DYNAMIC PROPERTIES OF UNTREATED AND TREATED COHESIVE SOILS

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SUMMARY

Dynamic properties of four compacted commercially produced cohesive soils with plasticity indices ranging from 22 to 600%, were determined under a wide range of shear strains and confining pressures. Two of the cohesive soils were treated with lime up to 8% by weight, and their dynamic properties were determined under the same conditions as the untreated cohesive soils. Shear strain, plasticity, lime treatment level, and confining pressure were determined to be the four main factors that control the main dynamic property of these soils, the shear modulus. Lime treatment greatly improved the shear modulus of these clays and considerably reduced their plasticity indices.

Models similar to those used in conventional geotechnical research, to predict the shear modulus as a function of shear strain, confining pressure, plasticity and treatment level, were developed for the treated and untreated cohesive soils. These models can be used to predict the shear modulus at different shear strain levels once the plasticity of the soil and the confining pressure are known. To verify the applicability of the form of the models to other soils, a compacted natural cohesive soil was also tested. The model for the natural clay was found to fall within the range of the models for the commercially produced clays. These models could thus be used in the analysis of dynamically loaded natural cohesive soil sites and for treated soils used as a foundation material in earthquake susceptible regions.

INTRODUCTION

To evaluate the effect of dynamic loads on civil engineering systems, it is necessary to understand, measure, and quantitatively model the dynamic and cyclic properties of the soils involved. Analysis of an engineering problem that involves dynamic loading of soils requires the determination of both the shear modulus (G) and damping ratio (D). These two parameters are usually determined either in the laboratory or in the field using various techniques. Numerous data on the dynamic properties of cohesionless and cohesive soils are available. Void ratio has been shown to be the most significant factor in the dynamic properties of cohesionless soils [Seed and Idriss 1970], whereas the plasticity index is very significant in the dynamic properties of cohesive soils [Vucetic and Dobry 1991]. Performing consistency limits of cohesive soils is cheap and fast compared to dynamic experiments. Interrelating mechanical properties of soils to the plasticity index of soils has been successful for some static properties of soils but has not been correlated with dynamic properties of cohesive soils. Developing a study of the dynamic properties of cohesive soils for a wide range of plasticity indices would give a clear understanding of the effect of the plasticity index on the dynamic properties of cohesive soils.

Some clay soils are very expansive and exist in many countries. In the presence of water, because of seasonal fluctuations due to climatic conditions and man-made structures, expansive soils become unworkable, lose their shear strength, and exhibit large deformations that cannot be predicted by the classical elastic or plastic theories. Seasonal fluctuations are common within the upper stratum of the soil profile, referred to as the zone of seasonal fluctuations. This zone of seasonal fluctuations varies in depth from 6 to 10 m depending on the soil [Nelson and

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Miller 1992]. Expansive soils are treated through chemical stabilization. Chemical stabilization is the mixing of soil with one, or a combination of admixtures for the general objective of improving or controlling its volume stability, strength and stress-strain behavior, permeability, and durability. Chemical methods of stabilization alter the inherent properties of soils and it is this ability that gives chemical stabilization its great promise. Lime has been used successfully as a chemical stabilizer for expansive soils. With advances in mixing and pressure injection equipment, chemical stabilization is possible at different depths depending on specific job conditions, e.g., typical depths under pavements are 1 to 1.5 m, under building foundations are 2 to 3 m, and 3 to 12 m for railroads, landfills, embankments and other deep problem areas [Boynton et al. 1985]. One of the most profound effects of chemical stabilization is the reduction of the plasticity of the soil. This paper reports the results of a testing program on cohesive soils treated with lime and provides analytical models that can predict the dynamic behavior of treated and untreated cohesive soils.

