Use of activation and accumulated energies to understand the cyclic and post-cyclic behavior of fine-grained soils

Utilisation des énergies d'activation et d'accumulation pour comprendre le comportement cyclique et post-cyclique des sols à grains fins

Beena Ajmera, Binod Tiwari, Cara Harrington
Department of Civil and Environmental Engineering, California State University, Fullerton, USA, bajmera@fullerton.edu

ABSTRACT: Recent earthquakes have demonstrated the disastrous consequences of failures triggered by strength loss in the underlying fine-grained soils. In this study, 17 soil samples prepared in the laboratory from kaolinite, montmorillonite and quartz and 2 natural samples were subjected to stress-controlled constant volume cyclic loading in the cyclic simple shear apparatus. Immediately following the cyclic loading, strain-controlled constant volume tests were used to determine the undrained shear strengths. In soils with kaolinite as the clay mineral, an increase in the plasticity index or an increase in the level of double amplitude shear strain experienced resulted in an increase in the accumulated energy. On the other hand, in soils with montmorillonite as the clay mineral, an increase in the double amplitude shear strain or a decrease in the plasticity index resulted in an increase in the accumulated energy. The degradation ratio was found to increase with an increase in the plasticity index for soils with plasticity indices less than 100. Soils with plasticity indices greater than 100 did not experience significant degradation in undrained shear strengths. The reduction in shear strength could be explained in terms of difference in activation and accumulated energies.

RÉSUMÉ : Les séismes récents ont démontré les conséquences désastreuses des défaillances provoquées par la perte de force dans les sols à grain fin sous-jacents. Dans cette étude, 17 échantillons de sol préparés au laboratoire à partir de la kaolinite, de la montmorillonite et du quartz et de 2 échantillons naturels ont été soumis à une charge cyclique à volume constant contrôlé par contrainte dans l'appareil de cisaillement cyclique simple. Immédiatement après le chargement cyclique, des essais de volume constant contrôlés par contrainte ont été utilisés pour déterminer les forces de cisaillement non drainées. Dans les sols à kaolinite comme minéral argileux, une augmentation de l'indice de plasticité ou une augmentation du niveau de souche de cisaillement à double amplitude expérimentée ont entraîné une augmentation de l'énergie accumulée. D'autre part, dans les sols à montmorillonite comme minéral argileux, une augmentation de la contrainte de cisaillement à double amplitude ou une diminution de l'indice de plasticité a entraîné une augmentation de l'énergie accumulée. On a constaté que le taux de dégradation augmentait avec une augmentation de l'indice de plasticité pour les sols dont l'indice de plasticité était inférieur à 100. Les sols présentant des indices de plasticité supérieurs à 100 n'avaient pas de dégradation significative des résistances au cisaillement non drainé. La réduction de la résistance au cisaillement pourrait être expliquée en termes de différence d'activation et d'énergie accumulée.

KEYWORDS: Cyclic behavior; post-cyclic shear strength; strength degradation; accumulated energy; activation energy.

1 INTRODUCTION

In seismic regions, the stability of infrastructure depends on the cyclic and post-cyclic behavior of soft clays particularly since the cyclic loading may result in lowered undrained bearing capacity of foundations (Andersen 1988, Chu et al. 2008), slope instability (Stark and Contreras 1998, Boulanger and Idriss 2004, Tiwari et al. 2016), or lateral spreading and settlement (Boulanger et al. 1998). Several parameters including the overburden pressure, stress history, type of soil, plasticity characteristics and mineralogical composition of the soils will influence the behavior observed during cyclic loading and the strength of the material immediately after the cyclic loading. However, despite the substantial amount of research, there is lack of consensus on the expected behavior under different conditions (Cao and Law 1992).

