Pseudo static analysis considering strength softening in saturated clays during earthquakes

L'analyse pseudo statique considérant la force de ramollissement dans l'argile saturée lors des tremblements de terre

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ABSTRACT: Cyclic softening and strength loss of saturated clays during earthquakes is often an important consideration in engineering problems such as slope stability, dam and levee safety, and foundation bearing capacity. This paper proposes/updates a simplified procedure for evaluating cyclic softening (amount of strength loss) that may be expected in saturated clays during earthquakes and implements it in pseudo-static analysis. The procedure has two main steps: (1) estimation of an equivalent cyclic shear strain amplitude and associated number of cycles induced in the soil mass by an earthquake; and (2) estimation of softening and strength loss in the soil mass. The proposed procedure provides reasonable, first-order estimates of cyclic softening consistent with the other developed procedures. In addition, the results of applying the estimated strength softening in pseudo-static analysis are compared with case histories identified as involving cyclic softening of clays.

RÉSUMÉ : Le ramollissement cyclique et la perte de résistance de l'argile saturée lors des tremblements de terre est souvent un facteur important dans les problèmes d'ingénierie tels que la stabilité des pentes, les barrages et digues de sécurité, et la capacité portante des fondations. Cet article propose /met à jour une procédure simplifiée pour l'évaluation du ramollissement cyclique (montant de la perte de résistance) auquel on peut s'attendre dans l'argile saturée lors de tremblements de terre et le met en œuvre a travers l'analyse pseudo statique. La procédure comporte deux étapes principales: (1). l'estimation d'une amplitude de déformation cyclique de cisaillement équivalent et le nombre de cycles associé induits dans la masse du sol par un tremblement de terre; et (2) l'estimation de ramollissement et la perte de force dans la masse du sol. La procédure proposée prévoit de façon raisonnable, des estimations cycliques de ramollissement de premier ordre, compatible avec d'autres procédures. Par ailleurs, les résultats de l'application de la force de ramollissement estimée dans l'analyse pseudo statique se comparent bien avec des cas identifiés dans le passé et impliquant le ramollissement cyclique des argiles.

KEYWORDS: Cyclic loading, strength softening, clay, strain approach

1 INTRODUCTION

Cyclic softening of clays is commonly understood as the reduction in soil stiffness and strength due to repeated cyclic loading, as shown in Figure 1. Experiments by Idriss et al. (1978) showed that the initial stiffness and the ordinates of the backbone curve of soft clay are reduced after several cycles. Matasovic and Vucetic (1995) discussed the coupling of cyclic pore-water pressure generation and softening of clay and the use of a threshold strain concept.

Recent case histories have revealed evidence of cyclic softening of clays during earthquakes. The 1999 Chi-Chi, Taiwan Earthquake caused extensive ground failure and structural damage in Wufeng, Taiwan. Some of the most interesting cases of damage involved ground failure in areas underlain by low plasticity clayey soils (Chu et al. 2008). During the 1999 Kocaeli earthquake, in situ deformation measurements at the Carrefour Shopping Center showed significant vertical strains in ML/CL and CH strata. Martin et al. (2004) concluded that the ML/CL layer had exhibited "liquefaction type behavior" while "a definitive explanation for significant earthquake-induced settlements in a high-plasticity clay stratum (CH) in Lot C has not yet been found."

Recently, Boulanger and Idriss (2007) developed a procedure for evaluating the potential for cyclic softening in clay-like fine-grained soils during earthquakes. Their procedure uses a stress-based approach similar to that used in semi-empirical liquefaction procedures. Mejia et al. (2009) applied a procedure similar to the Boulanger and Idriss procedure to estimate post-cyclic strengths of clay-like fine-grained soils and applied it to dam safety evaluation. Tsai and Mejia (2011) utilized cyclic strain to evaluate cyclic softening of clayey soils, which is different from the previous stress-based approach. This paper updates the Tsai and Mejia procedure by adopting the most recent study, especially for estimating equivalent number of cycles in the strain-based approach. The revised procedure is first compared with another more complex analysis procedure by analyzing the same design case. Then it is applied to case histories involving strength loss, and is found to provide reasonable estimates of behavior consistent with the field observations.

