} curves, such as sketched in Fig. 3. Such characterization is convenient for two reasons. First, both G_{s1}/G_{m1} and λ_1 are dimensionless parameters that are not significantly affected by effective consolidation stresses and OCR (Dobry and Vucetic 1987). Second, G_{m1} can be obtained from the *in-situ* seismic measurements of shear wave velocity, enabling the determination of field values of G_{s1} and τ_{c1}. By combining the *in-situ* G_{m1} with the laboratory G_{s1}/G_{m1} versus γ_{c1} curve, $G_{s1} = G_{m1}\ (G_{s1}/G_{m1})$ and $\tau_{c1} = G_{s1}\gamma_{c1}$. In fact, curves in Fig. 3 are the most popular design curves in seismic site response analyses.

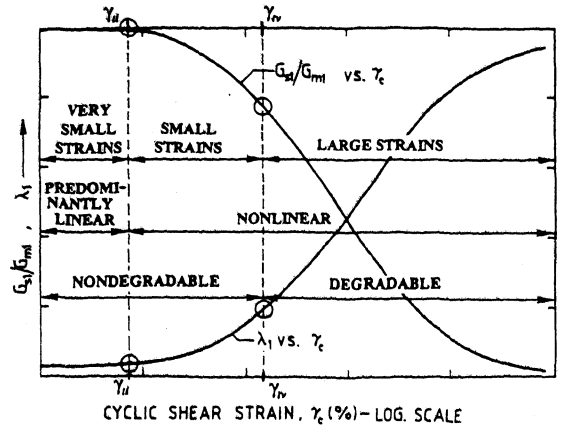

Fig. 3. Secant modulus reduction and damping curves

3. CYCLIC PORE PRESSURES AND THRESHOLD SHEAR STRAIN

Under different levels of cyclic shear strain, γ_c, soils behave differently. The approximate ranges of γ_c that correspond to distinctively different behavior are marked in Fig. 3. To fully appreciate the significance of different ranges of γ_c, it is necessary to understand that for all soils there is a γ_c below which soil structure does not change, and beyond which the soil skeleton (microstructure) starts to change irreversibly (Vucetic, 1991). This level of γ_c is usually called the threshold shear strain, or as suggested more recently (Matasovic and Vucetic, 1992), the volumetric threshold shear strain, γ_{tv}.

If during cyclic loading caused by an earthquake $\gamma_c > \gamma_{tv}$, a tendency toward volume change of the soil will develop, i.e., the tendency toward either compaction or dilation. In cases where soil deposit consists of dry cohesionless granular materials, the cyclic loading will cause compaction; while if such soils are fully saturated the cyclic loading will result in the development of excess pore water pressure, sometimes of sufficient magnitude to cause liquefaction. Under the same cyclic loading conditions, in fully saturated clayey soils the residual pore water pressures will also develop, which can be followed by either settlement or expansion after the cyclic loading. It is evident that cyclic pore water pressure generation and volume change of cohesionless and cohesive soils are conceptually interrelated in a similar manner. The above analogy between volume change and cyclic pore water pressure generation, which applies to both cohesionless and clayey soils, led to the name of the cyclic volumetric threshold shear strain, γ_{tv}. In other words, γ_{tv} is the smallest γ_c at which either settlement, expansion or cyclic pore pressures start to develop.

