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Empirical Relationship for Estimation of Mechanical Soil Properties in Local Ground Responds Analysis

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Abstract- Site effects are quantified via site response analysis, which involves the propagation of earthquake motions from the base rock, through the overlying soil layers, to the ground surface. Ground response problems involving small to moderate levels of shear strain are best analyzed with equivalent linear methods due to the simplicity of parameter selection and faster computation times when compared to nonlinear analysis. However, for large-strain ground response problems, EL methods tend to over-damp portions of the ground motion. This motivates the use of more accurate NL analysis for those conditions. One of important parameters for ground response analyses determines properties of soils. In this paper, empirical relationship from different investigation is gathered.

Keywords- *Ground Responds Analysis, Soil Properties, Nonlinear Behavior*

I. INTRODUCTION

The issue of the effect of local soil conditions, source and path effects are important topics in seismology as they are necessary for design of response spectra. The selection of appropriate elastic response spectra according to soil conditions and seismic intensity is the important way to account for site effects for engineering projects. The evaluation of site amplification effects is recognized as one of the most important activities of the seismology and earthquake engineering and the existence of soil amplification was amply demonstrated in many past destructive earthquakes [1]. The distribution of damage caused by earthquake ground shaking commonly reflects real differences in local soil conditions. Regional site conditions relevant for seismic hazard studies can be derived from various geologic, seismologic and geotechnical source [2]. Yang et al (2011) conducted a systematic investigation to understand the effects of permafrost on the ground motion characteristics using one-dimensional equivalent linear analysis. The results showed that the presence of permafrost can significantly alter the ground motion characteristics and it may not be wise to ignore the effects of permafrost in the seismic design of civil structures [3]. Phanikanth, et al (2011) studied the effect of local soil sites in modifying ground response by performing one dimensional equivalent-linear ground response analysis for some of the typical Mumbai soil sites [4]. Goda (2012) looked at nonlinear response potential of main shock, after shock sequences from the K-NET and KiK-net databases for Japanese earthquakes. This study examined the validity of artificially generated sequences based on the generalized Omori's law using a probabilistic framework analysis. He also showed that the peak ductility demand ratio between the main shock after shock sequences and main shock alone depends on main shock magnitude [5].Cadet et al (2012) have proposed a methodology to normalize the site amplification factors with respect to a standard outcropping rock site in line with the present design codes, by applying two correction factors, namely, the depth correction factor and the impedance contrast normalization factor [6].

Zahedi-Khameneh et al (2013) proposed a real-time prediction model of strong ground motions based on non-parametric wave type, in which an adaptive windowing technology is used to catch the dominant frequency of ground motions, and then a radial-basis function (RBF) network is incorporated to predict next time step acceleration of earthquake record [7].Shear-wave velocity (V_S) should be measured using appropriate techniques, which may include surface wave methods, suspension logging, down hole testing, and cross-hole testing. Remi-based techniques (e.g., Louie [2001]) should be avoided due to potential for bias, particularly at depth [Cox and Beekman 2011]. Although numerous techniques exist for estimating V_S from penetration resistance [e.g., Robertson (2009) for CPT and Brandenberg et al. [2010] for SPT], these should not be used for GRA applications, for which results can be very sensitive to small variations in V_S that can only be reliably evaluated using high-quality measurements. One such set of statistical relations are given by Toro [1995] and are based on 513 shear-wave velocity profiles in California and 44 profiles from the Savannah River site in Georgia, U.S. For a given depth, V_S is assumed to be log-normally distributed with a depth-dependent standard deviation (σ_{lnvs}).When material-specific test results are unavailable, MR curves can be estimated from relationships. A number of 'classical' MR curves have seen widespread use in geotechnical engineering practice, including Seed and Idriss [1970] (generic curves for clay and sand), Iwasaki et al. [1978] (overburden-dependent curves for sand), and Vucetic and

Dobry [1991] (PI-dependent curves for clay). While each of these models has substantially contributed to our knowledge of dynamic soil behavior, recent models are based on more extensive testing, making them a better choice for contemporary GRA. Darendeli [2001] and Zhang et al. [2008] analyzed the dispersion characteristics of the datasets used to develop their MR models. Whereas there is very little uncertainty in modulus reduction at small shear strains, there is substantial uncertainty in modulus reduction behavior at strains large enough for max G/G_{max} to be less than 1.0.In this paper empirical relationship for ground motion prediction include terms for modeling site response are discuss.

II. PROCEDURE OF SEISMIC GROUND RESPONSE ANALYSIS

One of the most important and most commonly encountered problems in geotechnical earthquake engineering is the evaluation of ground response. Ground response analyses are used to predict ground surface motions for development of design response spectra, to evaluate dynamic stresses and strains for evaluation of liquefaction hazards, and to determine the earthquake-induced forces that can lead to instability of earth-retaining structures. The procedure in the simplest form consists of the following steps: (1) to collect data, (2) to model them for computer programs, (3) to execute computer program, and (4) to interpret the results. Several input data are required in the seismic ground response analysis. They are classified into four categories:

1. Geological or topological configuration, such as soil profiles and cross-sectional shape

2. Mechanical properties

3. Input earthquake motion

4. Parameters to control the flow of the computer program or the method of the analysis

III. ESTIMATION OF MECHANICAL SOIL PROPERTIES

The best way to obtain the mechanical soil properties for seismic ground response analysis is in situ test for the elastic modulus and laboratory test on undisturbed sample for the nonlinear properties. In the engineering practice, however, this is not always possible. The cyclic deformation characteristics test is a costly test in the practice. The frozen sample requires huge cost to retrieve the highly undisturbed sample, and its applicability is limited to clean sand. As a means to overcome this issue, from a practical perspective, test data and empirical equations are introduced mainly based on the Japanese researches in this paper. It is noted that in using an empirical equation, tests on the similar material of concern are to be preferred. In addition, it is essential to understand that test data sometimes scatter very much resulting in a degree of error in analysis.

