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INFLUENCE OF POST-STRONG SEISMIC SHAKING ON THE RESIDUAL STRENGTH OF SATURATED SAND

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ABSTRACT

Several actual earthquake records recently obtained are analyzed and the post-strong seismic shaking is characterized using parameters such as number of cycles and average shear stress. These parameters are integrated with laboratory test results from the Japanese standard Toyoura sand, and applied to an ideal sandy soil profile to estimate the reduction of residual strength. Different levels of strength reduction are obtained depending on the characteristics of the post-strong seismic motion and sand relative density. Steady-state concepts are shown to overestimate the residual strength of saturated sands and provide dangerous evaluation of post-seismic stability analysis.

INTRODUCTION

Traditionally, the residual strength of earthquake-induced liquefied sand has been determined directly from the results of consolidated-undrained, strain-controlled triaxial-compression monotonic tests (Castro, 1975; Poulos, 1981; Castro et al., 1992, Ishihara, 1993). Based on an experimental program, the residual strength is the steady-state strength at which the soil will deform continuously without change in the resistance to deformation. The steady state theory neglects consideration of the likely magnitude and duration of relevant seismic events. For the case of liquefaction caused by earthquakes, it is hypothesized that the cyclic stresses do not play any role other than triggering liquefaction. Once liquefaction is triggered, the only stress acting on the liquefied soil mass is the static stress which drives the mass downwards. Consequently, the possibility of a soil mass flowing under seismic shaking is ignored. Steady-state concepts may be applicable for conditions other than liquefaction induced by earthquakes. For this case, they may lead to nonconservative assessments and may assign larger strength to soils.

In contrast to these assumptions, it is hypothesized in this study, that the post-strong seismic shaking plays an important role in the behavior of liquefied sand and its undrained strength. The post-strong seismic shaking is the sequence of small seismic shear stresses following the large amplitude seismic stresses that trigger liquefaction. The effects of these post-strong seismic shear stresses on the residual strength of Toyoura sand will be evaluated.

This evaluation is supported by the experimental program that included hollow cylindrical torsional and cyclic simple shear tests reported by Meneses et al. (1998, 1999, 2000). Meneses et al. (1998) extensively studied the behavior of saturated Toyoura sand subjected to simultaneous monotonic and cyclic stresses. Meneses et al. (1999) proposed the conditions of contractiveness and dilativeness of sand under the proposed loading scheme. Finally, Meneses et al. (2000) estimated the residual strength and discussed the implications for the steady-state theory. For the purpose of this study, it is assumed that the superimposed cyclic stresses of this experimental program represent the post-strong seismic shaking.

Several researchers in the past (Seed, H.B. and I.M. Idriss, 1982; Towhata et al., 1996; Igarashi, 1996) have studied liquefaction of sand using strong motion records, and have proposed prediction methods. Emphasis has been placed on the determination of the cyclic strength, triggering of liquefaction, and prediction of liquefaction. However the use of strong motion records for the estimation of residual strength of saturated liquefied sands has not been considered. In other words, the estimation of the residual strength of **earthquake-induced** liquefied sands used to completely neglect the relevant earthquake characteristics. These were taken into account only for the study of the so-called "cyclic mobility" phenomenon. So when cyclic stresses were applied in an experimental program, this was related to cyclic strength of sand but when monotonic stresses were applied, it was related to residual strength. Assuming that the strong phase of an earthquake triggers liquefaction, this paper attempts to include the post-strong seismic phase into the estimation of the residual strength of earthquake-induced liquefied sand, and provide a new insight into liquefaction and related phenomena.

CHARACTERIZATION OF THE POST-STRONG SEISMIC SHAKING FROM ACTUAL EARTHQUAKE RECORDS

A preliminary suggestion for this characterization is explained in Fig. 1 (Meneses, 1996). First, it is necessary to identify the strong shaking phase of the earthquake record that is distinguished by high levels of accelerations. Then, the post-strong seismic shaking is recognized as the subsequent sequence of small amplitudes until the end of the earthquake.