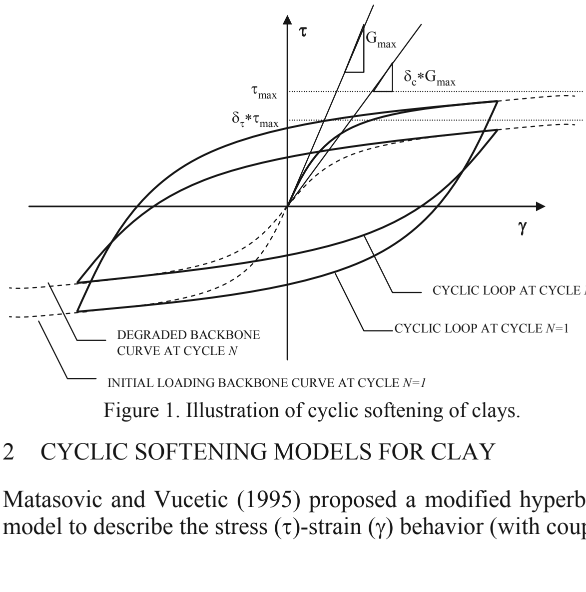


Figure 1. Illustration of cyclic softening of clays.

2 CYCLIC SOFTENING MODELS FOR CLAY

Matasovic and Vucetic (1995) proposed a modified hyperbolic model to describe the stress (τ)-strain (γ) behavior (with coupled pore water generation), modulus degradation, and strength softening of clays based on the following equation

$$\tau = \frac{\delta_c G_{max} \gamma}{1+\beta\left(\frac{\delta_c G_{max}}{\delta_\tau \tau_{max}}\right)^s} \quad (1)$$

Where G_{max} is small strain shear modulus, δ_c is the modulus degradation index function and δ_τ is the stress softening index function. Both indexes are coupled with the excess pore water pressure and are dependent on the soil type. β and s are two curve fitting parameters. For fine-grained clayey soil, the modulus degradation and stress softening indices are equal and can be correlated to the number of cycles as follows:

$$\delta_c = \delta_\tau = N_c^{-s(\gamma-\gamma_t)r} \quad (2)$$

s and r are curve fitting parameters, which are correlated to clay properties such as plasticity Index (PI) and overconsolidation ratio (OCR), as listed in Table 1 based on comprehensive laboratory test data (Vucetic and Dobry 1988). The threshold shear strain amplitude γ_t separates the domains of cyclic softening development and essentially no softening development. The corresponding cyclic softening model of clays is depicted in Figure 1. After N_c cycles, the clay exhibits softening behavior including both strength softening ($\delta_\tau \tau_{max}$) and modulus degradation ($\delta_c G_{max}$). Figure 2 presents an example of strength softening after several numbers of cycles with different cyclic strain amplitudes for a clayey soil with an OCR of 1. To quantitatively estimate cyclic softening using the stress softening index in Eq. (2), the equivalent amplitude of cyclic strain and the corresponding number of cycles need to be determined given an earthquake event.

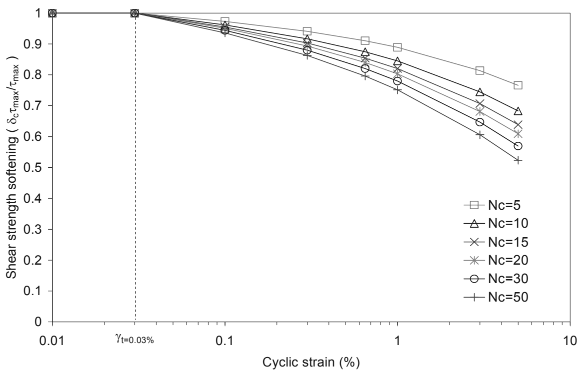


Figure 2 Strength softening versus cyclic strain and number of cycles

Table 1. Model parameters of cyclic softening (Vucetic and Dobry 1988)

OCR	s	r
1	0.075	0.495
1.4	0.064	0.520
2	0.054	0.480
4	0.042	0.423

3 EFFECTIVE CYCLIC STRAIN

An equivalent amplitude of cyclic shear strain is used by Tokimatsu and Seed (1987) to estimate the seismically-induced volumetric change of unsaturated cohesionless soils. The effective shear strain γ_{eff} is estimated from τ_{eff} using the effective shear modulus (G_{eff}) as follows:

$$\gamma_{eff} \frac{G_{eff}}{G_{max}} = 0.65 \cdot \frac{PGA}{g \cdot G_{max}} \sigma_0 \cdot r_d \quad (3)$$

where PGA is the ground surface peak ground acceleration, g = the acceleration of gravity and σ_o = the total overburden pressure at depth h , and r_d is a stress reduction factor. The product γ_{eff} (G_{eff}/G_{max}) in Eq. (3) can be readily translated to a shear strain amplitude γ_{eff} using published models for soil modulus reduction with increasing shear strain (i.e. models relating γ_{eff} to G_{eff}/G_{max}).

