Journal of Civil Engineering and Architecture 20 (2026) 163-170
doi: 10.17265/1934-7359/2026.04.004

Settlements of Mexico City Soil Induced by Cyclic Shear Loading

J. A. Díaz-Rodríguez

Department of Civil Engineering, Faculty of Engineering, National University of Mexico, Mexico City 04510, Mexico

Abstract: Intense ground shaking in Mexico City during three large-magnitude earthquakes caused excessive settlement and tilting of many building foundations. Even small differential settlements can significantly affect structural safety. To assess these effects, this paper presents results from cyclic simple shear tests on Mexico City soil samples. First, tests were performed under undrained conditions to evaluate behavior at different cyclic shear stress amplitudes. Second, tests were conducted under drained conditions to investigate the accumulation of vertical strain (cyclic subsidence) in soil specimens subjected to varying shear-stress amplitudes. The results confirm a threshold cyclic shear stress of 0.8, below which shear strain tends toward equilibrium plateaus as the number of cycles increases. The results also indicate that under drained conditions (a) cyclic shear stresses produce immediate vertical strains in Mexico City soil; (b) the magnitude of vertical strain depends on the cyclic shear stress level and the number of cycles; and (c) although the strain amplitudes are smaller than those due to consolidation, they might still cause significant damage to buildings and other structures.

Keywords: Seismic subsidence, Mexico City, simple shear tests, cyclic loading, vertical strain.

1. Introduction

During earthquakes, natural soil deposits experience cyclic shear stresses of varying amplitude and frequency, causing both temporary and permanent deformation. Buildings on these deposits may sustain significant damage from overall and differential settlement. Therefore, it is essential to evaluate the seismic behavior of any soil type — from sand to clay, whether saturated or unsaturated — by estimating the critical strains or strength reductions that could lead to ground deformation or instability during seismic events.

Pioneering research on seismic compression was conducted by Silver & Seed (1971) [1], Youd (1972) [2] and Seed & Silver (1972) [3] which focused on volumetric strains in dry, clean sands under cyclic loading without a mean shear stress. Additionally, Pyke, Seed, & Chan (1975) [4] investigated the seismic compression behavior of dry Monterey No. 0 sand using large-scale specimens tested on a shaking table.

Chu & Vucetic (1992) [5] examined the settlement behavior of compacted clay under shear loading. Specifically, the study sought to identify the threshold shear strain beyond which significant settlement occurs.

Stewart, Smith, Whang, & Bray (2002) [6] presented two case histories in which ground deformation was precisely measured using data collected before and after the 1994 Northridge earthquake. They described seismic compression as the buildup of contractive volumetric strains in unsaturated soil during strong earthquake shaking.

Tsukamoto, Ishihara, & Sawada (2004) [7] conducted tests on sands to evaluate the effect of saturation on the relative magnitudes of pre- and post-shaking volume changes.

Several researchers have continued to study seismic compression in various materials, including unsaturated sand [8], unsaturated compacted clays [9], and loess [10, 11].

In general, clay deposits have been considered more stable than sand under earthquake loading. In clay

---

Corresponding author: J. Abraham Díaz-Rodríguez, Ph.D., professor, research fields: geotechnical engineering, soil mechanics and soil dynamics.

deposits, researchers have focused primarily on post-earthquake settlements, when pore water pressure generated by seismic loading dissipates [12-16]. However, seismic subsidence can also occur in saturated fine-grained soils during a large earthquake (Fig. 1), resulting from the accumulation of vertical strain on a cycle-by-cycle basis.

Díaz-Rodríguez & Santamarina (2001) [17] documented the settlements in Mexico City during three large-magnitude earthquakes: 28 July 1957 (Mw = 7.6), 19 September 1985 (Mw = 8.0), and 19 September 2017 (Mw = 7.1).