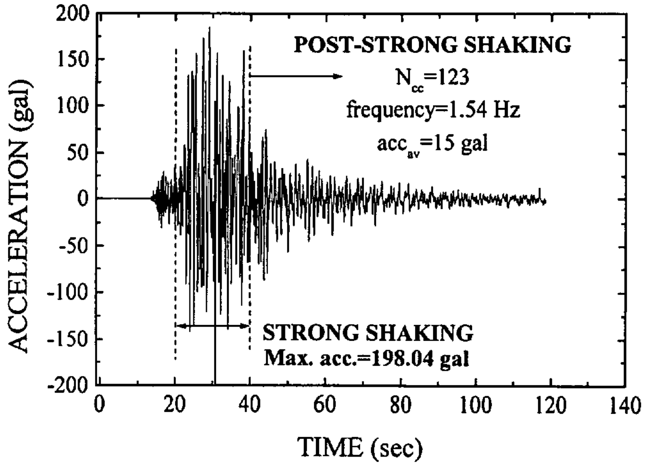

Fig. 1. Characterization of the post-strong seismic shaking, 1995 Hyogo-ken Nambu earthquake, Takasago site

Table 1 - Characterization of the post-strong seismic shaking

		Entire Seismic Record		Post-strong seismic shaking				
Earthquake	Component	Max. acc. (gal)	Duration (sec)	N_{cc}	Frequency (Hz)	Duration (sec)	acc_{av} (gal)	TSR
Miyagi-ken Oki, 1978(1)	T	270.27	40	72	3.6	20	20	0.058
Miyagi-ken Oki, 1978(1)	L	190.53	40	126	5	26	20	0.058
Wildlife, 1978 (2)	N-S	210.00	98	30	0.42	73	15	0.044
Hyogo-ken Nambu, 1995 (3)	E-W	524.77	30	49	2.13	23	20	0.058
Hyogo-ken Nambu, 1995 (4)	N-S	341.22	48	39	1.77	23	10	0.029
Hyogo-ken Nambu, 1995 (5)	E-W	198.04	120	123	1.54	80	15	0.044
Kocaeli, Turkey, 1999 (6)	T	322.20	135.81	41	0.48	85.81	10	0.029
Loma Prieta, 1989 (7)	CAP000	550.00	39.95	36	1.76	20.45	20	0.057
Northridge, 1994 (8)	090	1780.00	39.98	47	2.14	21.98	40	0.11
Chi-Chi, Taiwan, 1999 (9)	E-W	249	89.95	40	1.54	25.995	18	0.052

(1) Kaihoku site
(2) Superstition Hills
(3) Kobe Harbour
(4) Port Island, vertical array
(5) Takasago site
(6)Yarimca (YPT) station
(7)Capitola 47125 station
(8)Tarzana, Cedar Hill 24436
(9)TCU070 station

Once the post-strong shaking is identified, the number of zero-crossings is computed and the number of cycles, N_{cc}, is determined. The average uniform amplitude, acc_{av}, is estimated from the average of peak amplitudes, and then the cyclic stress is defined as follows:

$$\tau_{av}/\sigma'_v = r_d\,(acc_{av}/g)\,(\sigma_v/\sigma'_v) \qquad (1)$$

$$r_d = 1 - 0.015z \qquad (2)$$

where τ_{av} = average shear stress, σ'_v = effective vertical stress, r_d= stress reduction factor at depth of interest *z*, g= gravity, and σ_v = total vertical stress.

The cyclic stress ratio TSR is computed considering an ideal 10m deep soil profile of saturated Toyoura sand (γ_{sat} = 17.66 kN/m^3). So, if z=10m, from equation (2), we have: r_d = 1-0.015(10) = 0.85, and σ_v / σ'_v = 2.25. Then replacing these values into equation (1), we obtain:

$$\tau_{av}/\sigma'_v = 0.85\,(acc_{av}/g)\,2.25 = 1.9122\,(acc_{av}/g) \qquad (3)$$

Hence, the cyclic stress ratio, TSR, is defined as:

$$TSR = \tau_{av}/\sigma'_o = \tau_{av}/\,(2/3\,\sigma'_v) = (3/2)\,\tau_{av}/\sigma'_v \qquad (4)$$

Then equation (3) is replaced into equation (4) and the cyclic stress ratio is computed as:

$$TSR = 2.8683\,(acc_{av}/g) \qquad (5)$$

Finally, the frequency is evaluated as:

$$f = N_{cc} / t_d \qquad (6)$$

Ten earthquake records are analyzed following the procedure explained in Fig. 1. These records were obtained at sites where liquefaction occurrence was reported nearby. The names of the earthquakes, recording stations and components along with the characteristics of the entire record and the post-strong shaking phase are listed in Table 1. For the entire seismic record, the maximum acceleration and duration are given while for the post-strong seismic phase, the number of cycles N_{cc}, frequency, duration, uniform amplitude acc_{av}, and cyclic stress ratio TSR are shown. The number of cycles N_{cc} vary from 30 to 126, the frequencies from 0.42 to 3.6 Hz, the duration of the post-strong seismic shaking from 20 to 85.81 sec, the amplitude acc_{av} from 10 to 40 gals, and TSR from 0.029 to 0.11.