SOILS TESTED

The soils used in this study were four commercially produced clay and a natural clay. These soils were selected to cover as wide a range of plasticity index as possible. The four different soils are: Montmorillonite Sodium, commercially known as Super-Gell X; Montmorillonite Hydrogen, commercially known as Hydrogel Bentonite; Attapulgite clay, commercially known as Micro-sorb ES; and Kaolinite, commercially known as Speswhite. A summary of consistency limits, liquid limit (LL), plastic limit (PL), plasticity index (I_p); and specific gravity (G_s) of these clays is given in Table 1. The natural clay was obtained from a site in Virginia. The liquid limit, plastic limit, and plasticity index of the natural clay were found to be 98%, 40%, and 58%, respectively. The lime used was hydrated lime. This hydrated lime met the specifications required for lime for soil stabilization use [ASTM C977]. The percentage of lime used varied from 2 to 8% by weight.

TABLE 1. Consistency Limits and Specific Gravity of the Soils Tested

CLAY TYPE	LL (%)	PL (%)	I_p (%)	G_s
Kaolinite	53	31	22	2.63
Attapulgite	272	117	155	2.56
Montmorillonite Hydrogen	476	51	425	2.66
Montmorillonite Sodium	648	48	600	2.67

EXPERIMENTAL EQUIPMENT

To determine the dynamic properties, a resonant column device was used. The resonant-column test method [ASTM D 4015-87] is a relatively nondestructive test, used to determine the modulus and damping of soils by means of propagating waves in a cylindrical soil specimen (column). When sinusoidal torque is applied to the specimen, shear waves are propagated and when sinusoidal axial compression is applied, compressional waves are propagated. The primary advantage of using this testing technique is that very accurate moduli can be evaluated for a strain range of 10^{-4}% to 10^{-1}% under various excitations [Kim and Stokoe 1992; Zhang 1994; and Drnevich 1978]. A Drnevich-type resonant column apparatus was used for this research. A detailed description of this apparatus and the theoretical model employed can be found in Drnevich [1978] and Chepkoit [1999].

EXPERIMENTAL PROCEDURE

All samples were compacted with a mechanical compactor to the Standard Proctor energy level, as described in ASTM D698 method A. All samples were prepared at or slightly higher by 1 to 3% than the optimum moisture content. The Montmorillonite Sodium, Montmorillonite Hydrogen, and natural cohesive soil samples were mixed with the required water content and allowed 24 hours mellowing time before compaction, whereas, the Attapulgite and Kaolinite were allowed 1 hour mellowing time. All lime treated samples were prepared as per the procedure recommended in ASTM D3551. During the mellowing period, mixed samples were kept in sealed plastic bags so that evaporation and carbonation were kept at a minimum. The following abbreviations are used for the four soils: Kaol for Kaolinite, Attap for Attapulgite, M-H for Montmorillonite Hydrogen, M-Na for Montmorillonite Sodium and Nat-clay for natural clay. Lime is abbreviated by the letter L.

After compaction, specimens were sampled using a thin-walled Shelby tube of 35 mm internal diameter and 80 mm long. As soon as the specimen was extruded, it was wrapped in plastic wrap, waxed wrap, and aluminum foil to minimize the loss of moisture and to prevent carbonation reaction in the treated samples. The treated samples were placed in a curing tank (to maintain 100% humidity) and kept in the oven for 65 hours at 105^{o} F [Drake and Haliburton 1972; and TRB State of the Art Report 5 1987]. Untreated samples were kept in the curing room (temperature at 74^{o}F and humidity greater than 94%) for 65 hours to achieve the same environmental conditions as the treated specimens. Isotropic consolidation was performed on cured samples under four different confining pressures: 1 kPa, 70 kPa, 140 kPa, and 210 kPa.

DYNAMIC PROPERTIES OF TESTED SOILS

Effect of Shear Strain Amplitude: As shown in Figure 1, the shear modulus of the untreated soils G is independent of the shear strain at very low strain levels, which indicates the elasticity of the material. The variation of the elastic range depends on the type of soil. Beyond the elastic state, the shear modulus decreases with the increase in shear strain. However, the rate of decrease or degradation differs with the type of soil. The maximum shear modulus G_{max} is defined as the shear modulus at a very low strain level, i.e., when the soil is in its elastic state and when the shear modulus is independent of the strain level.