Then, the aim of this study is twofold. First, cyclic simple shear tests were conducted under undrained conditions (i.e., constant-height testing) to assess behavior at various cyclic shear stress amplitudes. Second, cyclic simple shear tests were performed under equivalent-drained conditions (i.e., constant vertical stress testing) to determine the vertical strain accumulation characteristics (cyclic subsidence) of Mexico City soil specimens subjected to different shear stress amplitudes.

Fig. 1 Settlement of a building at Mexico City during 19 September 1985 Michoacán earthquake.

2. Materials and Testing Methods

Soil samples for this study were obtained from Ramón López Velarde Park (19.40 ° N, 99.15 ° W), located in the lake zone of Mexico City, at depths of 9 to13 m. The samples were recovered from two parallel holes, 2 m apart. Shelby tubes (OD = 128 mm, ID = 125 mm, area ratio 4.9%) were used to collect the samples.

Each tube was X-rayed, and no cracks or edge effects were observed.

The subsoil of Mexico City consists of volcanic-lacustrine soil deposits with the following average characteristics: natural water content, w_n = 375%, specific gravity, G_s= 2.44, liquid limit, w_l = 401%, and plastic limit, w_p = 130%. Grain-size distribution analysis indicates 50% silt content (diatoms). Diatoms are porous particles composed of the fossilized remains of microscopic algae that confer physicochemical and geotechnical properties on the clay matrix, properties that do not neatly fit within standard classification systems or empirical correlations. Samples retrieved from boreholes were classified as high-plasticity clay (CH) according to the Unified Soil Classification System. A more complete description of the subsoil of Mexico City is provided by Díaz-Rodríguez, Lozano-Santa Cruz, Dávila-Alcocer, Vallejo, & Girón (1998) [18] and Díaz-Rodríguez & Santamarina (2001) [17].

The Norwegian Geotechnical Institute (NGI) simple shear (SS) apparatus [19] was used in this study. The cyclic SS setup closely simulates vertically propagating horizontal shear waves while providing a simple configuration for applying K_0 -conditions, thereby confining the soil element. Subsequently, test data may be obtained by testing samples at different amplitudes of cyclic shear stress.

The apparatus is fully computer-controlled and consists of a pneumatic, stress-controlled system that generates cyclic shear stress.

The test sample has a diameter of 71 mm and a height of 20 mm. It is enclosed in a flexible membrane, with lateral confinement provided by a stack of 31 circular, Teflon-coated, rigid aluminum rings, each 0.94 mm thick. Lubricant oil is applied between the rings to minimize friction. Vertical pressure is applied to the sample by the bottom platen, which is fixed horizontally but can move vertically. The upper and lower platens have two-mm-high needles to prevent sliding between the platens and the soil specimen.

To simulate a soil element subjected to an earthquake, the testing procedure was as follows (Fig. 2): After the consolidation step under K_0-conditions to the final vertical stress, σ'_{vc}, the specimen was subjected to cyclic shear stress-controlled loading, in the form of a sinusoidal wave at a frequency of 0.5 Hz (i.e., the dominant frequency during the 1985 Mexico City earthquake). The tests were terminated after 1 minute of loading (30 cycles). Two Series (A and B) of tests constitute an ongoing cyclic loading program on Mexico City soil.

In Series A (Table 1), tests were conducted using the "constant-height" test method (i.e., equivalent-undrained conditions), simple shear (CHSS) tests. All soil specimens were reconsolidated to an average effective consolidation stress $\sigma'_{vc} = 150\ kPa$. ($\sigma'_{vc} > \sigma'_{v0}$), where σ'_{vc} is the effective consolidation stress in the laboratory and σ'_{v0} is the effective vertical stress in the field. A monotonic CHSS test was conducted in accordance with ASTM D 6528 2017 [1] to obtain, τ_f, which served as a reference for the cyclic SS tests. During shearing, the volume was maintained constant by keeping the specimen height constant. Under such conditions, the changes in σ_v are equivalent to the change in pore pressure generated if the specimen were saturated while shearing in undrained conditions [19, 21]. The strain rate was 1% per hour, and the test was terminated at a maximum shear strain of 30%.