Bray and Sancio (2006) and Bray et al. (2004) developed cyclic strength curves, plots of the number of cycles versus the cyclic stress ratio (CSR), from the cyclic simple shear and cyclic triaxial tests on undisturbed samples obtained from Turkey. They found that the cyclic resistance, a measure of the ability of the soil to withstand cyclic loading, was greater in soils with plasticity indices between 12 and 18 than in soils with plasticity indices less than 12. On the other hand, Guo and Prakash (1999) and El Hosri et al. (1984) found that an increase in the plasticity index will result in a reduction in the cyclic resistance for soils with plasticity indices less than 5. However, the cyclic resistance would increase with an increase in the plasticity index for soils with plasticity indices greater than 5. Similarly, Sandoval (1989) and Prakash and Sandoval (1992) also suggested that the cyclic resistance would decrease as the plasticity index was increased from 2 to 4.

Perhaps of more importance than the cyclic behavior of the soils, in examining the stability of infrastructure after cyclic loading, is the strength of the material after cyclic loading, or the post-cyclic shear strength. Most studies conclude that a greater reduction in the undrained shear strength may be expected in soils with lower plasticity indices (Ishihara and Yasuda 1980, Ishihara 1993, Guo and Prakash 1999, Bray et al. 2004, Bray and Sancio 2006), but the suggested reductions in this strength vary substantially in the literature.

The inconsistency in the observed cyclic and post-cyclic behavior despite the large amount of available literature may be a result of the simulations of cyclic loading in the laboratory using either strain-controlled or stress-controlled experiments. However, as neither the strain nor the stress can represent all the factors influencing the cyclic and post-cyclic behavior, differences in the testing procedures will inherently result in variations in the observed responses. This will make it difficult to arrive at consistent conclusions using traditional analyses. However, Nemat Nasser and Shookoh (1979) and Voznesensky and Nordal (1999) suggested that the examination of the cyclic and post-cyclic response in terms of the accumulated and activation energies may serve to alleviate the dependence of the observed behavior on the testing procedures.

The accumulated energy is a fraction of the total applied energy. It is equal to the total amount of energy that is dissipated into and accumulated by the soil. Cao and Law (1992), Dief and Figueroa (2007), Baziar and Jafarian (2007) found that this energy can be calculated in each cycle as the area of the stress-strain hysteresis loop. This energy will be used to cause a rearrangement of the particles and alter the bonds between the particles causing plastic strains (Cao and Law 1992, Voznesensky and Nordal 1999, Okur et al. 2008).

Glasstone et al. (1941) defined the activation energy as “the minimum energy the system must acquire before it can undergo the appropriate change.” For soils subjected to cyclic loading, this is the minimum energy that the soil mass must acquire before inter-particle bonds are altered resulting in deformation. It was suggested by Voznesensky and Nordal (1999) that the activation energy is equal to the maximum area of a closed stress-strain hysteresis loop. They further noted that the structural bond between the clay particles is the sole parameter controlling the activation energy.

The focus of literature to date has been to determine meaningful relationships between certain factors influencing cyclic behavior and the accumulated energy. Furthermore, these relationships and the findings presented have been (1) generalized based on the results for only one or two different soils and (2) only examine the observed cyclic behavior and not the post-cyclic shear strengths. Therefore, in this study, 17 laboratory prepared samples were used to conduct static and cyclic simple shear tests in order to evaluate the cyclic and post-cyclic behavior of fine-grained materials in the context of the accumulated and activation energies. The results were also used to evaluate the degradation in strength as a result of cyclic loading.

2 MATERIALS AND METHODS

Seventeen samples were prepared in the laboratory using dry and powdered form of montmorillonite, kaolinite, and silt-sized quartz. Using ASTM D4318-10, the Atterberg limits were determined for each of the samples. The resulting plasticity characteristics are plotted in the plasticity chart in Figure 1. In Figure 1, the numbers next to each point represent the sample number, as presented in Table 1. Table 1 also contains the proportion of montmorillonite, kaolinite and quartz in each laboratory prepared sample. N1 and N2 are the natural samples. N1 was obtained from a housing development project in Mission Viejo, CA, while N2 was collected from the montorillonite layer of the Portguese Bend Landslide in Rancho Palos Verde, CA.