A. Elastic Properties

A direct in situ measurement of elastic modulus is difficult to perform mainly because accurate measurement of strain is difficult. Therefore, instead, the wave velocities are measured for evaluating the elastic modulus. There are many empirical equations available for evaluating the shear wave velocity from the SPT N-value.

1) Equation by Imai et al.

The pioneering research to correlate shear wave velocity and SPT N-value was performed by Imai et al. (Imai 1977). They gathered Vs and Vp data from various soils and found that there are correlations between the Vs and the SPT N-value. For all soils, the relationship yields

$$V_S=91.0N^{0.337} \qquad (r=0.889)[m/s] \qquad (1)$$

where r denotes coefficient of correlation. In addition, the relationships shown in Fig.1 were proposed for soils classified as per soil type and geologic age, expressed as

$$\begin{aligned} &V_S=102N^{0.292} && \text{Holocene Clay (Ac)} \\ &V_S=80.6N^{0.331} && \text{Holocene Sand (As)} \\ &V_S=114N^{0.294} && \text{Pleistocene Clay (Dc)} \qquad (2) \\ &V_S=97.2N^{0.323} && \text{Pleistocene Sand (Ds)} \end{aligned}$$

Dashed lines in Fig.1 are added by the author and they correspond to half and twice of the S-wave velocities evaluated from Eq. (2). It can be easily observed that the scatter within the data lies between the indicated boundary lines. In other words, these equations exhibit a variation between the half and the twice of the measured velocities. As the shear modulus is proportional to the square of the S-wave velocity, this yields scatter in the shear modulus ranging from 1/4 to 4 times. Recall that a change in the elastic modulus results in shift in the shear strength. In addition, the shear strength is related to the maximum acceleration at the ground surface. This indicates that the maximum acceleration also scatters within 1/4 and 4 times the mean values. This scatter may be outside of allowable range in an engineering practice.

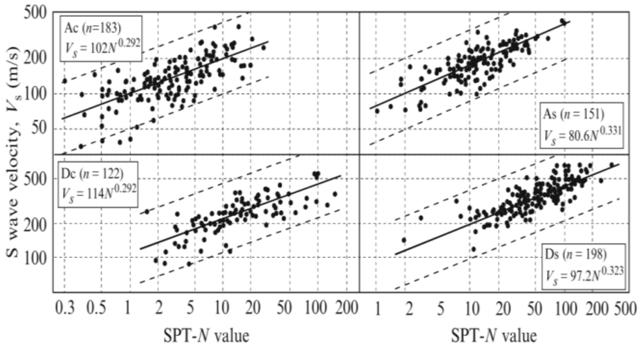

Figure 1. Relationships between SPT N-value and Vs for various soil types (Modified from Imai1977)

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TABLE I. RELATIONSHIP BETWEEN S-WAVE VELOCITY, ELASTIC SHEAR MODULUS, AND SPT N-VALUE

Soil type	Vs=AN^m [m/s]			G0=BN^n [MN/m^2]		
	A	m	r	B	n	r
Clayey fill	98.4	0.248	0.574	15.4	0.577	0.582
Sand or gravel fill	91.7	0.257	0.647	14.2	0.500	0.647
Holocene clay	107	0.274	0.721	17.6	0.607	0.715
Holocene peat	63.6	0.453	0.771	5.37	1.08	0.768
Holocene sand	87.8	0.292	0.69	12.5	0.611	0.671
Holocene gravel	75.4	0.351	0.791	8.25	0.767	0.788
Loam, Shirasu	131	0.153	0.314	22.4	0.383	0.494
Tertiary sand/clay	109	0.319	0.717	20.4	0.668	0.682
Pleistocene clay	128	0.257	0.712	25.1	0.550	0.712
Pleistocene sand	110	0.285	0.714	17.7	0.631	0.729
Pleistocene gravel	136	0.246	0.550	31.9	0.528	0.552

Subsequent to this report, they gathered more data and proposed empirical equations for the S-wave velocity Vs as well as the elastic shear modulus G_0. The results are summarized in Table .1. Here coefficients of correlation are similar for Vs and G_0, which indicates that the density can be measured accurately. S-wave velocities used in this study were obtained through down hole test. Correlation between the S-wave velocity and the SPT N-values may not be appropriate for a detailed analysis because Vs measured by the down hole method is sometimes an average for a certain layer thickness. Therefore, the involved variability might be smaller if S-wave velocity obtained through the suspension logging method is used. In addition, the confining stress dependency of the SPT N-value is not considered in this paper.