Five cyclic tests were performed by first applying a consolidation stress to the specimen $\sigma'_{vc} = 150\ kPa$, and then subjecting the specimens to a uniform cyclic shear stress amplitude τ_{cyc} that varied from 10 to 50 kPa (i.e., cyclic stress ratio, $R_{cyc} = \frac{\tau_{cyc}}{\tau_f} = 0.2\ to\ 1$). The number of cycles applied in each test was approximately 30.

In Series B (Table 2), all soil specimens were reconsolidated to an average effective consolidation stress $\sigma'_{vc} = 150\ kPa$ ($\sigma'_{vc} > \sigma'_{v0}$). Cyclic simple shear tests were performed under constant vertical stress, CVSSS (i.e., equivalent-drained conditions) to determine vertical strain accumulation characteristics (cyclic subsidence) in Mexico City soil specimens subjected to a uniform cyclic shear stress amplitude τ_{cyc} that varied from 14 to 56 kPa (i.e., cyclic stress ratio, $R_{cyc} = \frac{\tau_{cyc}}{\tau_f} = 0.2\ to\ 0.8$). The bottom plate was left

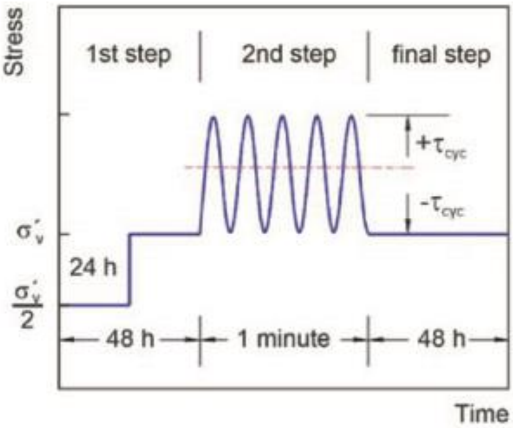

Fig. 2 Schematic testing loading sequence of soil specimens.

Table 1 Summary of testing program — Series A.

Test	z	w	σ'_v	τ_{cyc}/τ_f	τ_{cyc}	γ_{cyc}	Δu_{cyc}	$\Delta u_{cyc}/\sigma'_v$
No.	(m)	(%)	kPa		kPa	%	kPa	
MC1	11.90	271.16	150	CHSS MONOTONIC TEST (τ_f= 50 kPa)				
MC2	12.35	225.38	150	0.2	10	0.048	2.059	0.014
MC3	12.30	268.19	150	0.4	20	0.202	2.501	0.017
MC4	12.25	228.33	150	0.6	30	0.414	6.314	0.042
MC5	12.20	274.70	150	0.8	40	0.779	8.386	0.056
MC6	12.60	280.15	150	1.0	50	1.257	24.651	0.164

Table 2 Summary of testing program – Series B.

Test	z	w	σ'_v	τ_{cyc}/τ_f	τ_{cyc}	γ_{cyc}	ε_v
No.	(m)	(%)	kPa		kPa	%	(%)
MC7	11.8	271.16	150	CVSSS MONOTONIC TEST (τ_f= 70 kPa)			
MC8	11.85	296.44	150	0.2	14	0.155	0.053
MC9	11.95	305.71	150	0.4	28	0.446	0.120
MC10	12.10	303.81	150	0.6	42	0.907	0.145
MC11	12.15	286.58	150	0.8	56	1.676	0.238

free to move vertically during cyclic shear loading, allowing the specimen to settle freely while the vertical strain was monitored. During those tests, no back pressure was applied, and pore pressure was not measured.