The samples were tested using Norwegian Geotechnical Institute (NGI)-type static and cyclic simple shear devices. To prepare the samples, the dry soils were batch mixed with sufficient de-aired distilled water to have an initial moisture content equal to the liquid limit. The samples were allowed to hydrate for a period of 24 hours prior to testing. During this time, careful attention was paid to ensure that no moisture was lost. A portion of the hydrated sample was carefully placed into the rubber membrane and confined by a stack of Telfon® rings in either the static or cyclic simple shear device such that the initial density of the specimens was the same in all tests for any sample. Stress increments of 25, 50, and 100 kPa were used to consolidate the specimens to a vertical pressure of 100 kPa. Real-time computer generated logarithm of time versus vertical deformation curves were used to monitor the consolidation process and to determine the end of the primary consolidation phase. At this point, specimens in the static simple shear were subjected to undrained shearing at a rate of 5% per hour, based on the suggestions in ASTM D6528-07. On the other hand, specimens in the cyclic simple shear apparatus were subjected to strain-controlled constant volume cyclic loading via the application of a sinusoidal loading function with a frequency of 0.5 Hz (based on the recommendations in Silver et al. 1976, Silver 1976, and Boulanger et al. 1993) upon the completion of the primary consolidation. The desired cyclic stress ratio, or the ratio of the amplitude of the cyclic stress to the consolidation pressure, was used to determine the amplitude of the cyclic loads. The cyclic loading phase was continued until the specimen experienced 10% double amplitude shear strain or a maximum of 500 cycles (based on the suggestions in ASTM D5311-04). At the end of the cyclic loading, the specimens were subjected to monotonic undrained strain-controlled loading similar to that applied in the static simple shear tests. In both the static and cyclic simple shear tests, the monotonic shearing phase was terminated when the peak undrained shear strength was obtained or at 25% shear strain. The specimen was then allowed to swell for the smaller of the time required for 90% of the primary consolidation or one hour (ASTM D6528-07).

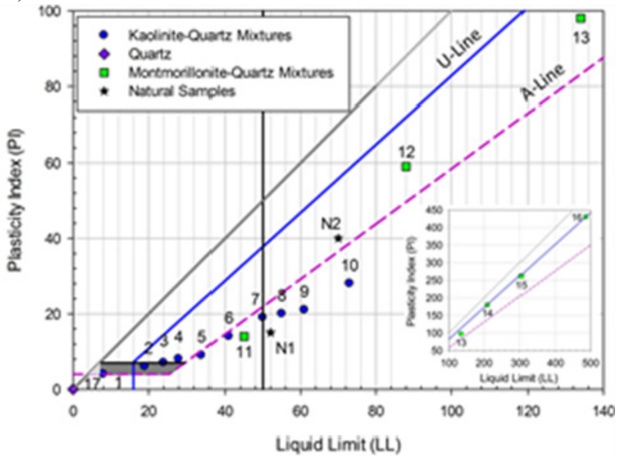

Figure 1. Plasticity characteristics of samples tested in this study.

Table 1. Mineralogical compositions of samples.

SN.	M	K	Q	SN.	M	K	Q
1	0	10	90	10	0	100	0
2	0	20	80	11	10	0	90
3	0	30	70	12	20	0	80
4	0	40	60	13	30	0	70
5	0	50	50	14	50	0	50
6	0	60	40	15	70	0	30
7	0	70	30	16	100	0	0
8	0	80	20	17	0	0	100
9	0	90	10				

3 RESULTS AND DISCUSSION

The results related to the logarithm of time versus vertical deformation, void ratio versus logarithm of pressure, cyclic loading function and the resulting pore pressure, effective vertical stress, shear strain, hystersis loops as well as the shear strain versus shear stress and pore pressure for the post-cyclic shearing phase could not be presented here due to space limitations, but are available in Ajmera (2015).