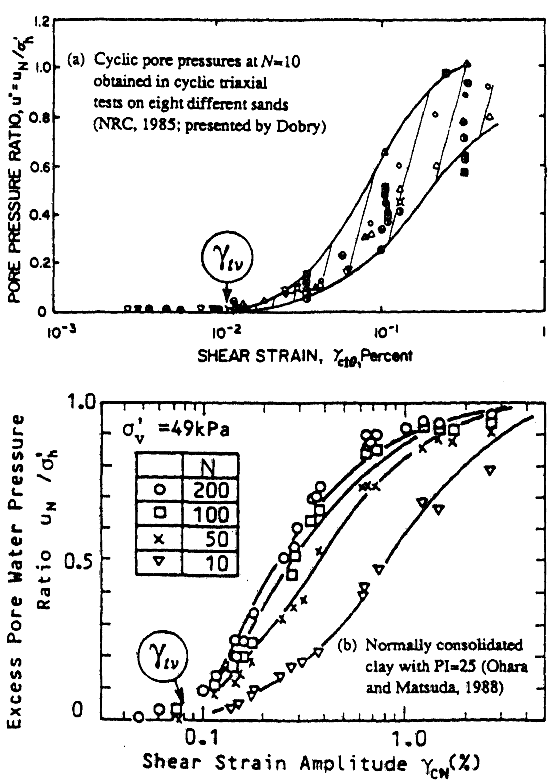

Fig. 4. Cyclic pore pressures in cyclic strain-controlled tests

Figure 4 shows difference between γ_{tv} in sands and clays. For many different sands $\gamma_{tv} \cong 0.01\%$, while for clays γ_{tv} can be one order of magnitude larger. Such difference suggests that γ_{tv} depends on soil microstructure, and may possibly be correlated to the soil's PI. The values of γ_{tv}, evaluated directly or indirectly from nine different studies, are plotted versus the corresponding values of PI in Fig. 5. A trend of increasing γ_{tv} as PI increases is evident. With exceptions for indirectly obtained data points and data pertaining to a partially saturated compacted clay, a relatively narrow band of γ_{tv} can be established. On the same chart, the threshold shear strain which divides essentially linear behavior from the nonlinear, γ_{tl}, is also correlated to PI. This correlation is constructed from the correlation between the modulus reduction curves and PI presented in Fig. 6.

4. CYCLIC RESPONSE DESIGN CHARTS

The correlations between the PI of the soil and the shear modulus reduction curve, G_{s1}/G_{m1} vs. γ_{c1}, and the damping curve, $\lambda_1$vs. γ_{c1}, for wide ranges of OCR are presented in Fig. 6. The approximate range of γ_{tv} from Fig. 5 is conveniently incorporated in both charts. In Fig. 7 the correlation between the degradation parameter curve, t vs. γ_c, and PI is presented for normally consolidated clays. The correlation is refined at small strains by taking into account the corresponding values of γ_{tv}; if $\gamma_c > \gamma_{tv}$, soil degrades and $t > 0$, while if $\gamma_c < \gamma_{tv}$, there is no degradation and $t \cong 0$.

The charts in Figs. 5, 6 and 7 show that soils of high plasticity (large PI), as compared to soils of low plasticity (small PI), behave as more flexible and linearly elastic materials up to larger levels of cyclic shear strains, i.e., larger levels of γ_{tl}. In soils having larger PI degradation of stiffness and residual pore pressures