3. Test Results

3.1 Undrained Stress-controlled Simple Shear Tests (Series A)

3.1.1 Monotonic behavior

The MC1 test exhibited a well-defined failure. The peak τ_f = 50.8 kPa ($\frac{\tau_f}{\sigma'_{vc}} = 0.34$) occurred at a shear strain, γ_f of 14.4%. The induced pore pressure, Δμ, estimated from the change in total vertical stress required to maintain a constant height, rose steadily with shear strain. The excess pore pressure ratio, $r_u = \frac{\Delta u}{\sigma'_{vc}}$, was approximately 0.39. The maximum value observed at large strains was about 0.56. The low values of the induced normalized pore pressure at peak deformation are noteworthy. This behavior is typical of soils in Mexico City, which are diatomaceous [22].

3.1.2 Cyclic behavior

Figs. 3 and 4 summarize the results of Series A. Fig. 3 shows the development of cyclic shear strain, $\gamma_{cyc} = (\gamma_{cyc\ max} - \gamma_{cyc\ min})/2$, during undrained loading at various cyclic stress levels. Two response patterns are evident. For cyclic stress ratios less than 0.8 (80% of the static shear strength), the development of peak strain, after 3 or 4 cycles, tends toward equilibrium plateaus with the number of cycles. For cyclic stress ratios higher than 0.8, the increase in shear strain shows an upward trend after a certain number of cycles, indicating a gradual degradation of stiffness. The above facts corroborate [23] earlier findings.

Fig. 3 shows that the relationship between the shear deformation γ_{cyc} and N is nearly constant beyond the initial three cycles, with the ordinate depending on the value of R_{cyc} applied to soil specimens; therefore, a simple expression can be derived:

$$\gamma_{cyc} = 1.2R_{cyc}^2$$

That expression permits estimating the cyclic strain based on the shear stress amplitude, up to R_{cyc}< 0.8 for $\sigma'_{vc} = 150\ kPa$.

Fig. 4 shows the development of the mean cyclic induced pore-pressure, denoted as, $\Delta\mu_{cyc\ mean} = (\Delta\mu_{cyc\ max} + \Delta\mu_{cyc\ min})/2$ with N. Two response patterns are clearly distinguishable. For a cyclic stress ratio less than 0.8, pore pressure increases steadily with N, reaching very

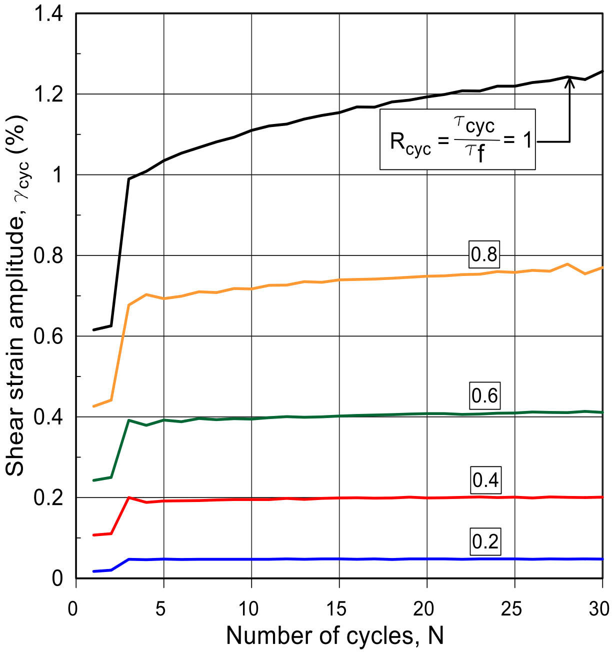

Fig. 3 Summary of shear strain results of undrained stress-controlled cyclic simple shear tests of Series A.