3 .1 *Cyclic Behavior*

Cyclic strength curves were developed based on the number of cycles required to cause 2.5%, 5%, and 10% double amplitude shear strains to represent the seismic performance levels of immediate occupancy, life safety, and collapse prevention, respectively. These curves could not be presented here due to space limitations, but can be found in Ajmera (2015). It was seen that in soils with kaolinite as the clay mineral, an increase in the plasticity index resulted in an upward shift of the increasing linear cyclic strength curves. On the other hand, there was little change in the vertical position of the cyclic strength curves for the soils with montmorillonite as the clay mineral. However, when the plasticity index was less than 45, the cyclic strength curves exhibited a larger degree of curvature, while the curvature remained nearly constant when the plasticity index was greater than 45 (Ajmera et al. 2015).

The variation of the accumulated energy with the plasticity index at different levels of double amplitude shear strain is shown in Figure 2. Similarly, Figure 3 contains the relationship between the activation energy and the plasticity index. The activation energy was computed from each of the cyclic tests as the maximum area of the closed hysteresis loop. The results were similar despite the variations in the amplitude of the cyclic loads. In soils with kaolinite as the clay mineral, an increase in the amount of accumulated energy is noted with an increase in the double amplitude shear strain. This is attributed to the fact that additional energy will be accumulated by the soil as additional cyclic loads are applied in order for the soil to deform to higher levels of double amplitude shear strain. The increase in the accumulated energy with an increase in the plasticity index is also a result of the fact that the activation energy of soils with kaolinite as the clay mineral appears to increase with an increase in the plasticity index as shown in Figure 3. As such, additional amounts of energy must be accumulated by the soil mass before permanent deformations can occur. Moreover, soils with higher plasticity indices will exhibit increased resistance to movement, and thus, be able to accumulate more energy. On the other hand, the magnitude of the accumulated energy in soils with montmorillonite as the clay mineral was substantially larger than that in soils with kaolinite as the clay mineral, but appears to decrease with an increase in the plasticity index. This is attributed to the fact that the activation energy (Figure 3) in soils with montmorillonite as the clay mineral is larger than the activation energy in soils with kaolinite as the clay mineral. Hence, larger amounts of energy must be accumulated by the soil mass in order to overcome the activation energy before plastic deformations can occur. Since activation energy of soils with montmorillonite as the clay mineral decreases with an increase in the plasticity index (Figure 3), the amount of energy accumulated will also decrease with an increase in the plasticity index, as seen in Figure 2.

3 .2 *Post-Cyclic Behavior*

The static undrained strength ratios (USRs) of the samples not subjected to cyclic loading ranged from 0.23 to 0.36. For the samples subjected to cyclic loading, the post-cyclic undrained shear strength was normalized by the consolidation pressure to obtain the post-cyclic undrained strength ratio (PC-USR). The PC-USRs were found to range between 0.20 and 0.34 for the soils tested in this study. Details of the USR and PC-USR values for each sample tested may be found in Ajmera et al. (2015), who showed that higher PC-USR values were observed in soils with montmorillonite as the clay mineral in comparison to the soils with kaolinite as the clay mineral.

Due to the prevalence of static strength measurements in comparison to the post-cyclic strength testing, the degradation ratio, defined as the ratio of the post-cyclic undrained shear strength to the static undrained shear strength, was computed. The variation in the degradation ratio with plasticity index is presented in Figure 4. The degradation ratio found to range between 0.5 and 1, suggesting that the post-cyclic undrained shear strength will be between 50% and 100% of the static undrained shear strength. Figure 4 shows that the degradation ratio increases with increases in the plasticity index for soils with plasticity indices less than 100 and remains nearly constant at unity (no strength degradation) for soils with plasticity indices greater than 100. Shown in Figure 5 is the relationship between the degradation ratio and the difference in the accumulated and activation energies of the soil mass. Since soils that are capable of accumulating more energy within the inter-particle bonds of the soil particles will experience less permanent deformation, lower reductions in the undrained shear strength are expected. Thus, as the difference between the accumulated and activation energies increased, the reduction in the undrained shear strength due to cyclic loading appeared to decrease.