Effect of Confining Pressure: The shear modulus increases with the increase in the confining pressure especially at low strain levels and the increase is highly pronounced in soils of low plasticity and high treatment levels.

Effect of Plasticity: Generally, cohesive soils with low plasticity exhibit a high shear modulus at low strain levels. At high strain levels, all soils regardless of their plasticity tend to converge, because the shear modulus of low plasticity soils decreases rapidly with increases in shear strain, as compared to cohesive soils with high plasticity (Figure 1).

Effect of Treatment Level: Lime treatment has been shown to reduce the plasticity of cohesive soils, hence it will have a significant effect on the dynamic properties of the treated cohesive soils. For example, at 8% lime, the plasticity index for the M-Na soil was reduced from 600 to 120 and for the M-H soil from 425 to 302, and 6% lime was sufficient to reduce the plasticity index of the Nat-clay to zero. High treatment levels of lime exhibit high shear modulus in all treated cohesive soils at low strain levels. At high strain levels, the shear modulus of all soils regardless of their level of treatment tend to converge. As an example, Figure 2 shows the shear modulus of the treated M-Na soil as a function of shear strain.

MODELING OF SHEAR MODULUS

The stress-deformation characteristics of soils vary significantly depending upon the shear strains to which the soils are subjected. Formulation of a material model is an important step in obtaining solutions for practical engineering problems. A model should be sufficiently comprehensive to represent all important material behavior that occurs within the given structure under the prevailing loading conditions. When the behavior of a soil is expected to stay within the range of small strains, the use of an elastic model is justified and the wave propagation method based on linear elastic theory can be employed. At a medium range of strains, the soil behavior becomes elasto-plastic and the shear modulus tends to decrease with the increase in shear strain. To describe the non-linear behavior of soils a unified model was proposed by Ishibashi and Zhang [1993]. The unified model can be presented in a simplified form as:

$$G = k(\gamma)\, f(e)\, \sigma^{m(\gamma)} \tag{1}$$

where k(γ) = decreasing function of the cyclic shear strain amplitude γ and is unity at a very small strain ($\gamma \lhd 10^{-6}$), f(e) = function of void ratio e, σ = the mean effective confining pressure, and m(γ) = increasing function of strain (γ). The expression of the initial shear modulus, G_{max} is defined as [Ishibashi and Zhang 1993]

$$G_{max} = k_o\, f(e)\, \sigma^{m_o} \tag{2}$$

Dividing the expression of general shear modulus (G) shown in equation (1) with the initial shear modulus (G_{max}) shown in equation (2), eliminates the effect of void ratio:

$$\frac{G}{G_{max}} = k(\gamma)\, \sigma^{(m(\gamma) - m_o)} \tag{3}$$

Transforming this power model (equation (3)) to log base gives a linear model:

$$Log(\frac{G}{G_{\max}}) = Log(k(\gamma)) + (m(\gamma) - m_o) L\log\sigma \tag{4}$$

By plotting the $\log(G/G_{max})$ against $\log\sigma$, a linear relationship is obtained where the slope is $m(\gamma)$-m_o and the intercept is $\log(k(\gamma))$. Various researchers such as Fahoum et al. [1996], Dobry and Vucetic [1987], and Kim and Stokoe [1992], have shown that plasticity in untreated cohesive soils and lime treatment level in treated cohesive soils have significant effect on shear modulus. To include the plasticity of cohesive soils and the lime treatment level, equation (3) was modified in this research to be

$$\frac{G}{G_{\max}} = k(\gamma, I_p, L)\sigma^{(m(\gamma, I_p, L) - m_o)} \tag{5}$$

where $k(\gamma,I_p,L)$ is a function of shear strain and plasticity or lime treatment level and $m(\gamma,I_p,L)$-m_o is an exponential of confining pressure and is a function of shear strain and plasticity or lime treatment level. Development of a model for G will depend on the determination of the functions $k(\gamma,I_p,L)$ (for simplicity, this will be referred to as $k(\gamma)$) and $m(\gamma,I_p,L)$-m_o (for simplicity this will be referred to as $m(\gamma)$-m_o) and the value of G_{max}. The experimental test results indicated that the main variables of the normalized shear modulus are shear strain, plasticity, treatment level, and confining pressure. The variations of $k(\gamma)$ and $m(\gamma)$-m_o for the soils tested were plotted as a function of the shear strain. The data for cohesionless soil (Ottawa 30) from Al-Sanad [1984] and Zhang [1994] were used to determine the extreme limits of the functions.