2) Evaluation by Japan Road Bridge Design Specifications

As can be seen in Fig.1, range of the SPT N-value depends on the soil type and the geologic age. This signifies that applicability range of an empirical equation need not be infinitely large but can be bounded to a small extent of SPT-N value. Considering this, a more simple equation is provided by the Japan Road Association (1985):

$V_S=100N^{1/3}$ Clayer Soil

$V_S=80N^{1/3}$ Sandy Soil (3)

These equations may be more useful in the practice than the former equations because it is sometimes difficult to distinguish the Holocene and the Pleistocene soils. A comparison between Eqs (2) and Eqs (3) is shown in Fig.2. Equation (3) is similar to the Holocene sand in Eq. (2).The applicable ranges are between 1 and 15 SPT N-value for clays and between 1 and 50 SPT N-value for sands.

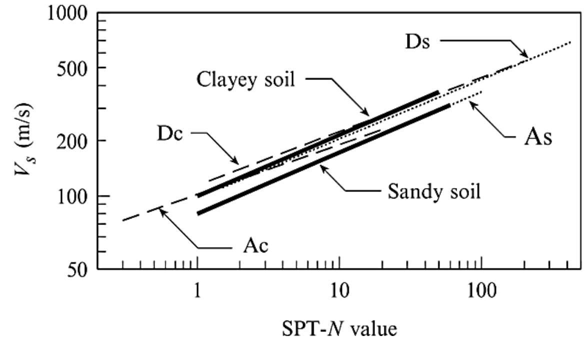

Figure 2. Comparison between Imai et al. and road bridge (After Japan Road Association 2002) (Thin line indicates Eq. 2, and thick line indicates Eq. 3)

3) Equations Developed for Port Facilities

Various empirical equations that are applicable for the remedial measures against soil liquefaction of fill material are introduced in Coastal Development Institute of Technology (1997). Among them, representative equations are Eq. (4) (sandy soil (Imai and Tonouchi 1982)) and Eq. (5) (Clayey soil (Zen et. al.1987)):

$$G_0=144N^{0.68} \tag{4}$$

$$G_0=170q_u \tag{5}$$

where q_u denotes unconfined compression strength. A rather good correlation between the elastic modulus and the shear strength is known to exist for clayey soils. In addition, Eq. (6) is introduced as confining stress dependency (Zen et al.1987; Uwabe et al. 1982):

$$V_s=V_{S0}\left(\frac{\sigma_v'}{\sigma_{v0}'}\right)^B \tag{6}$$

where V_{s0} and V_s denote the S-wave velocities before and after the construction, respectively, and σ_{v0}' and σ_v' are effective overburden stresses before and after the construction, respectively. Finally, exponent B is a power of the confining stress dependency and is defined as 0.25 for sandy and 0.5 for clayey soils. Thus, the elastic modulus of the sand is proportional to the square root of the effective confining stress and that of the clay is proportional with the effective stress itself. Here, equation for sandy soil is relevant for soils with plastic index Ip less than or equal to 30 and that for clayey soils with Ip greater than 30. In addition, Eq. (7) is shown in the design specification for port facilities (1989 version) (Ports and Harbours Bureau, Transport Ministry 1999):

$$G_0=98\left(285-2I_p\right)\sigma_m'$$

$$\left(I_p\geq 30\right)$$

$$G_0=9.90\frac{\left(1.6I_p+185\right)\left(2.973-e\right)^2}{1+e}\sigma_m'^{0.5} \tag{7}$$

$$G_0=6929\frac{(2.17-e)^2}{1+e}\sigma_m'^{0.5}$$

(Sandy Soil with round particle)

$$G_0=3267\frac{(2.973\text{-}e)^2}{1+e}\sigma'^{0.5}_m$$

(Gravelly Soil with angular particle)

4) Equations Frequently Used in Buildings Design

Ohta and Goto (1978) classified the indices that affect elastic modulus into four categories when developing an empirical equation. They are SPT N-value, depth, geologic age, and soil type. Here SPT N-value and depth are quantitative indices, whereas geologic age and soil type are qualitative indices which cannot be counted in the ordinary sense. A multivariate analysis is possible to evaluate best parameters if indices are composed of quantitative indices only, but it cannot be used when qualitative indices are involved. They developed an alternative procedure to consider both quantitative and qualitative parameters. Totally 300 data were compiled with V_s ranging from 50 to 620 m/s, SPT N-values between 2 and 200, and depth between 1and 80 m. The coefficient or correlation depends on the choice of the parameters for an individual analysis set. The highest coefficient of correlation was obtained when considering all four parameters, resulting in

$$V_s=68.79N^{0.171}H^{0.199}EF \quad (r=0.856) \tag{8}$$

where H denotes layer depth in meters and E and F are influence coefficients for the geologic age and the soil type, respectively, and are tabulated in Table 2. If effect of layer depth is neglected, then the following equation is obtained:

$$V_s=93.10N^{0.249}EF \quad (r=0.787) \tag{9}$$

Values of E and F are also shown in Table 2.Equation (8) is frequently referred in publications on the building design such as Building Research Institute, Land, Infrastructure and Transportation Ministry (2001).A simpler empirical equation derived by using a similar approach was proposed from Japanese Central Disaster Prevention Council (2005) as

$$V_s=112.73N^{0.256}EF \tag{10}$$

where E is 1.000, 1.223, and 1.379 for the Holocene, the Pleistocene, and the Tertiary age soils, respectively, and F is 1.000, 0.855, and 0.90 for clayey, sandy, or gravelly soils, respectively.

5) Equations by Iwasaki et. al.