The modulus reduction curve may be selected from published models based on soil index properties. Darendeli and Stokoe (2001) proposed a family of modulus reduction curves based on a large suite of test results considering the effects of effective stress (σ'), soil plasticity (as represented by PI), and OCR. Figure 3 shows a family of modulus reduction curves (based on the Darendeli and Stokoe model) for varying PI and OCR and for σ_v'=1 atm. Note that the plots in Figure 3 are formatted to directly estimate shear strain γ from the product $\gamma(G/G_{max})$. Once the product $\gamma(G/G_{max})$ is calculated using Eq (3), effective cyclic strain can be directly determined from Figure 3.

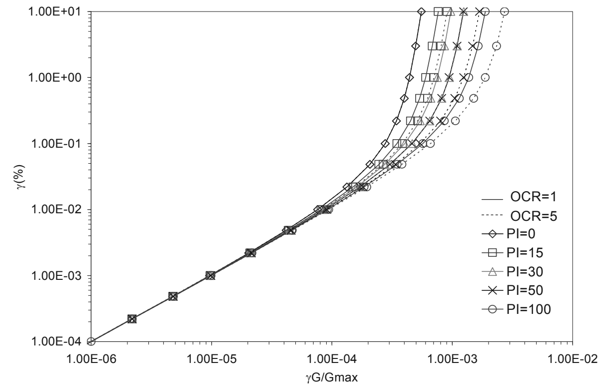


Figure 3 Modulus reduction curves from Darendeli and Stokoe (2001) re-expressed in the format for estimation of shear strain amplitude

4 EQUIVALENT NUMBER OF UNIFORM STRAIN CYCLES (N_C)

Using a strong motion data set from tectonically active regions, Liu et al. (2001) developed empirical regression equations to evaluate the equivalent number of uniform stress cycles of earthquake shaking as a function of magnitude (M_w), site-source distance, site condition, and near-fault rupture directivity effects. The N_c values were derived based on weighting factors specific to the evaluation of soil liquefaction triggering and a linear relationship between cyclic resistance ratio (CRR) and N_c in log-log space is established from laboratory tests;

$$CRR = a \cdot N_c^{-b} \quad (4)$$

where a and b depend on soil type and b stands for the relative contribution of cycles with different amplitudes to the failure. Typically, b ranges from 0.1 to 0.5 representing clay to sand with respect to liquefaction triggering. With regard to cyclic softening of clay, b is approximately one given different strength softening conditions, as shown in Figure 4, where the vertical axis represents cyclic strain instead of CRR. Therefore, Liu et al.'s empirical regression equations are not applicable for the case of cyclic softening because the weighting factors are different due to different b values and, consequently, estimated N_c is diverse.

Kishida and Tsai (2012) proposed an empirical equation that can consider different b values for estimating N_c as follows:

$$\ln N_c = \ln\left[\frac{\exp(c_0 + c_1 \ln PGA + c_2 \ln S_1 + c_3 M_w + c_4 \ln b + c_5 b T_s) + 0.5}{0.65^{\frac{1}{b}}}\right] \quad (5)$$

where, c_0 = -3.43, c_1 = -0.352, c_2 = -0.402, c_3 = 0.798, c_4 = 1.72, and c_5 = -1.50. The prediction equation can also estimate N_c at a selected depth (z) by introducing the parameter

$$T_s = \frac{4z}{V_s} \tag{6}$$

where *V*s is average shear wave velocity within depth z. In addition, the site effect is accounted for by

$$S_1 = \frac{Sa(1.0)}{Sa(0.2)} \tag{7}$$

S_1 is the spectral ratio between 1.0 sec spectrum acceleration Sa(1.0) and 0.2 sec spectrum acceleration Sa(0.2). Kishida and Tsai have shown that their prediction equation can estimate similar *N*c values as those of Lui et al. for sand (b=0.35) at the ground surface.