RESIDUAL STRENGTH REDUCTION FACTOR, R_D, NUMBER OF CYCLES, $N_{1\%}$, AND CYCLIC STRESS RATIO, TSR

An estimation of the residual strength of saturated Toyoura sand when subjected to simultaneous monotonic and cyclic stresses is given by Meneses et al. (2000). This is expressed as a function of the residual strength obtained from tests where monotonic stress was applied alone, as follows:

$$(S_{us}/\sigma'_v)_{cc} = (S_{us}/\sigma'_v)_{ml} * R_D \tag{7}$$

where S_{us}= residual strength; subscript cc = denotes the inclusion of the effects of the cyclic stresses; subscript ml = indicates that monotonic loading is applied alone; and R_D is a strength reduction factor that takes into account the influence of the cyclic stresses and affects the undrained strength obtained from monotonic tests. This reduction factor is defined as:

$$R_D = (p'_{PT})_{cc} / (p'_{PT})_{ml} \ ; \ 0 \le R_D \le 1 \tag{8}$$

where p'_{PT} = mean effective stress at phase transformation. R_D depends on the void ratio after consolidation, e_c, the number of cycles per 1% of shear deformation in the direction where the monotonic loading is applied, $N_{1\%}$, and TSR of the cyclic stress that is simultaneously applied with the monotonic stress. Eq. (7) indicates that the strength calculated from tests in which monotonic shear stress is applied alone constitutes the upper bound of the minimum strength, i.e., $R_D = (p'_{PT})_{cc} / (p'_{PT})_{ml} = 1.0$, and that the lower bound may be extremely low and even zero, depending upon TSR and $N_{1\%}$ of the cyclic stress.

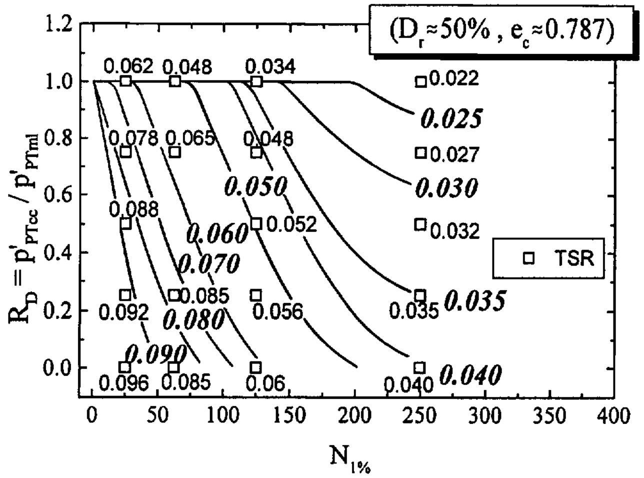

Fig. 3. Strength Reduction Factor R_D versus $N_{1\%}$ for Different Levels of TSR (D_r = 50%) (Meneses et al., 2000)

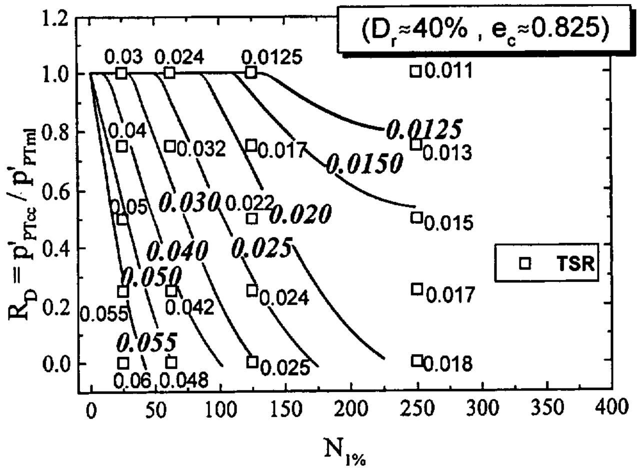

Fig. 2. Strength Reduction Factor R_D versus $N_{1\%}$ for Different Levels of TSR (D_r = 40%) (Meneses et al., 2000)

Meneses et al. (2000) show the variations of the strength reduction factor, R_D, versus the number of cycles, $N_{1\%}$, for two different relative densities D_r = 40 and 50%, respectively (Figs. 2 and 3). To attain the same value of R_D, as the number of cycles $N_{1\%}$ decreases, TSR should increase. Looser sand samples require lower values of TSR to reduce R_D while denser samples require higher values.