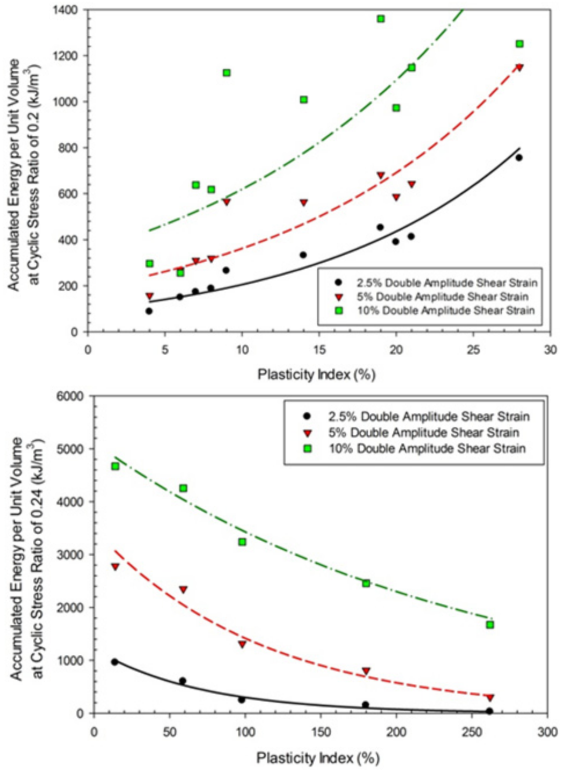

Figure 2. Relationship between accumulated energy and plasticity index for (top) soils with kaolinite at CSR of 0.2 and (bottom) soils with montmorillonite at CSR of 0.24.

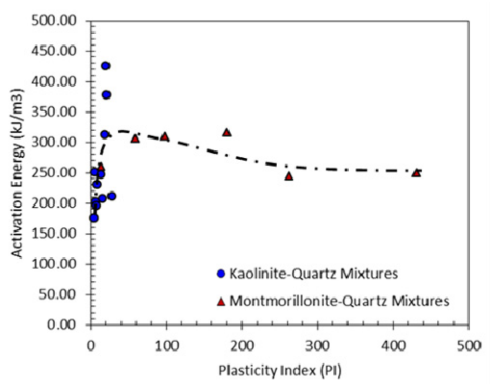

Figure 3. Relationship between activation energy and plasticity index.

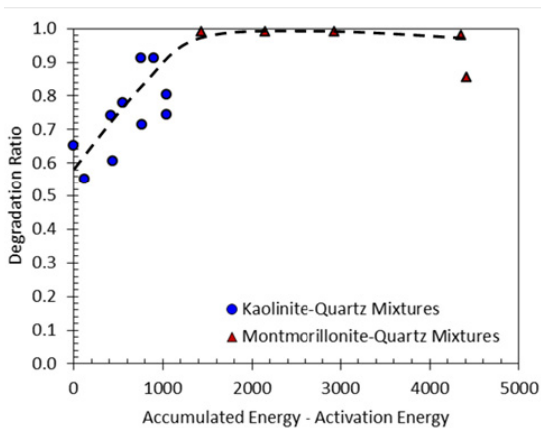

Figure 4. Variation in degradation ratio with plasticity index.