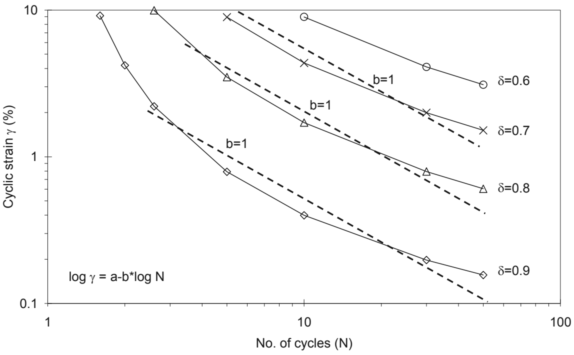


Figure 4 b for cyclic softening of clay

5 ANALYSIS PROCEDURE

The procedure to consider strength softening in pseudo-static analysis has four general steps: (1) estimation of the shear strain amplitude and the equivalent number of uniform strain cycles from the peak acceleration at the ground surface and other seismological and site parameters; (2) estimation of strength softening δτ of the soil based on the effective shear strain amplitude, and the equivalent number of uniform strain cycles; (3) estimation of the cyclic soil strength after cyclic softening by multiplying the strength for 1 cycle by δτ .; (4) implementation of reduced strength (representing post-earthquake condition) in pseudo-static analysis. Details on the analysis steps were described in Tsai and Mejia (2011). For preliminary studies, the strength for 1 cycle of most clays may be conservatively assumed equal to their static strength.

6 COMPARISON TO CASE HISTORIES

In this section, the above analysis procedure to estimate strength softening was implemented in pseudo static analysis and compared with the predicted consequences of such softening with observed ground failure during past earthquakes.

1.1 *Berryman Reservoir, California, CA, USA*

Berryman Reservoir, owned and operated by the East Bay Municipal Utility District, USA, is located in the City of Berkeley in Alameda County, California, and is within the State Alquist-Priolo Earthquake Fault Zone of the Hayward fault. Previous seismic hazard investigations concluded that active traces of the Hayward fault bisect the reservoir. Previous and recent field and laboratory investigations indicated that generally stiff cohesive soils (medium to high PI) grading to highly weathered bedrock are present at the site.

To evaluate the seismic performance of the embankment, URS (2008) developed a design response spectrum and site-specific earthquake ground motions for input to the dynamic slope stability analyses. Following the Mejia et al. (2009) procedure, two-dimensional dynamic response analyses were performed using QUAD4M to estimate the cyclic stresses and accelerations induced by the design earthquake within the reservoir embankment. This comprehensive analysis indicated that the undrained strengths of the saturated clayey soils could be reduced by as much as 40 percent under the post-earthquake condition. Given the same design scenario as listed in Table 2, strength softening was also calculated using the simplified procedure proposed in this paper. It was found that the strengths of the saturated soils could be reduced by approximately 25-30 percent. Although the predicted strength reduction is less than that by Mejia et al.'s procedure, it is still a reasonable, first-order estimate of cyclic softening of stiff clay. The yield accelerations, obtained from pseudo static analyses using UTEXAS4, were 0.16g for the pre-earthquake (no strength reduction) and 0.12g and 0.1 for post-earthquake condition with 30% and 40% strength softening, respectively. The critical failure plane is shown in Figure 5.

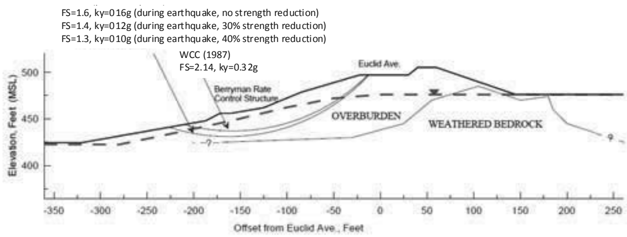


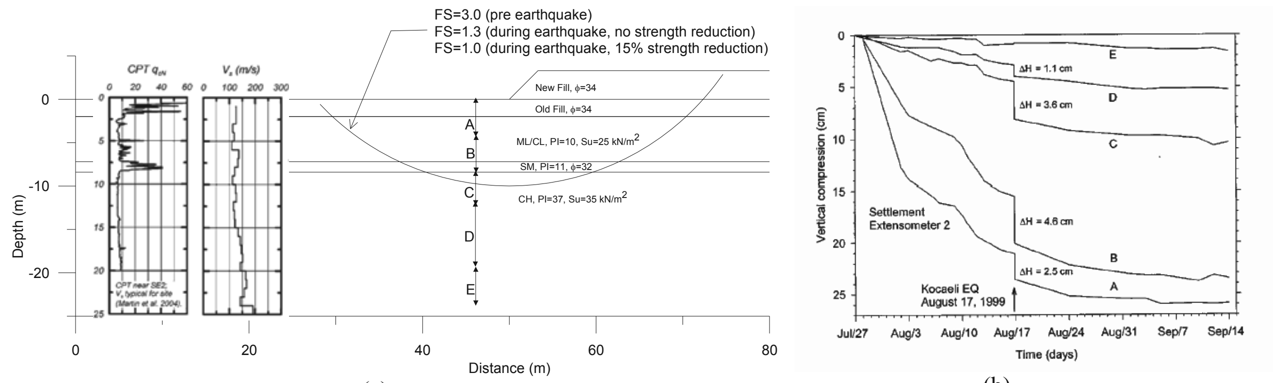
Figure 5 Pseudo-static analysis considering strength softening at Berryman Reservoir