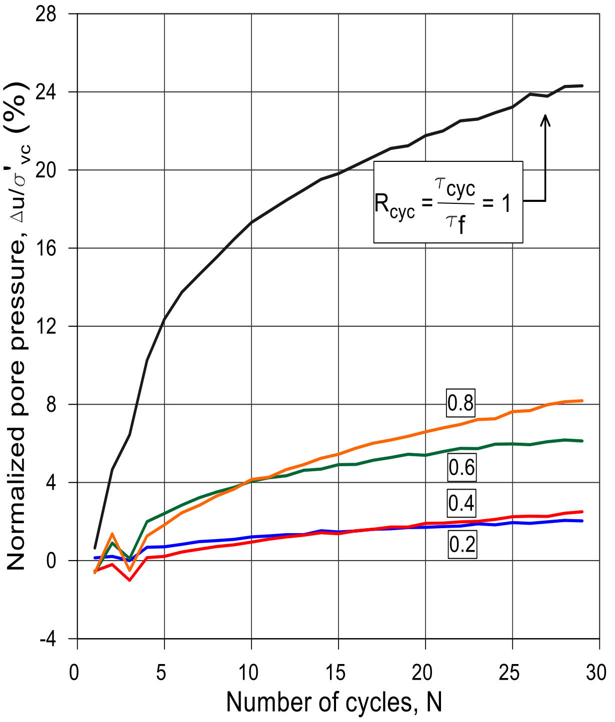

Fig. 4 Summary of normalized pore pressure results of undrained stress-controlled cyclic simple shear tests of Series A.

low values around N = 30. For cyclic stress ratios greater than 0.8, an initial sharp rise is followed by a steady increase in pore pressure with N. At higher cyclic stress levels, large strains would occur, and the accumulation of pore pressure would cause the effective stress path to migrate toward the failure envelope. Hence, failure cannot be ruled out if the number of cycles increases.

3.2 Equivalent Drained Cyclic Stress-controlled Simple Shear Tests (Series B)

3.2.1 Monotonic Behavior

The MC 7 test showed that shear stress continued to increase throughout the test; consequently, a failure value could not be determined. The figure shows that the soil specimen reaches a $\tau_f = 70\ kPa$ ($\frac{\tau_f}{\sigma'_{vc}} = 0.47$) at a shear strain γ of 30%.

3.2.2 Cyclic Behavior

All the tests for Series B were performed by first applying a consolidation stress to the specimen $\sigma'_{vc} = 150\ kPa$, and then subjecting the specimens to a uniform cyclic shear stress amplitude τ_{cyc} that varied from 14 to 56 kPa (i.e., $R_{cyc} = \frac{\tau_{cyc}}{\tau_f} = 0.2\ to\ 0.8$).

The vertical stress was kept constant during cyclic shearing, and the specimens were allowed to settle freely. Continuous readings of vertical deformation were taken, enabling the measurement of vertical strains, $\varepsilon_{v\,cyc} = \Delta H/H$, to be evaluated as a function of the number of strain cycles (*N*). Where ΔH is the settlement, and H is the height of the specimen before shearing.

Fig. 5 (MC9 test) shows a typical set of results from an equivalent drained, cyclic, stress-controlled simple shear test. Fig. 5a shows the time record of the applied shear stress on the soil specimen. Fig. 5b displays a typical time record of cyclic shear strain, γ_{cyc}. The time history of the shear strain is uniform, with a very slight asymmetry that depends on the initial loading direction despite the symmetric two-way cyclic loading. Fig. 5c shows, first, the progressive increase in the permanent (or residual) vertical strain, ε_{vp}, of the soil specimen with time. The permanent vertical strain, ε_{vp}, corresponds to the vertical strain remaining after the loading cycle when the shear stress returns to zero (denoted by a dashed line in Fig. 5c). Second, superimposed on the steady increase, there is an essential cyclic component of vertical deformation that remains constant with respect to time. Fig. 6 summarizes the relationship between ε_{vp} and R_{cyc}, and N.