Iwasaki et al. (1977) derived relationships between the Vs and SPT N-values through least square analysis on the borehole test results in the coastal area in Tokyo, Kawasaki, and Kobe as

$$\begin{aligned} &V_s=103N^{0.211} && \text{Holocene Sandy Soil (As)} && (N=1\text{-}30) \\ &V_s=143N^{0.0777} && \text{Holocene clayey Soil (Ac)} && (N=1\text{-}7) \\ &V_s=205N^{0.125} && \text{Pleistocene Sandy Soil (Ds)} && (N=5\text{-}500) \\ &V_s=172N^{0.183} && \text{Pleistocene clayey Soil (Ds)} && (N=2\text{-}200) \end{aligned} \tag{11}$$

Here, SPT N-values in the parenthesis resemble the region shown in Fig.3. They also reported that strain range in the elastic wave measurement is in the order of 105 near the ground surface and order of 108 at depth greater than 50 m. Note that in this saturation, the power in the SPT N-value is smaller than the prior empirical equations.

TABLE II. VALUES OF PARAMETERS OF EQS. (8) AND (9) EQUATION (8)

Geologic age	E	Soil type	F
Holocene	1	Clay	1.000
Pleistocene	1.303	Fine sand	1.086
		Medium sand	1.066
		Course sand	1.135
		Sand gravel	1.153
		Gravel	1.448
Equation (9)			
Geologic age	E	Soil type	F
Holocene	1.000	Clay	1.000
Pleistocene	1.448	Fine sand	1.056
		Medium sand	1.013
		Course sand	1.039
		Sand gravel	1.069
		Gravel	1.221

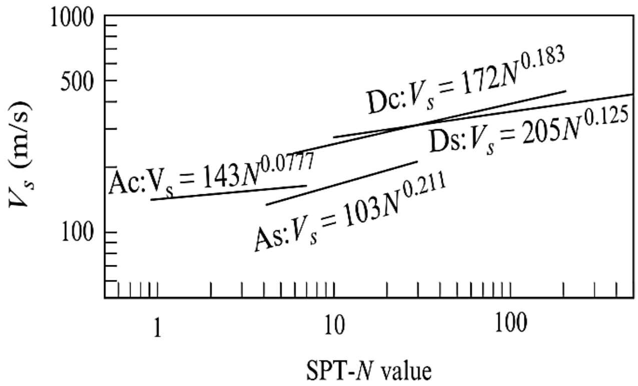

Figure 3. Relationships between SPT N-value and Vs shown by Iwasaki et al

6) Equations Based on Laboratory Tests

The elastic moduli obtained through the laboratory test by means of, so-called, undisturbed samples do not have sufficient accuracy. Therefore, the error for reconstituted sample is to be much larger than them. This indicates that the use of laboratory test results instead of the in situ elastic modulus is not suited for practical applications. In addition, it is difficult to obtain the effective mean stress σ'_m because the coefficient of earth pressure at rest K_0 is difficult to measure accurately. The soils employed in the shaking table or the centrifuge tests are fresh soils. Then, empirical equations based on laboratory test are applicable. In the same manner, they may be applicable to the filled soil as they are also freshly reconstituted soils. Many empirical equations have been proposed in the past, which are summarized in Table 3. Generally speaking, each empirical equation is expressed as functions of the void ratio e and the effective confining stress σ'_m

$$G_0=Af(e)\sigma'^{n}_m \tag{12}$$

where A denotes equation coefficient and f(e) denotes the function of the void ratio e. Effects resulting from the presence of the pores can also express by a relative density Dr , but the expression by means of the void ratio e is considered to be good for the clean sands (Adachi and Tatsuoka 1986). The term "2.17 e" appears frequently. This term appeared in the first study of this kind and the following studies seem to use the same term. As pointed out in Iida (1939), S-wave does not propagate in the sand when e is larger than 2, and this term seems to be consistent with it. When using the empirical equations, one needs to take into consideration the test method and the effective strain range for which the interested relationship is pertinent. According to Shibata and Soelarno (1975), elastic modulus obtained using the simple shear method is a half of that determined by the ultrasonic pulse test. It was very difficult to measure behaviors at small strains accurately in the old days by static test. Therefore, resonant column tests were frequently used. However, at present, small strains with 10^{-6} level can be measured according to the development of measurement devices. Many engineers feel that the damping ratio becomes more meaningful when shear strains are larger than 10^{-5} or 5×10^{-5} (JGS 1994). This may indicate that the accuracy may be taken less at small strain for nonacademic purposes. Figure.4 shows variation of the exponent n of the confining stress dependency (Kokusho 1980a). The value of n is less than 0.5 at the strain of 10^{-6}. It gradually reaches 0.5 at strains 10^{-5} –10^{-4} and becomes larger with strains. Therefore, values of the coefficient A as well as exponent n in Eq. (12) depend on the effective strain level of the test procedure. The grain size also affects the coefficient A for gravelly soils. On the other hand, n=0.5 is frequently used in the engineering practice. As shown in Fig.4, however, this n value corresponds to 10^{-5} –10^{-4} shear strain and do not resemble the elastic state. There is another method to evaluate appropriate value for the exponent n. As Vs, from Eq. (3), is proportional to $N^{1/3}$, elastic shear modulus is proportional to the effective confining stress with exponent 2/3. Note that confining stress dependency is not considered in Eq. (3). SPT N-value is proportional to the power of a half of the effective overburden stress when corrected SPT N-value N_1. In conclusion, the elastic shear modulus appears to be proportional to the effective confining stress with exponent of 1/3. This exponent seems to agree with Fig.4 at very small strain.