The k(γ) Function: The $k(\gamma)$ function was plotted as shown in Figure 3 for the untreated soils and Figure 4 for the treated M-Na soils. The figures show that the $k(\gamma)$ function decreases as the shear strain increases for all the soils, which agrees with Ishibashi and Zhang [1993]. The curves tend to move towards increasing strain as the plasticity index I_p increases or as lime treatment level decreases. High plasticity or low lime content soils tend to locate at the high end of the strain range and low plasticity or high lime content soils tend to locate at the low end of the strain. A numerical least squares algorithm [McCuen 1993] was customized and used to fit the data obtained. The hyperbolic trigonometric functions shown below gave the best prediction and goodness-of-fit statistics.

$$k(\gamma) = \frac{C_1}{1.0 + \left[\frac{C_2 I_p^{C_3}}{\gamma}\right]^{C_4}} \tag{6}$$

The unknowns C_i were calibrated with two predictor variables (plasticity index or lime content and shear strain) and one criterion variable $k(\gamma)$. A summary of the calibrated equations; sample size, n; and goodness-of-fit statistics of the calibrated model, i.e., ratio of standard error of estimate (S_e) to standard deviation (S_y), square of the correlation coefficient (R^2), and relative bias ($\bar{e}$/Y) is given in Table 2.

The m(γ)-m$_o$ Function: The $m(\gamma)$-m_o function increases as the shear strain increases in all the soils (Figures 5 and 6). For the effect of the plasticity index or lime treatment level, the curves tend to move towards increasing strain as I_p increases or lime content decreases. Low plasticity or high lime content soils tend to locate at the low end of the strain range and high plasticity or low lime content soils tend to locate at the high end of the strain range. The same hyperbolic model as discussed before in the $k(\gamma)$ function with a slight modification gave the best prediction and was adopted.

$$m(\gamma)-m_o=\frac{C_1 I_p^{C_2}}{1.0+\left[\frac{C_3 I_p+C_4}{\gamma}\right]^{C_5}} \tag{7}$$

The unknown C_i were calibrated with two predictor variables (plasticity index or lime treatment level and shear strain) and one criterion variable $m(\gamma)-m_o$. A summary of the calibrated equations; sample size, n; and goodness-of-fit statistics of the calibrated model, i.e., ratio of standard error of estimate (S_e) to standard deviation (S_y), square of the correlation coefficient (R^2), and relative bias ($\bar{e}/Y$) is given in the Table 2.

TABLE 2. Summary of Shear Modulus Models for Untreated and Lime Treated Soils

Untreated Soils n = 116, R^2 = 91%, S_e/S_y = 31%, $\bar{e}/Y$ = 0.0	$k(\gamma)=\frac{1.0}{1.0+\left[\frac{0.0109 I_p^{0.486}}{\gamma}\right]^{-1.0}}$
n = 116, R^2 = 99% S_e/S_y = 8%, $\bar{e}/Y$ = 0.0	$m(\gamma)-m_o=\frac{0.366 I_p^{-0.177}}{1.0+\left[\frac{0.000398 I_p+0.0233}{\gamma}\right]}$

M-Na Plus Lime n = 98, R^2 = 97% S_e/S_y = 18%, $\bar{e}/Y$ = 0.0	$k(\gamma)=\frac{1.0}{1.0+\left[\frac{0.350 L^{-1.068}}{\gamma}\right]^{-1.0}}$
n = 98, R^2 = 95% S_e/S_y = 22%, $\bar{e}/Y$ = 0.02	$m(\gamma)-m_o=\frac{0.0262 L^{0.813}}{1.0+\left[\frac{0.0864 L^{-0.565}}{\gamma}\right]}$