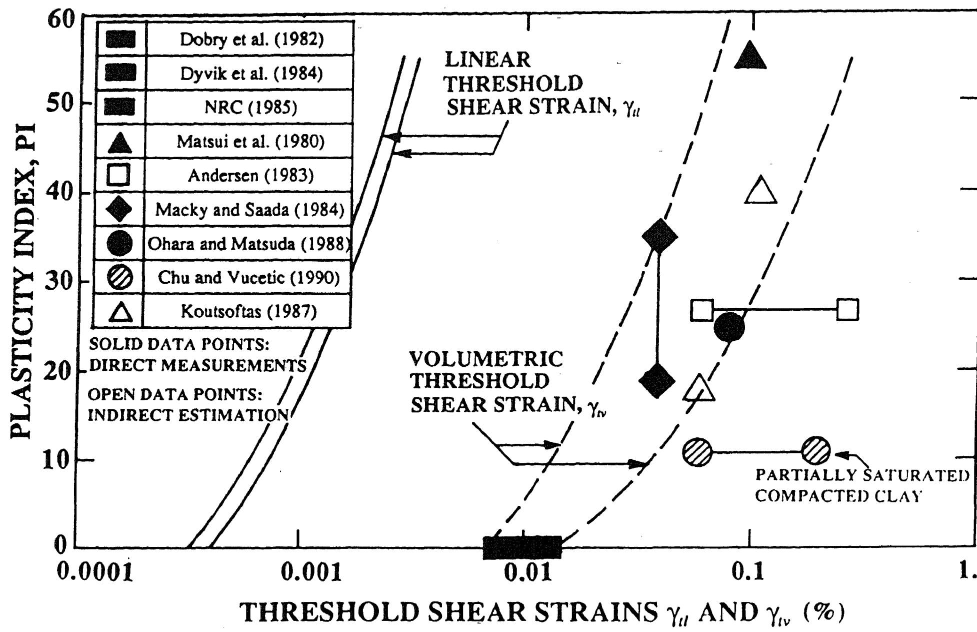

Fig. 5. Effect of PI on threshold shear strains γ_{tl} and γ_{tv}

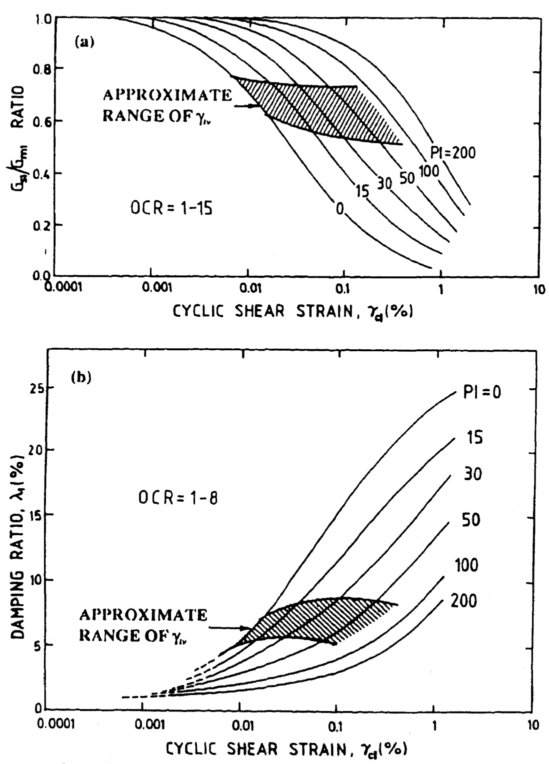

Fig. 6. Effect of PI on modulus reduction and damping curves (Modified from Vucetic and Dobry, 1991)

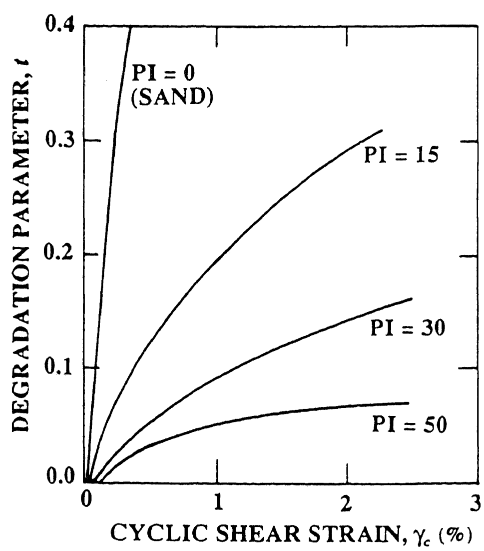

Fig. 7. Effect of PI on cyclic degradation

start to develop at larger γ_c, i.e., γ_{tv} is larger as PI increases. Also, the rates of cyclic degradation (Fig. 7) and damping (Fig. 6) are smaller in high plasticity soils. On the other hand, soils having small PI, and in particular saturated sands and non plastic silts for which PI = 0, are less flexible and highly nonlinear. They have small γ_{tl}, cyclic pore pressure starts to build up at smaller γ_c (i.e., γ_{tv} is smaller), and damping is larger (Fig. 6). Consequently, the stiffness and strength of low plasticity soils degrade much faster during cyclic loading (Fig. 7).