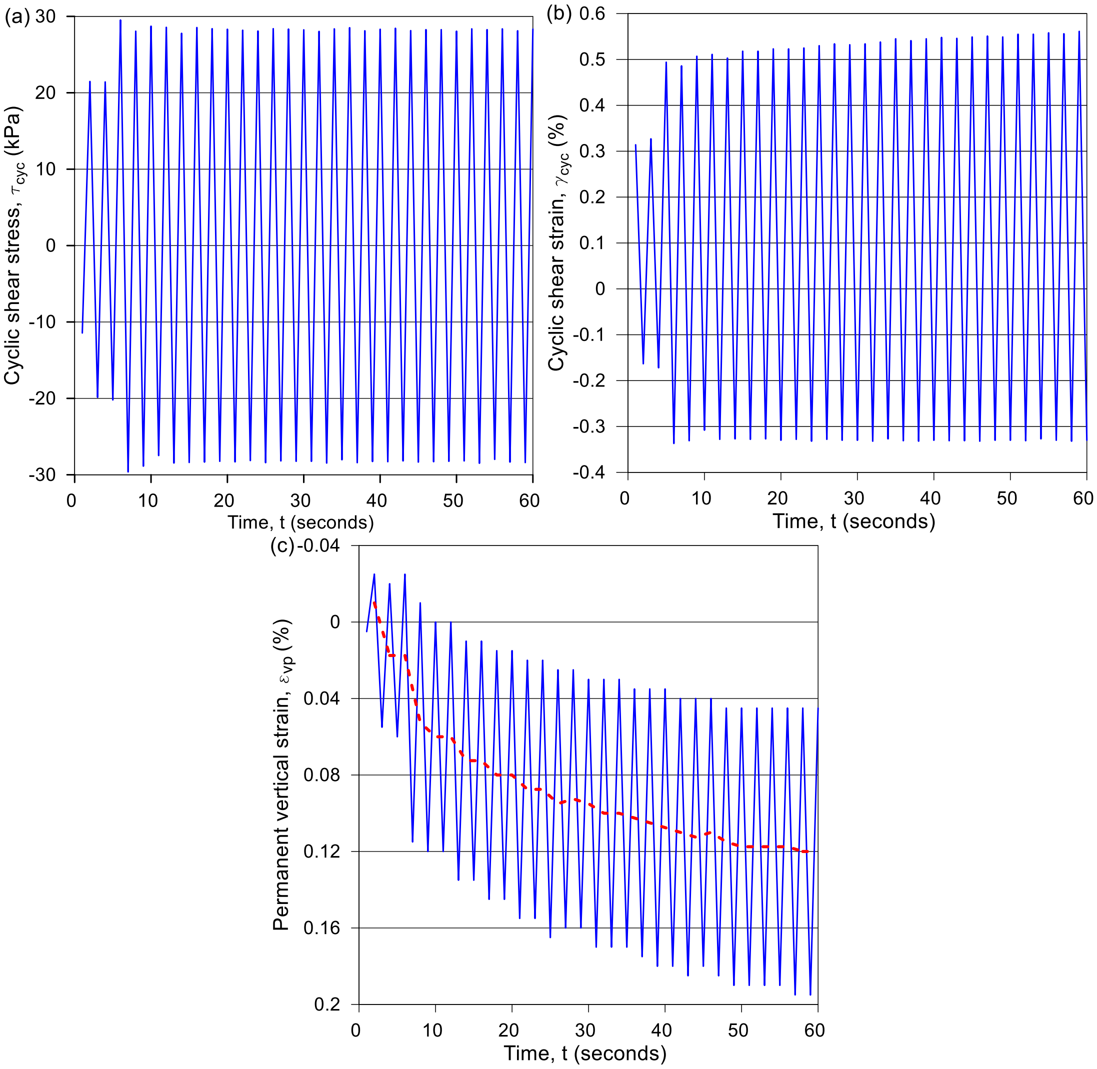

Fig. 5 **Drained stress-controlled cyclic simple shear tests on specimen MC9.**

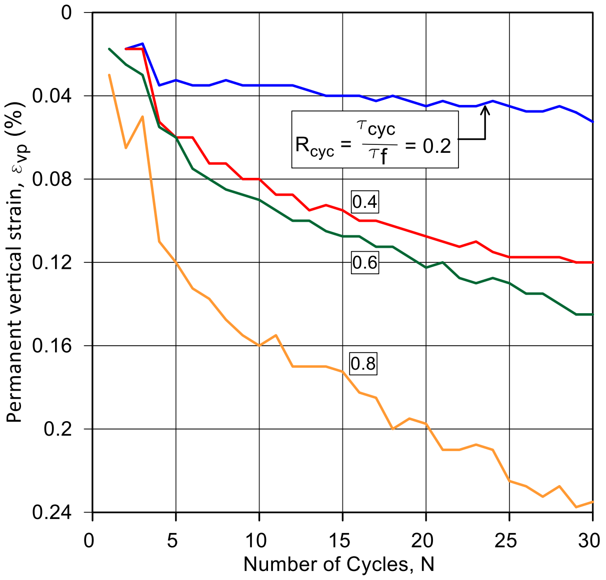

Fig. 6 **Summary of results of drained stress-controlled cyclic simple shear tests of Series B.**

4. Summary and Conclusions

Mexico City has experienced excessive settlement and tilting during the past three large-magnitude earthquakes. To assess the sudden subsidence phenomenon, two series of cyclic simple shear tests were conducted on Mexico City soil specimens. Given the limited amount of reliable test data available to date, only tentative conclusions can be drawn at this stage.

Measurements of vertical strain in Mexico City soil samples under cyclic simple shear loading and drained conditions showed that immediate vertical strain occurs. The magnitude of vertical strain depends on the applied shear stress amplitude and the number of cycles.

One mechanism hypothesized to explain how cyclic shear stresses deform the soil sample is the cyclic movement of soil particles, which produces changes in clay structure through physical sliding and reorientation, resulting in progressive vertical strain and, consequently, immediate subsidence during cyclic loading.

Further investigation is needed to robustly determine the underlying causes of the sudden settlements in Mexico City. Nonetheless, the findings presented here provide a starting point for future examination.