VARIATIONS OF R_D DUE TO POST-STRONG SEISMIC SHAKING

In the experimental loading scheme proposed by Meneses et al. (1998) and Meneses et al. (2000), the superimposed cyclic stresses represented the minor seismic shaking following the strong shaking that is still acting on a liquefied sand driven by the in situ static stress. This minor shaking is the post-strong seismic shaking already identified in the earthquake records shown in Table 1. Using TSR and assuming two different relative densities 40 and 50% for the 10m deep ideal Toyoura sand soil profile, it is possible to define ranges of variations of R_D from Figs. 2 and 3. Since, at this point, there is not a clear correspondence between N_{cc} and $N_{1\%}$, it is proposed ranges of variations of $N_{1\%}$ and strain rates for each of the three levels of R_D as shown in Tables 2 and 3. Strain rates are calculated as the ratios of frequency to $N_{1\%}$ (strain rate = $f / N_{1\%}$). These ranges are shown in Tables 2 and 3 for relative densities of 40 and 50% respectively.

Knowing the value of TSR, it is possible to define the range of variation of $N_{1\%}$ for which $R_D = 1.0$, $0.0 < R_D < 1.0$, and $R_D = 0.0$ respectively. For example, from Fig. 2, for a TSR=0.029, R_D equals 1.0 when $N_{1\%}<25$; R_D is between 0 and 1 ($0<R_D<1$) when $25<N_{1\%}<125$; and finally R_D equals 0.0 when $N_{1\%}>125$. Table 2 (D_r = 40%) shows the three ranges of strength reduction that the ideal soil element at z = 10m may experience when subjected to the selected earthquake records. All values are evaluated from Fig. 2. It is observed that under most of the analyzed earthquake records the soil element reduces its strength. Except the Kocaeli and Hyogo-ken Nambu earthquake, recorded at Port Island (vertical array), the others always induce residual strength reduction, i.e., $0 \le R_D \le 1$. As $N_{1\%}$ increases, the strength reduction becomes more pronounced. The post-strong phase of the Northridge earthquake record shows a high TSR and therefore will make the sand always achieve the condition of zero residual strength.

Table 3 (D_r = 50%) shows different ranges of $N_{1\%}$ for three different levels of residual strength reduction. As the sand becomes denser, the ranges of $N_{1\%}$ with no strength reduction (R_D=1.0) become wider, i.e., the number of conditions to be unaffected by the post-strong seismic shaking increases. Due to the low amplitude TSR of the seismic record in Port Island (vertical array), the sand does not have possibilities to achieve the zero residual strength condition for $N_{1\%}$-values smaller than 250 that is the maximum value used by Meneses et al. (2000).

Table 2 – Ranges of Strength Reduction for Toyoura Sand with Relative Density D_r = 40%

		$R_D = 1.0$		$0.0< R_D <1.0$		$R_D = 0.0$	
Earthquake	Comp.	$N_{1\%}$	Strain Rate (%/min)	$N_{1\%}$	Strain Rate (%/min)	$N_{1\%}$	Strain Rate (%/min)
Miyagi-ken Oki, 1978	T			<48	>4.50	>48	<4.50
Miyagi-ken Oki, 1978	L			<48	>6.25	>48	<6.25
Wildlife, 1987	N-S			<75	>0.336	>75	<0.336
Hyogo-ken Nambu, 1995 (K.H.)	E-W			<48	>2.84	>48	<2.84
Hyogo-ken Nambu, 1995 (P.I.)	N-S	<25	> 4.25	25-125	0.85-4.25	>125	<4.25
Hyogo-ken Nambu, 1995 (T.)	E-W			<75	>1.23	>75	<1.23
Kocaeli Turkey, 1999	T	<25	>4.25	25-125	0.85-4.25	>125	<4.25
Loma Prieta, 1989	CAP000			<49	>2.16	>49	<2.16
Northridge, 1994	090					>0	Any
Ji-ji, 1999	E-W			<70	>1.32	>70	<1.32

