UNDRAINED SHEAR BEHAVIOR OF MEXICO CITY SEDIMENTS DURING AND AFTER CYCLIC LOADING

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SUMMARY

This paper describes a study of the stress-strain-pore water pressure behavior of Mexico City sediments subjected to one hundred sinusoidal stress-controlled cycles of loading. The influence of factors, such as, yielding stress, confining pressure, cyclic stress level and number of cycles was examined. Also the influence of cyclic loading on the post-cyclic undrained stress-strain characteristics of Mexico City sediments was studied. An isotropic consolidation test was conducted using a triaxial-cell method stress to define the yielding stress, σ'_y. Then, two series of CU cyclic triaxial tests were conducted under different isotropic consolidation stress, to study the process of destructuration of the samples.

INTRODUCTION

Considerable research on the behavior of clays during cyclic loading has been done during last four decades. Clay behavior has been studied under constant stress and constant-strain amplitude cyclic loading, both one-way and two-way loading, under isotropic and anisotropic consolidation stress, using triaxial, simple shear and torsional shear apparatus (Sangrey [13]; Andersen [1, 2]; Azzouz et al. [3]).

Sangrey et al. [13] has shown that at cyclic stress levels below a certain magnitude called the critical level of repeated loading (CLRL) the clay would develop strain until a state of equilibrium is reached. At this stage, a closed hysteresis loop occurs with subsequent cycles. At cyclic stress levels above CLRL, the strain and pore water pressure increase as the number of cycles increases.

An important consideration in the seismic design of foundation on clay is the undrained response of the clay during and after cyclic loading. Post-cyclic behavior of clay is generally considered to depend on maximum strain developed during cyclic loading. Some conclusions from these studies are conflicting. Post-cyclic stiffness of clays undergoing small axial strain during cyclic loading has been shown to increase or decrease when compared to pre-cyclic stiffness (Motherwell and Wright [12]; Taylor and Bacchus [15]).

The ratio of post-cyclic and pre-cyclic undrained strength is generally considered to be related to the ratio of maximum axial strain developed during cyclic loading and the axial strain at peak strength during pre-cyclic monotonic loading. The literature suggests that the undrained strength ratio is related not only to the maximum axial strain during cyclic loading, but it also depends on other cyclic loading parameters, such as cyclic stress level, number of cycles, residual strain and pore pressure induced due to cyclic loading.

Koutsoftas [10] reported stiffness degradation as $\varepsilon_{a\ max}$ increases in constant-stress amplitude cyclic loading on two marine clays. Motherwell and Wright [12] found that clays that reached an equilibrium condition during constant stress amplitude one-way cyclic loading demonstrate somewhat stiffer stress-strain response on subsequent static loading, such a tendency may also be noticed in the data reported by Andersen [1] in similar tests on Drammen clay. Similarly Frydman et al. [7] showed only an 8% reduction in strength following a mobilized cyclic shear strain of 13% in stress-controlled simple shear testing of plastic marine clay.

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TESTING PROGRAM

An important group of geotechnical problems requires knowledge of the cyclic undrained shear strength, the strain of soil induced by cyclic loading and the peak strength of soil after a limited number of repeated loading cycles. Thus, a specific testing program was followed.

The studied site is located in the lacustrine zone of Mexico City, in the neighborhood of one of the most damaged area during the earthquake of September 1985. The sediments from Mexico City are complex mixtures of crystalline minerals and amorphous material that challenge a simple nomenclature; they are heterogeneous volcanic and lacustrine sediments with a proportion and variety of microorganisms that add compounds to the water present in the sediment. Physical, chemical and mineralogical properties of Mexico City sediments have been described by Díaz-Rodríguez *et al.* [6] and will not be repeated here.

All the samples were obtained using a 127 mm-diameter thin-walled Shelby tube. The properties of Mexico City clay are usually variable from place to place and with depth, but the samples used here were relatively homogeneous with water content varying from 300% to 314% with an average of 307% for Series 1 and from 223% to 302% with an average of 262% for Series 2. Some physical properties of the clay are summarized as followed: The liquid limit is 350% the plasticity index is 294%, and the specific gravity is 2.5. In the following, the term "Mexico City sediments" refers only to the soil that was tested in this investigation.

All tests were carried out using the triaxial apparatus. A low-friction continuously air-leaking hydrostatic seal was employed, this enabled confident measurements of axial load outside the cell. The final trimmed size of the test specimens was 36 mm in diameter and 80 mm in height. The specimens were consolidated to an all round effective stress (σ'_c) using a back pressure of 100 kPa. This back pressure was sufficient to achieve a satisfactory degree of saturation (pore-pressure parameter B = 0.96). During consolidation, double drainage was permitted for 24 hours. Drainage was then terminated and the sample left undrained under pore pressure equalization.