Acknowledgments

The DGAPA, UNAM, has supported this study. Thanks are due to Arturo Moreno for their assistance during the testing program. Editing of the manuscript was completed with the assistance of Guadalupe Salinas.

References

[1] Silver, M. L. and Seed, H. B. 1971. “Volume Changes in Sand During Cyclic Loading”, *Journal of Soil Mechanics and Foundations Division* 97 (9): 1171-1180.

[2] Youd, T. L. 1972. “Compaction of Sands by Repeated Shear Straining”, *Journal of the Soil Mechanics and Foundation Division* 98 (SM7): 709-724.

[3] Seed, H. B., and Silver, M. L. 1972. “Settlement of Dry Sands During Earthquakes”, *Journal of Soil Mechanics and Foundations Division* 98 (SM4): 381-397.

[4] Pyke, R., Seed, H. B. and Chan, C. K. 1975. “Settlement of Sands Under Multidirectional Shaking”, *Journal of the Geotechnical Engineering Division* 101(GT4): 379-398.

[5] Chu, H. H., and Vucetic, M. 1992. “Settlement of Compacted Clay in a Cyclic Direct Simple Shear Device”, *Geotechnical Testing Journal* 5 (4) 371-379.

[6] Stewart, J. P., Smith, P. M., Whang, D. H., and Bray, J. D. 2002. “Documentation and Analysis Od Field Case Histories of Seismic Compression During the 1994 Northridge, California, Earthquake”. *PEER Report 2002/09,* University of California, Berkeley.

[7] Tsukamoto, Y., Ishihara, K., and Sawada, S. 2004. “Settlement of Silty sand Deposits Following Liquefaction During Earthquakes.” *Soils and Foundations* 44 (5): 135-148.