1.2 *Carrefour Shopping Center 1999 Kocaeli*

The Carrefour Shopping Center Lot C case history (Martin et al. 2004) provided a unique set of in situ ground deformation measurements in ML/CL and CH strata from settlement extensometers during the 1999 Kocaeli earthquake. This case history provides an excellent example of how fine-grained soils can develop significant strains or fail due to seismic loading, and an opportunity to evaluate the procedures presented herein.

As shown in Figure 6a, the soil profile at Lot C includes a surface layer of approximately 2 m of medium dense fill (gravelly clay, GC). The next 5 m of soil consists of saturated, soft to firm, low plasticity silt and clay (ML/CL) having average PI and LL values of 10 and 33, respectively. This layer is underlain by about 1.2 m of loose to medium-dense silty sand, and sand (SP/SM) having a typical equivalent clean sand corrected SPT blow count ($(N_1)_{60,cs}$) value of about 12. The sand layer is underlain by about 0.9 m of ML/CL soils, followed by medium to stiff, high plasticity clay (CH) that extends to depths greater than 35 m and has an average PI value of 37.

The vertical strains induced in the fine-grained soil layers by the earthquake are largely attributed to undrained shear failure beneath the surcharge, as illustrated in Figure 6a. The settlement records in Figure 6b do show a modest increase in the rate of settlement from just before the earthquake to just after the earthquake. It is reasoned that the earthquake likely induced moderate excess pore pressures and that the increase in settlements was largely due to undrained shear failure induced by bearing-capacity mode of deformation.



(a) (b)

Figure 6 (a) Subsurface condition at Carrefour Shopping Center (b) Extensometer measurements at Carrefour Shopping Center (Martin et al., 2004)

Table 2. Summary of input parameters and estimated strength loss of two case histories

Case	Seismological information			Site/ soil condition			Estimated results			
	Mw	R	PGA	Vs	OCR	PI	Nc	γeff	δτ	Strength loss
	-	(km)	(g)	m/sec	-	-	(-)	(%)	-	(%)
Berryman	7.0	0.2	0.72	240	3-5	15-30	29	1-3	0.75	25-30
Carrefour	7.4	5	0.24	120	1	10-37	28	0.3	0.88	10-15

Bearing-capacity was evaluated using a slope stability approach before the earthquake and during the earthquake with a pseudo-static type analyses. The failure surface was constrained to a depth of 10 m below the ground surface because extensometer measurements indicate that large deformation occurred at this depth. As shown in Figure 6, the FS for slope stability is 3.0 pre-earthquake and becomes 1.0 with a 15% estimated strength reduction (as listed in Table 2), compared to 1.3 without considering strength loss during the earthquake. The result suggests that the soil layer exhibited strength loss and that the amount of strength loss simply estimated by the proposed procedure leads to an analysis outcome consistent with the field observations.

7 CONCLUSION

In this paper we present an analysis procedure to estimate cyclic softening of saturated clays under seismic loading. Unlike common liquefaction potential analysis procedures that use a stress-based approach, the procedure uses a strain-based approach to estimate cyclic softening and associated strength loss. The procedure has two main components: (1) estimation of the shear strain amplitude and the equivalent number of uniform strain cycles within the soil mass induced by an earthquake event; and (2) estimation of the softening and associated strength loss within the soil given the effective shear strain amplitude and the equivalent number of uniform strain cycles. The procedure is successfully implemented in pseudo-static analysis for analyzing one design case and one field case history and is found to generally provide reasonable, first-order estimates of cyclic softening consistent with the field observations.

8 ACKNOWLEDGEMENTS

This study was mainly performed by the first author during his employment with URS Corporation. The authors acknowledge URS' support. The authors also would like to acknowledge Mr. Fidèle Nikiema in translating the abstract into French.

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