5. SEISMIC RESPONSE OF DIFFERENT SOILS

The seismic response of different natural soil deposits must reflect the different dynamic properties of soil material. To explain the effects of the type of soil on the seismic response in a systematic manner, it is convenient to link the cyclic behavior to the standard Unified Soil Classification used in geotechnical engineering practice. Such link can be conveniently established via PI, and can be summarized in the following three points:

Highly plastic soils have small damping and relatively large range of linearly elastic behavior, which are both prerequisites for a resonance. Therefore, when seismic waves propagate through a deposit composed of such soils, a resonance of the deposit may take place. Such resonance can greatly amplify the incoming seismic motion and cause considerable damage. Soil failure due to cyclic degradation of stiffness and strength in the highly plastic clay deposits is unlikely, because the degradation is small.

Low plasticity fully saturated soils and soils with no plasticity, like fully saturated sands, exhibit a highly nonlinear behavior and considerable damping even at very small shear strains, and during continuous cyclic loading their stiffness and strength degrade significantly. An amplification of motion through such soils

Table 1: Effect of soil type on ground response

TYPES OF SOIL			
NON PLASTIC PI = 0	LOW PLASTICITY PI = LOW	MEDIUM PLASTIC PI = MEDIUM	HIGHLY PLASTIC PI = HIGH
SANDS AND NONPLASTIC SILTS	SILTY CLAYS, CLAYEY SILTS, LOW PLASTICITY CLAYS	MEDIUM PLASTIC CLAYS	HIGH PLASTICITY CLAYS
EFFECT ON GROUND RESPONSE			
AMPLIFICATION OF GROUND MOTION			
INSIGNIFICANT OR NONE (ATTENUATION POSSIBLE)	SMALL, INSIGNIFICANT, OR NONE (ATTENUATION POSSIBLE)	MODERATE	LARGE
LENGTHENING OF PREDOMINANT PERIOD			
SIGNIFICANT DURING LIQUEFACTION PROCESS	LIKELY IF LARGER PORE PRESSURES BUILD UP	UNLIKELY OR INSIGNIFICANT	UNLIKELY OR IMPOSSIBLE
DEGRADATION OF STIFFNESS AND STRENGTH; REDUCTION OF BEARING CAPACITY			
LARGE, OR COMPLETE DURING FULL LIQUEFACTION	SIGNIFICANT FOR NORMALLY CONSOLIDATED OR SMALL OCR SOILS	SMALL, OR INSIGNIFICANT FOR OVERCONSOLIDATED SOILS	INSIGNIFICANT

IMPORTANT: CEMENTED, HIGHLY SENSITIVE, AND OTHER "SPECIAL SOILS" ARE NOT INCLUDED.

caused by the resonance of the deposit is therefore hardly possible. However, due to the significant softening of the soil, the predominant period of the ground surface motion can considerably increase (Vucetic and Zorapapel, 1990; Zorapapel and Vucetic, 1990), and the structures with similar predominant period may be affected. The most critical aspect of the response of such soils is large drop in soil strength due to cyclic degradation, which may cause the failure of the foundation soil and collapse of supported structures. The extreme examples of such failure are the liquefaction-induced failures.

The seismic response of medium plasticity soils, i.e., the medium plasticity clays, is somewhere between the two extreme cases described above. Such soils can moderately amplify the incoming seismic motion, and their bearing capacity can be moderately decreased by a continuous cyclic loading.

Table 1 presents these conclusions in a systematic manner.

6. CLOSING COMMENT

In this paper only fully saturated soils have been treated. Also, only a simple case of vertically propagating seismic shear waves has been considered. In the real situations, however, seismic loads are never so simple and other types of soils may be involved (partially saturated clays, sensitive clays, dry sands and cemented sands). The results and conclusions put forward in this paper therefore represent only a framework of understanding of the seismic response of different soil deposits, rather than providing procedures for direct engineering applications.

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