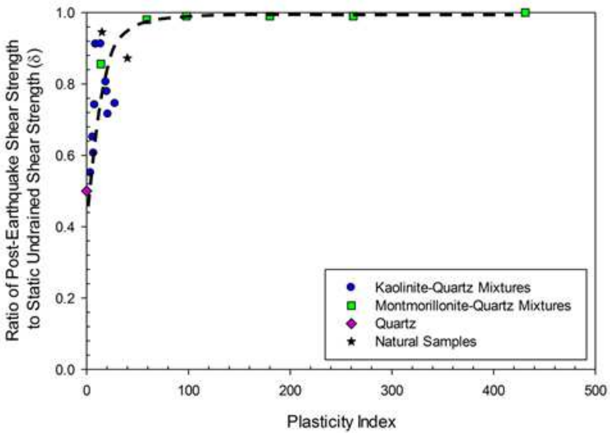

Figure 5. Variation in degradation ratio with difference in accumulated and activation energies.

4 CONCLUSIONS

To evaluate the cyclic and post-cyclic behavior of fine-grained soils in terms of the accumulated and activation energies, static and cyclic simple shear tests were conducted on seventeen laboratory prepared and two natural soil samples. In soils with kaolinite as the clay mineral, an increase in the amount of accumulated energy is noted with an increase in the double amplitude shear strain and an increase in the plasticity index. On the other hand, the accumulated energy in soils with montmorillonite as the clay mineral was substantially larger than that in soils with kaolinite as the clay mineral, but appears to decrease with an increase in the plasticity index. This behavior could be attributed to the increase in the activation energy with plasticity index in soils with kaolinite as the clay mineral and a decrease in the activation energy with an increase in the plasticity index in soils with montmorillonite as the clay mineral.

The degradation ratio is found to increase with an increase in the plasticity index for soils with plasticity indices less than 100 and remains nearly constant at a value of one for soils with plasticity indices greater than 100. This behavior is a result of the amount of energy the soil accumulates prior to exhibiting permanent deformations. Specifically, the greater the difference between the accumulated and activation energies, the more energy is accumulated between the inter-particle bonds of the soil particle. Therefore, less deformation is expected and thus, results in lower reductions in the undrained shear strength with cyclic loading.

5 ACKNOWLEDGEMENTS

The authors would like to appreciate the generous support provided by the National Science Foundation Graduate Research Fellowship (Fellowship ID 2011105853), the Charles E. Via Doctoral Fellowship, and the United States Society of Dams.

6 REFERENCES

Ajmera B. 2015. Factors affecting the post-earthuake shear strength. *Ph.D. Dissertation, Virginia Polytechnic Institute and State University*.

Ajmera B., Brandon T., and Tiwari B. 2015. Cyclic strength of clay-like materials. *Proceedings of the 6th International Conference on Earthquake Geotechnical Engineering*, New Zealand.

Andersen K.H. 1988. Properties of soft clay under static and cyclic loading. *Proceedings of International Conference on Engineering Problems of Regional Soils*.

ASTM D 4318.2010. Standard test methods for liquid limit, plastic limit and plasticity index of soils. *ASTM International*.

ASTM D 5311. 2004. Standard test method for load controlled cyclic triaxial strength of soil. *ASTM International*.

ASTM D 6528. 2007. Standard test method for consolidated undrained direct simple shear testing of cohesive soils. *ASTM International*.

Baziar M.H. and Jafarian Y. 2007. Assessment of liquefaction triggering using strain energy concept and ANN model: capacity energy. *Soil Dynamics and Earthquake Engineering* 27, 1056-1072.

Boulanger R.W. and Idriss I.M. 2004. Evaluating the potential for liquefaction or cyclic failure of silts and clays. *Report No. UCD/CGM-04/01 at University of California, Davis*.

Boulanger R.W., Chan C.K., Seed H.B., Seed R.B., and Sousa J.B. 1993. A low compliance bi-directional cyclic simple shear appartus. *ASTM Geotechnical Testing Journal* 16(1), 36-45.

Boulanger R.W., Meyers M.W., Mejia L.H., and Idriss I.M. 1998. Behavior of fine-grained soil during the Loma Prieta earthquake. *Canadian Geotechnical Journal* 35, 146-158.