B. Nonlinear Properties

As mentioned, nonlinear soil properties are obtained through the cyclic shear deformation characteristics test and the obtained results are usually expressed in terms of the G–γ and the h– γ relationships. A schematic figure of a typical cyclic shear deformation characteristics test result is shown in Fig.5. Two sets of test data, denoted as L and M, are shown in the figure. In practice, however, the maximum value of test results may become slightly smaller than 1.0 when the maximum shear modulus is evaluated using, for example, the Hardin–Drnevich model, and it may be a little larger than 1.0 when, for example, the shear modulus at the smallest shear strain is used as denominator of the shear modulus ratio. On the other hand, the damping ratio is written in the right ordinate. It is expressed as ratio or in percent. The larger shear modulus ratio performance (L in the figure) can be interpreted as the nonlinear behavior appears at larger strain range than the smaller shear modulus ratio data such that for M. Therefore, the damping ratio of data L is generally smaller than that of data M. Shear strain at G/G_0=0.5 is often denoted as $\gamma_{0.5}$ and is frequently used to be a representative value for the G/G_0 – γ curve. It is used in many mathematical models such as Hardin–Drnevich model, hyperbolic model, and Romberg–Osgood model, in which it is termed the reference strain and is designated as γ_r. The damping ratio at small strains is sometimes close to 0, while in some cases finite value 2–4 %. As the maximum damping ratio scatters from less than about 0.1 (10 %) to nearly 0.4 (40 %) depending on soil type, the maximum value of the left ordinate may significantly differ from figure to figure. It may be fixed to 1.0 (100 %), 0.6 (60 %), 0.4 (40 %), 0.3 (30 %), or 0.2 (20 %) for various purposes. Coordinate axis becomes the same in both left and right ordinates when 1.0 is used. As a maximum value of 0.6 is close to theoretical maximum value (2/π), almost all data can be drawn when coordinate axis is set to 0–0.6, whereas data is better outlined when the maximum of 0.2–0.4 is chosen.

1) Equations by PWRI

Public Work Research Institute, Land, Infrastructure, and Transportation Ministry of Japan carried out series of test on various soils and reported for separate soil types: Holocene clay (Iwasaki et al. 1979, 1980a), Pleistocene clay (Yokota and Tatsuoka1982), and sand (Iwasaki et al. 1980b); overall results are compiled in PWRI (1982).

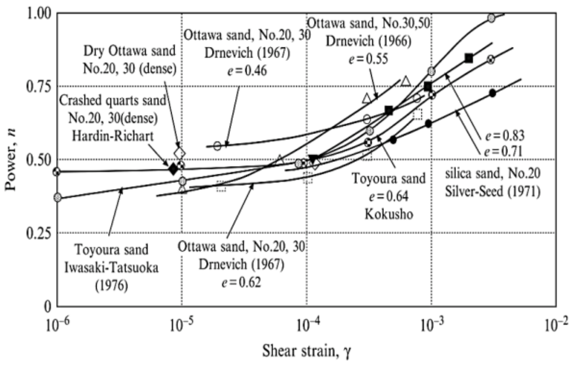

Figure 4. Confining stress dependency of shear modulus exponent (After Kokusho 1980)

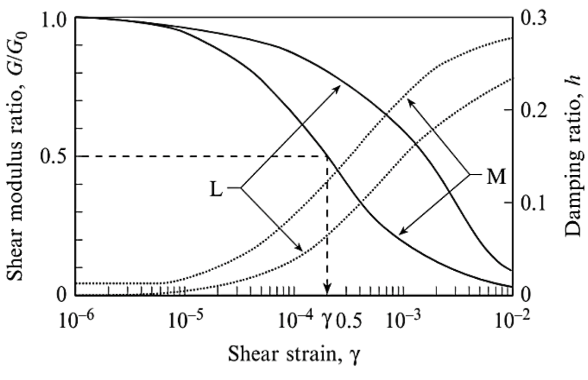

Figure 5. Example of cyclic shear deformation characteristics [30].