M-H Plus Lime n = 79, R^2 = 99%, S_e/S_y = 8%, $\bar{e}/Y$ = 0.0	$k(\gamma)=\frac{1.0}{1.0+\left[\frac{0.235 L^{-0.712}}{\gamma}\right]^{-1.0}}$
n = 79, R^2 = 97% S_e/S_y = 17%, $\bar{e}/Y$ = 0.11	$m(\gamma)-m_o=\frac{0.1112L+0.246}{1.0+\left[\frac{0.762 L^{0.782}}{\gamma}\right]}$
Where $k(\gamma)$ = decreasing function of the shear strain (γ), $m(\gamma)-m_o$ = increasing function of shear strain (γ), L = lime content (% by weight), I_p = plasticity index (%), γ = shear strain (%), and σ = confining pressure (kPa).	

CONCLUSION

An experimental program to determine the dynamic behavior of four compacted commercially produced cohesive soils and one natural clay was undertaken. Models similar to those used in conventional geotechnical research to predict the shear modulus as a function of shear strain were developed for untreated and lime treated cohesive soils. In these models, the void ratio effect was eliminated by dividing the shear modulus function at different strain levels with the shear modulus function at a low strain level. The normalized shear modulus (G/G_{max}) was then expressed as a function of confining pressure with a multiplying function $k(\gamma)$ and exponential function $m(\gamma)$-m_o. These two parameters were modeled as functions of shear strain, and plasticity or treatment level. Analysis of global and local bias, and a summary of goodness-of-fit statistics confirmed a model that could be used to predict the functions $k(\gamma)$ and $m(\gamma)$-m_o.

The maximum shear modulus, G_{max} of the cohesive soils can be obtained from the field using field techniques or empirical equations. The confining pressure could be obtained based on the unit weight of the soil and depth of the soil element, and the plasticity of the soil could be obtained from simple laboratory experiments. With the availability of these parameters (G_{max}, σ, and I_p), the models of $k(\gamma)$ and $m(\gamma)$-m_o could be used to predict the shear modulus at different strain levels. The applicability of the form of the model to be used for other soils was verified by testing compacted natural soils. The model for the natural clay was found to fall within the range of the models for the commercially produced clays. These models could thus be used in the analysis of treated or untreated soils used as a foundation material in earthquake susceptible regions.

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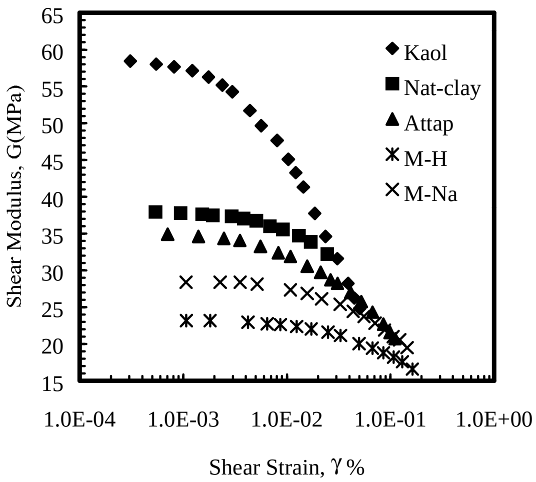

FIGURE 1. Shear Modulus of Untreated Cohesive Soils as a Function of Plasticity (σ = 1 kPa)

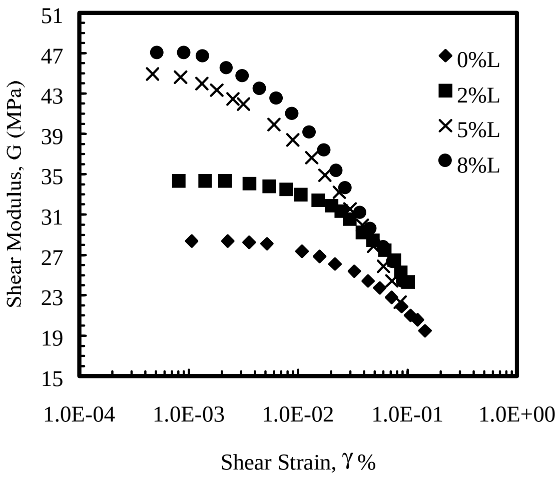

FIGURE 2. Shear Modulus of Lime Treated M-Na Soils as a Function of Lime Content (σ = 1 kPa)