Bray J.D. and Sancio R.B. 2006. Assessment of liquefaction susceptibility of fine-grained soils. *Journal of Geotechnical and Geoenvironmental Engineering*, 132(9), 1165-1177.

Bray J.D., Sancio R.B., Riemer M., and Turan Durgunoghr H. 2004. Liquefaction suceptibility of fine-grained soils. *Proceedings of the Engineering and 3rd International Conference on Earthquake Geotechnical Engineering* 1, 655-662.

Cao Y.L. and Law K.T. 1992. Energy dissipation and dynamic behavior of clay under cyclic loading. *Canadian Geotechnical Journal* 29, 103-111.

Chu D.B., Stewart J.P., Boulanger R.W., and Lin P.S. 2008. Cyclic softening of low plasticity clay and its effect on seismic foundation performance. *Journal of Geotechnical and Geoenvironmental Engineering* 134(11), 1595-1608.

Dief H.M. and Figueroa J.L. 2007. Liquefaction assessment by the unit energy concept through centrifuge and torsional shear tests. *Canadian Geotechnical Journal* 44(11), 1286-1297.

El Hosri M.S., Biarez H., and Hicher P.Y. 1984. Liquefaction characteristics of silty clay. *Proceedings of 8th World Conference on Earthquake Engineeirng* 3, 277-284.

Glasstone S., Laidler K., and Eyring, H. 1941. The theory of rate process. *McGraw-Hill*, New York.

Guo T. and Prakash S. 1999. Liquefaction of silts and silt-clay mixtures. *Journal of Geotechnical and Geoenvrionmental Engineeirng* 125(8), 706-710.

Ishihara K. 1993. Liquefaction and flow failure during earthquakes. *Géotechnique* 43(3), 351-415.

Ishihara K.and Yasuda S. 1980. Cyclic strengths of undisturbed cohesive soils of western Tokyo. *International Symposium on Soils under Cyclic and Transient Loading*, 57-66.

Nemat-Nasser S. and Shokooh A. 1979. A unified approach to densification and liquefaction of cohesionless sand in cyclic shearing. *Canadian Geotechnical Journal* 16(4), 659-678.

Okur V. Altun S., and Ansal A. 2008. Cyclic failure of fine-grained soils during the 1989 Kocaeli earthquake. *Proceedings of Geotechnical Earthquake Engineering and Soil Dynamics IV* 181.

Prakash S. and Sandoval J. 1992. Liquefaction of low plasticity silts. *Journal of Soil Dynamics and Earthquake Engineering* 71(7), 373-397.

Sandoval J. 1989. Liquefaction and settlement characteristics of silt soils. *Ph.D. Dissertation, Unviersity of Missouri*.

Silver M.L. 1976. Laboratory triaxial testing procedures to determine the cyclic strength of soils. *NUREG-00031 U.S. Nuclear Regulatory Commission*.

Silver M.L., Chan C.K., Ladd R.S., Lee K.L., Tiedemann D.A., Townsend F.C., Valera J.E., and Wilson, J.H. 1976. Cyclic triaxial strength of standard test sand. *Journal of the Geotechnical Engineering Division* 102(5), 511-523.

Stark T.D. and Contreras I.A. 1998. Fourth Avenue Landslide during 1964 Alaskan earthquake. *Journal of Geotechnical and Geoenvironmental Engineering* 124(2), 99-109.

Tiwari B., Ajmera B., and Yamashiro B. 2016. Causes of cyclic shear failure at Lokanthali of Araniko Highway after Mw 7.8 2015 Gorkha earthquake. *Proceedings of the 5th International Conference on Forensic Geotechnical Engineering* (In Press).

Voznesensky E.A. and Nordal S. 1999. Dynamic instability of clays: an energy approach. *Soil Dynamics and Earthquake Engineering* 18.