TABLE III. EMPIRICAL EQUATIONS FOR ELASTIC SHEAR MODULUS OBTAINED THROUGH LABORATORY TEST

	References	A	f(e)	n	Sample	Test
Sand	Hardin and Richart (1963)	7000	$(2.17-e)^2/(1+e)$	0.5	Round Ottawa sand	R
	Shibata and Soelarno (1975)	42000	$0.67-e/(1+e)$	0.5	Three clean sands	P
	Iwasaki et al. (1978)	9000	$(2.17-e)^2/(1+e)$	0.38	11 clean sands	R
	Kokusho (1980b)	8400	$(2.17-e)^2/(1+e)$	0.5	Toyoura sand	T
	Yu and Richart (1984)	7000	$(2.17-e)^2/(1+e)$	0.5	Three clean sands	R
	Lo Presti et al.(1997)	9014	$e^{-1.3}$	0.45	Toyoura sand	H,R
	Numata et al. (2000)	29718	$(0.79-e)/(1+e)$	0.55	Three clean sands	T
	Asonuma et al.(2002)	10276	$e^{-2.46}$	0.52	Undisturbed Shirasu	T
	Saxena and Reddy (1989)	3062	$1/(0.3+0.7e^2)$	0.574	Monterey sand	R
	Hardin and Black (1968)	31.5	$(32.17-14.8e)^2/(1+e)$	0.5	Ottawa sand,	R
Clay	Hardin and Black (1968)	3300	$(2.97-e)^2/(1+e)$	0.5	Kaolinite et al.	R
	Marcuson and Wahls (1972)	4500	$(2.97-e)^2/(1+e)$	0.5	Kaolinite, Ip=35	R
	Zen et al. (1978)	2000-4000	$(2.97-e)^2/(1+e)$	0.5	Disturbed clay,Ip=0-50	R
	Kokusho et al. (1982)	141	$(7.32-e)^2/(1+e)$	0.6	Undisturbed clay,Ip=40-85	R
Organic	Ishihara et al. (2003)	0.236	$(91.5-e)^2/(1+e)$	0.65	Undisturbed organic soil	R
Gravelly Soil	Prange (1981)	7230	$(2.97-e)^2/(1+e)$	0.38	Ballast,D_{50}=10.7 mm,U_c=13.8	R
	Kokusho and Esashi (1981)	13000	$(2.17-e)^2/(1+e)$	0.55	Quarry,D_{50}=30mm,Uc=10	T
	Tanaka et al. (1987)	3080	$(2.17-e)^2/(1+e)$	0.6	Gravel,D_{50}=10mm,Uc=2	T
	Goto et al. (1987)	1200	$(2.17-e)^2/(1+e)$	0.85	Gravel,D_{50}=10.7mm,Uc=13.8	T
	Nishio et al. (1985)	9360	$(2.17-e)^2/(1+e)$	0.44	Undisturbed gravel,D_{50}=10.7mm,Uc=13.8	T
	Asonuma et al. (2002)	2488	$e^{-0.56}$	0.68	Volcanic ash soil, D_{50}=6.6 mm,Uc=6	T
	Tanaka et al. (1985)	2056	$(2.17-e)^2/(1+e)$	0.62	Gravel contents=25 %	T
	Nishi et al. (1983)	2393	$(2.17-e)^2/(1+e)$	0.66	Gravel contents=50 %	T

Test methods: R resonance column, T triaxial, P pulse, and H torsional shear tests

a) Holocene Clayey Soils

Undisturbed clay soil samples at the Kawasaki City and the Nagoya City were consolidated by an in situ overburden stress and were tested by means of cyclic triaxial test. The shear modulus reduction curve is shown in Eq. (13):

$$\frac{G}{G_0}=\left[A.\sigma_m'^{B}\right]_{\gamma=\gamma_i} \qquad (10^{-6}\leq\gamma\leq5\times10^{-4})$$

$$\frac{G}{G_0}=\left[A.\sigma_m'^{B}\right]_{\gamma=5\times10^{-4}}.\left[K\right]_{\gamma=\gamma_i}\ (5\times10^{-4}\leq\gamma\leq2\times10^{-2}) \qquad (13)$$

where σ_m' is the mean effective stress, and the values for coefficients A, B, and K are tabulated in Table 4. On the other hand, only one averaged curve, shown in Fig. 6 is shown for the damping ratio because of the limited number of data. It is read as in Table 5 (PWRI 1982). Example of cyclic shear deformation characteristics for various mean effective stresses is shown in Fig.6.

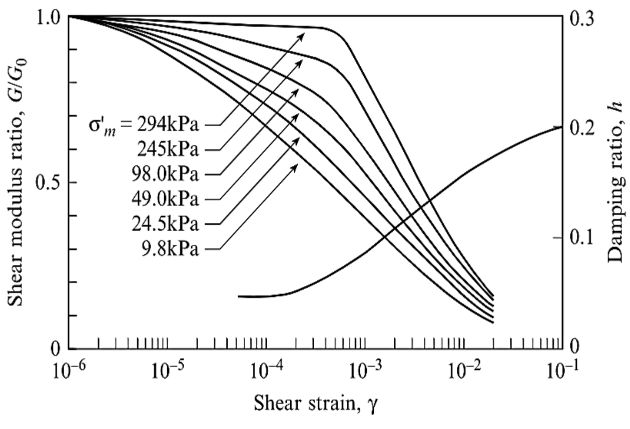

Figure 6. Cyclic shear deformation characteristics of Holocene clay [30].

TABLE IV. G-Γ RELATIONSHIP FOR HOLOCENE CLAY

γ	A	B	γ	K
2×10^{-6}	0.979	0.00258	5×10^{-4}	1.000
5×10^{-6}	0.896	0.016	10^{-3}	0.831
10^{-5}	0.826	0.0275	2×10^{-3}	0.655
2×10^{-5}	0.74	0.0443	5×10^{-3}	0.431
5×10^{-5}	0.617	0.0727	10^{-2}	0.282
10^{-4}	0.515	0.101	2×10^{-2}	0.170
2×10^{-4}	0.43	0.129	5×10^{-2}	0.06
5×10^{-4}	0.301	0.185	10^{-1}	0.03

TABLE V. H-Γ RELATIONSHIP FOR HOLOCENE CLAY

γ	h	γ	h
10^{-6}	0.02	5×10^{-4}	0.073
2×10^{-6}	0.023	10^{-3}	0.092
5×10^{-6}	0.028	2×10^{-3}	0.110
10^{-5}	0.032	5×10^{-3}	0.140
2×10^{-5}	0.036	10^{-2}	0.161
5×10^{-5}	0.044	2×10^{-2}	0.176
10^{-4}	0.05	5×10^{-2}	0.192
2×10^{-4}	0.057	10^{-1}	0.200

b) Pleistocene Clayey Soils

Soils sampled at the Nagoya City were tested and compiled. Samples are composed of low to medium plastic clay with the SPT N-value ranging between 15 and 35 and Vs ~300 m/s. Both resonant column test and cyclic torsional shear test (low-frequency test) are performed. Initial axial stress was set same with the in situ overburden stress, and lateral stress was set half of the overburden stress to reproduce the in situ stress state. Test results are summarized in Fig.7. Data of the Toyoura sand is also shown in the figure. Shear modulus reduction curves for clay and Toyoura sand with high initial stress (196 kPa) are similar to each other. There are slight differences between the resonant column test and the cyclic torsional shear test results; they do not lie in one line.