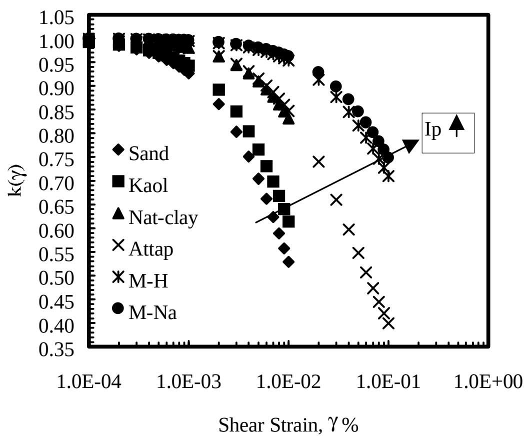

FIGURE 3. k(γ) for Untreated Cohesive Soils as a Function of Plasticity Index

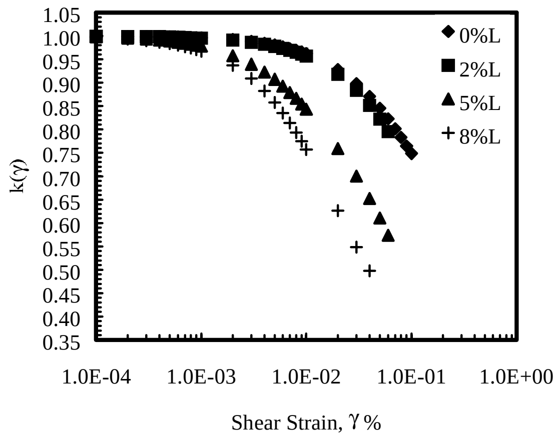

FIGURE 4. k(γ) for M-Na Soils Treated with Lime as a Function of Lime Treatment Level

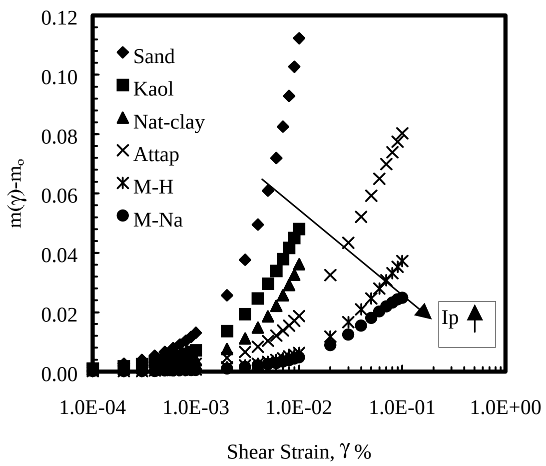

FIGURE 5. m(γ)-m$_o$ for Untreated Cohesive Soils as a Function of Plasticity Index

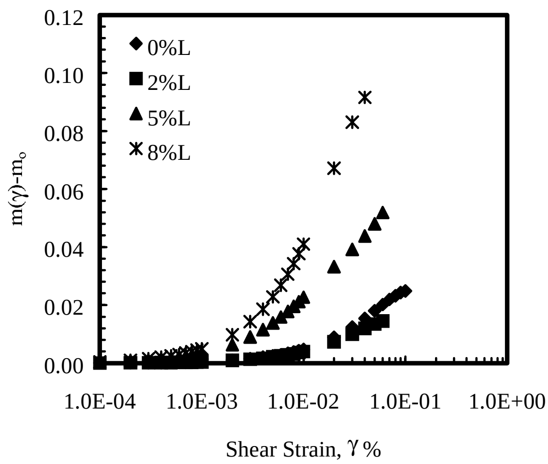

FIGURE 6. m(γ)-m$_o$ for M-Na Soils Treated with Lime as a Function of Lime Treatment Level