[8] Rong, W. and McCartney, J. S. 2019. “Effect of Suction on the Drained Seismic Compression of Unsaturated Sand, *E3S Web of Conference 92, 08004*, IS-Glasgow.

[9] Whang, D. H., Stewart, J. and Bray, J. D. 2004. “Effect of Compaction Conditions on the Seismic Compression of Compacted Fill Soils.” *Geotechnical Testing Journal* 27 (4): paper ID GTJ11810.

[10] Wang, L., Sun, J. Huang, X., Xu, S., Shi, Y., Qiu, R., and Zhang, Z. 2011. “A Field Testing Study on Negative Skin Friction Along Piles Induced By Seismic Subsidence of Loess.” *Soil Dynamics and Earthquake Engineering* 31: 54-58.

[11] Wang, Y., Wei, T., Ren, Y., Gao, Y., and Yang, Q. 2022. “Experiment Study on Normalized Compression Behavior of Marine Diatomaceous Soil Considering Microstructure of Biogenic Silica.” *Applied Ocean Research* 125: 103254.

[12] Ohara, S., and Matsuda, H. 1988. “Study on the Settlement of Saturated Clay Layer Induced by Cyclic Shear.” *Soils and Foundations* 28 (3): 103-113.

[13] Hyodo, M., Yasuhara, K., and Murata, H. 1988. “Earthquake Induced Settlement in Clays.” *Proc. Ninth World Conference on Earthquake Engineering*, Tokyo-Kyoto, Japan.

[14] Yasuhara, K., and Andersen, K. H. 1991. “Recompression of Normally Consolidated Clay After Cyclic Loading.” *Soils and Foundations* 31 (1): 83-94.

[15] Yasuhara, K., Konami, T., Hyodo, M., and Hirao, K. 1991. “Earthquake-Induced Settlement in Soft Grounds.” In: *Proc. Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics*. St louis, Missouri.

[16] Nhan, T. T., Matsuda, H., Sato, H., Thie, D. Q., Tien, P. V., and Nhan, N. T. T. 2022. Pore water pressure and settlement of clays under cyclic shear: effects of soil plasticity and cyclic shear direction, *Journal of*

Geotechnical and Geoenvironmental Engineering 148 (2): 04021185.

[17] Díaz-Rodríguez, J. A., and Santamarina, C. 2001. "Mexico City Soil Behavior at Different Strains — Observations and Physical Interpretation." *ASCE Journal of Geotechnical and Geoenvironmental Engineering* 127 (9): 783-789.

[18] Díaz-Rodríguez, J. A., Lozano-Santa Cruz, R., Dávila-Alcocer, V. M., Vallejo, E. and Girón, P. 1998. "Physical, Chemical and Mineralogical Properties of Mexico City Sediments: A Geotechnical Perspective." *Canadian Geotechnical Journal* 35 (4): 600-610.

[19] Bjerrum, L., and Landva, A. 1966. "Direct Simple-Shear Test on Norwegian Quick Clay." *Géotechnique* 16 (1): 1-20.

[20] ASTM D 6528 2017. "Standard Test Method for Consolidated Undrained Direct Simple Shear Testing of Cohesive Soils." *ASTM International*, West Conshohochen, PA.

[21] Dyvik, R., Berre, T., Lacasse, S., and Raadim, B. 1987. "Comparison of Truly Undrained and Constant Volume Direct Simple Shear Tests." *Géotechnique* 37 (1): 3-10.

[22] Díaz-Rodríguez, J. A. 2011. "Diatomaceous Soils: Monotonic Behavior." In: *International Symposium on Deformation Characteristics of Geomaterials*, Seoul, Korea.

[23] Díaz-Rodríguez, J. A. 1989. "Behavior of Mexico City Clay Subjected to Undrained Repeated Loading." *Canadian Geotechnical Journal* 26 (1): 159-162.