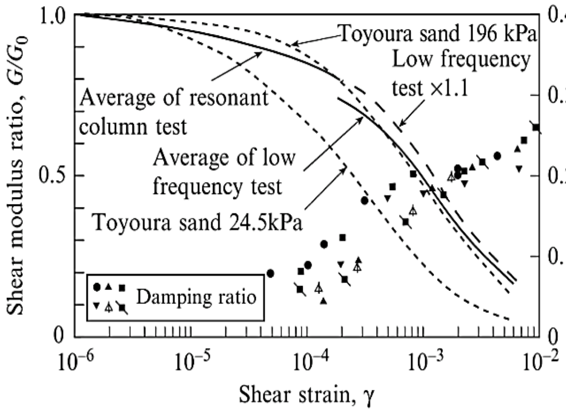

Figure 7. Cyclic shear deformation characteristics of Pleistocene clay

However, if the shear modulus ratio obtained through the torsional shear test result is multiplied by 1.1, resultant curve seems continuous from resonant column test result; hence, they suggest using this curve for shear modulus reduction curves for Pleistocene clay. Average damping ratio is employed the same as that for Holocene clay.

2) Equations Involved in Technical Standards for Port and Harbor Facilities

Design specification for port facilities (1989 version) (The Overseas Coastal Area Development Institute of Japan 1989) expressed shear modulus reduction curve as a function of the shear strain amplitude γ and the plasticity index Ip:

$$\frac{G}{G_{max}} = \bar{A}(I_p,\gamma)\left(\frac{\sigma'_m}{98}\right)^{n(I_p,\gamma)} \quad (14)$$

where $\bar{A}(I_p,\gamma)$ and $n(I_p,\gamma)$ are given in Table 6.

TABLE VI. $\bar{A}(I_p,\gamma)$ AND $n(I_p,\gamma)$

Shear strain amplitude γ	Plasticity index, Ip					
	Ip less than 9.4		9.4 less than 30		More than or equal to 30	
	A (Ip , γ)	n(Ip , γ)	A (Ip , γ)	n(Ip , γ)	A (Ip , γ)	n(Ip , γ)
10^{-6}	1	0	1	0	1	0
10^{-5}	0.93	0.01	0.96	0	0.97	0
5×10^{-5}	0.83	0.03	0.91	0.01	0.93	0
10^{-4}	0.75	0.05	0.84	0.02	0.89	0
2.5×10^{-4}	0.56	0.1	0.74	0.05	0.82	0
5×10^{-4}	0.43	0.16	0.59	0.09	0.70	0
10^{-3}	0.30	0.22	0.45	0.16	0.58	0
2.5×10^{-3}	0.15	0.30	0.26	0.22	0.40	0
5×10^{-3}	-	-	0.12	0.26	0.25	0
10^{-2}	-	-	-	-	0.18	0

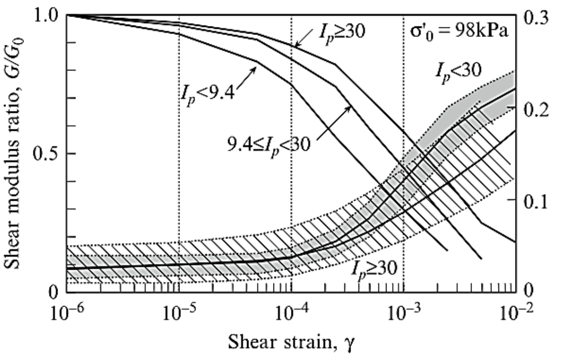

Figure 8. Cyclic shear deformation characteristics described by technical standards for port and harbor facilities [30].

Data on Ip < 30 is relevant when the equations are adopted for sands. Note that Ip=30 is used as a boundary between sandy and clayey soils. However, in reality, the relation between sandy soils and Ip is not unique. For example, sands with clay are classified as sandy soil, but Ip for this soil type is usually large. According to the author's experience, in practice, it is likely that Ip for sands can exceed 30. The cyclic shear deformation characteristics at σ'_m =98 kPa are shown in Fig.8. In Table .6, n(Ip ,γ) = 0 indicates that the confining stress dependency is not considered or observed. Generally speaking, confining stress dependency becomes small when Ip is larger than 30–40. Compared with the results by PWRI, the shear modulus reduction curve shows similar shape. Nonetheless, there exists a significant difference between the damping ratios at large strains induced by the difference in the drainage conditions. In addition, some scattering is seen on the damping ratio, whereas unique value is given by the PWRI data as there is no significant difference between soils.