Table 3 – Ranges of Strength Reduction for Toyoura Sand with Relative Density $D_r = 50\%$

		$R_D = 1.0$		$0.0< R_D <1.0$		$R_D = 0.0$	
Earthquake	Comp.	$N_{1\%}$	Strain Rate (%/min)	$N_{1\%}$	Strain Rate (%/min)	$N_{1\%}$	Strain Rate (%/min)
Miyagi-ken Oki, 1978	T	<40	>5.4	40-150	1.44-5.4	>150	<1.44
Miyagi-ken Oki, 1978	L	<40	>7.5	40-150	2-7.5	>150	<2.0
Wildlife, 1987	N-S	<85	>0.3	85-230	0.3-0.11	>230	<0.11
Hyogo-ken Nambu, 1995 (K.H.)	E-W	<30	>4.26	30-150	0.85-4.26	>150	<0.85
Hyogo-ken Nambu, 1995 (P.I.)	N-S	<150	>0.71	>150	<0.71		
Hyogo-ken Nambu, 1995 (T.)	E-W	<85	>109	85-230	0.4-1.09	>230	<0.40
Kocaeli Turkey, 1999	T	<150	>0.192	>150	<0.192		
Loma Prieta, 1989	CAP000	<45	>2.35	45-155	0.68-2.35	>155	<2.35
Northridge, 1994	090			<12	>10.7	>12	<10.7
Ji-ji, 1999	E-W	<55	>1.68	55-200	0.46-1.68	>200	<0.46

Because sand is denser, the post-strong phase of Northridge earthquake will make sand achieve the zero strength condition only when $N_{1\%}$ is greater than 12. Tables 2 and 3 also show the probably generated strain rates calculated as frequency/$N_{1\%}$. However, these strain rates should be regarded as reference values and not as the strain rates under which sand would flow.

CONCLUSIONS AND RECOMMENDATIONS

Inclusion of earthquake records into the estimation of the residual strength of saturated liquefied sands is attempted. Using several earthquake records and identifying the post-strong phase, different levels of residual strength reduction have been estimated in an ideal soil profile. The number of cycles and the average cyclic stresses of the post-strong seismic shaking play an important role in the reduction of residual strength estimated from monotonic tests. It is recommended to further improve the characterization of the post-strong phase of an earthquake record and its representation in an experimental program.

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REFERENCES

Castro, G. [1975]. Liquefaction and cyclic mobility of saturated sands, J. of Geotech. Engrg., ASCE, No. 101(6), pp. 551-569.

Castro, G., Seed, R.S., Keller, T.O. and Seed, H.B. [1992]. Steady-state strength analysis of lower San Fernando Dam slide. J. of Geotech. Engrg., ASCE, No. 118(3), pp. 406-427.

Ishihara, K. [1993]. Liquefaction and flow slide during earthquakes. Geotechnique, No. 43(3), pp. 351-415.

Meneses, J.F. [1996]. Effects of complementary cyclic shear stress on the undrained behavior of saturated sand under monotonic loading. Doctoral thesis, Dept. of Civ. Engrg., Univ. of Tokyo, Tokyo.

Meneses, J.F., K. Ishihara and I. Towhata [1998]. Effects of superimposing cyclic shear stress on the undrained behavior of saturated sand under monotonic loading. Soils and Found., Tokyo, No. 38(4), pp. 115-127.

Meneses, J.F., K. Ishihara and I. Towhata [1999]. Effects of continuous low-amplitude cyclic stress on the undrained monotonic behavior of Toyoura sand. Proc. 13th ASCE Engrg. Mech. Div. Conf., Baltimore, MD.

Meneses, J.F., K. Ishihara and I. Towhata [2000]. Flow failure of saturated sand under simultaneous monotonic and cyclic stresses. J. Geotech. and Geoenvir. Engrg., No.126(2), pp. 131-

138.

Poulos, S.J. [1981]. The steady state of deformation. J. of Geotech. Engrg., ASCE, No. 107(5), pp. 553-562.

Seed, H.B. and I.M. Idriss [1982]. Ground motions and soil liquefaction during earthquakes. EERI, Berkeley, California.

Towhata, I., J.K. Park, R.P. Orense and H. Kano [1996]. Use of spectrum intensity for immediate detection of subsoil liquefaction. Soils and Found., Tokyo, No. 36(2), pp. 29-44.