3) Study Compiled by Seed and Idriss

Seed and Idriss gathered and compiled various test data (Seed and Idriss 1970). Range and mean of the reviewed data are shown in Fig.9. The effect of the confining stress, the internal friction angle, saturation ratio, and the coefficient of earth pressure at rest are discussed for sand. Among them, the effect of confining stress dependency hows very similar with previously described stress dependency behavior. The shear modulus ratio increases and the damping ratio decreases with an increase in internal friction angle. The effect of K_0 is minor, but strains exhibiting nonlinear behavior are seen to be largest when K_0=1. They also showed that evaluation of accurate cyclic shear deformation characteristics is difficult for clayey soil partly because sample disturbance has a very considerable effect on the test results and partly because there is no available in situ technique to measure the deformation characteristics at large strain. The mean values of the modulus reduction curve are shown in Fig.10a; individual test data and existing range are shown in Fig.10b. Here, the shear modulus is normalized by the shear modulus at 3×10^{-6} strain; thus, the shear moduli at strains smaller than 3×10^{-6} are larger than unity. As compared with the data described in this book as well as other data, shear modulus ratio in Fig.10a is very small, which indicates that the nonlinear behavior begins to develop at smaller strains. Therefore, use of this data should be performed with care.

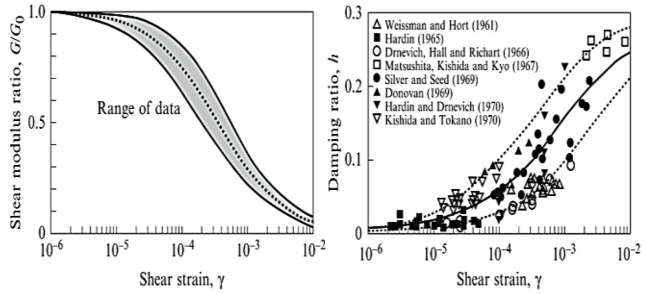

Figure 9. Cyclic shear deformation characteristics of sand

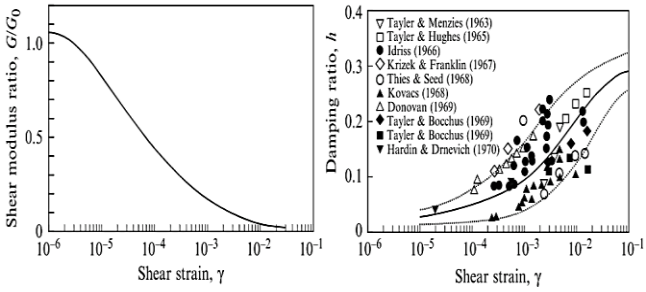

Figure 10. Cyclic shear deformation characteristics of saturated clay

4) Study by Vucetic and Dobry

Vucetic and Dobry (1991) gathered 16 technical papers on the cyclic shear deformation characteristics for various soils and concluded that the plasticity index is the most predominant factor that affects the cyclic deformation characteristics. They compiled the gathered data as a function of Ip, as shown in Fig.11. However, they sometimes do not agree well with those in Japan. For example, soils with Ip=0–15 are clearly classified as sand, but confining stress dependency is not considered. Shear modulus ratio around strain 10^{-2} is very small compared with other data.

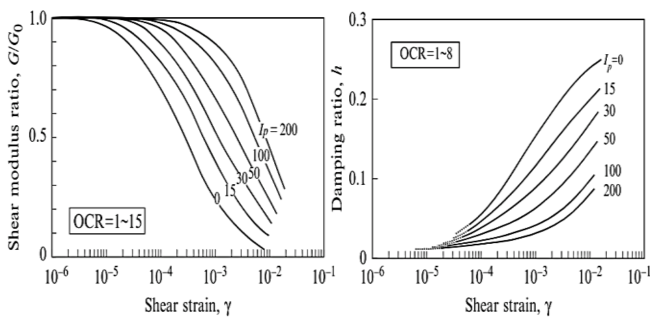

Figure 11. Cyclic shear deformation characteristics with Ip (After Vucetic and Dobry 1991)

IV. CONCLUSION

Along with source and path effects, site response analysis is a vital component of earthquake ground motion prediction. Semi-empirical ground motion prediction equations (GMPEs) include terms for modeling site response that are based on simple metrics of site condition, such as the time-average shear-wave velocity in the upper 30 m of the site (V_{S30}). Because site terms in GMPEs are derived from global ground motion databases and are based on incomplete information on the site condition, their predictions represent average levels of site response observed at sites conditional on V_{S30}. Most of the investigation presented in this paper concerns recommendations for determination of static and dynamic soil properties for performing GRA and using the results of those analyses to develop hazard-consistent estimates of site-specific ground motions. Some important aspects of these recommendations include the following:

1. Shear-wave velocity profiles should be based on measurements, not estimates;

2. Nonlinear modulus reduction and damping versus strain curves can be derived from material-specific tests or generic relationships derived from test databases, but these relationships are generally not reliable at strains beyond about 0.3-0.5%;

3. The shear strength of soil should be considered in developing modulus reduction (MR) relationships at large strains;

4. Equivalent-linear methods of GRA should be used for small- to moderate-strain problems, and diagnostics are presented for identifying when such methods become unreliable;

5. For each depth interval in a discretized soil column, dynamic soil properties are needed to describe the shape of the backbone (i.e., shear stress versus shear strain, τ-γ) curve and the relationship between hysteretic damping ratio and shear strain (D- γ curves).

6. When empirical models for G/G_{max}- γ and D- γ curves are used, difficulties are encountered for sites that mobilize large strains, requiring the use of shear strength as an addition parameter explicitly considered in the analysis.

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