The present study includes two series of triaxial tests. Series 1 was conducted on specimens consolidated isotropically in the structured branch (over-consolidated range) and Series 2 was conducted on specimens consolidated isotropically in the structured branch but close to the yielding stress. For each series three pre-cyclic undrained monotonic compression tests were conducted to establish reference data for cyclic loading and post-cyclic monotonic loading tests. Pre-cyclic tests were done under strain controlled conditions at a strain rate of 1% per hour.

CU cyclic tests were conducted using constant stress amplitude cyclic loading at cyclic stress levels varying from 0.36 to 1.0 of static undrained strength for the Series 1 and from 0.38 to 0.87 of static undrained strength for the Series 2. Since two-way cyclic loading is more damaging than one way cyclic loading, it was decided to study soil behavior under two-way cyclic loading.

If the sample did not fail during the cyclic loading, the excess pore pressure was allowed to equalize under zero deviator stress for 12 hours, and a post-cyclic undrained monotonic compression tests were done under strain controlled conditions at a strain rate of 1% per hour. The loading scheme employed in this study is schematically illustrated in Figure 1.

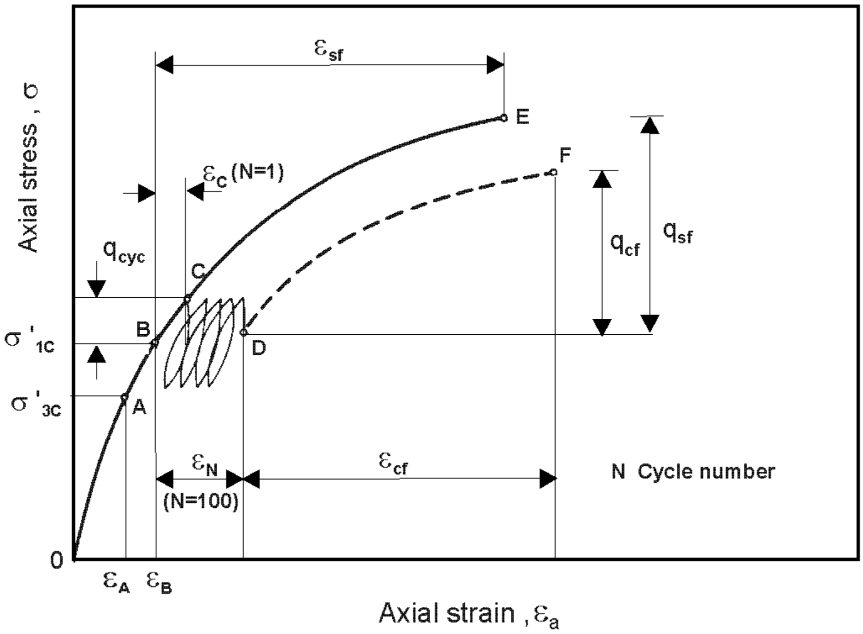


Figure:1 testing program scheme and definition

TEST RESULTS

An isotropic consolidation test was conducted to establish the yielding stress, σ'_y = 121 kPa, that corresponds to the passage from the structured branch (over-consolidated branch) to the beginning of the destructured (normally consolidated) branch (Díaz-Rodríguez et al. [5]). A total of 19 triaxial tests were performed. A summary of the test program is shown in Table 1.

Series 1 consisted of three CU-static tests and six CU-cyclic tests on the structured branch at effective consolidation stresses varying between 51.5 and 57.2 kPa, with an average of 54.3 kPa. The Series 2 consisted of three CU-static tests and seven CU-cyclic tests at effective consolidation stresses varying between 84.3 and 95.3 kPa, with an average of 89.8 kPa.

Test	Water Cont	Confining P	Pre-cyclic Monotonic Loading			Cyclic Loading							Post-cyclic Monotonic Loading		
	w	σ'_c	q_{sf}	ε_{sf}	u_{sf}	q_{cyc}	R	N	ε_N	$-\varepsilon$	$+u_{cyc}$	$-u_{cyc}$	q_{cf}	ε_{cf}	u_{cf}
No.	%	kPa	kPa	%	kPa	kPa	—	—	%	%	kPa	kPa	kPa	%	kPa
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
Series 1															
MX 1-1	300	53	96.6	3.2	41										
MX 1-2	301	51.8				35.2	0.36	100	0.68	0.81	6.38	9.96			
MX 1-3	300	51				54.1	0.56	100	1.16	1.88	16.8	10.14	103.3	3.59	34.5
MX 1-5	302	56.7	110.9	4.4	46.8										
MX 1-4	300	51				63	0.57	100	1.27	2.5	34	17.32	114.4	4.76	32.9
MX 1-6	301	54.6				74.9	0.67	100	2.1	8	82.6	48.3			
MX 1-7	303	53.4	85	4.6	38										
MX 1-9	313	53				73	0.86	28	5	24.3	70.3	21.1			
MX 1-8	314	51.5				85.6	1	6	8	24.5	66.5	10.7			
Series 2															
MX 2-1	292.7	84.4	104.7	8	61.6										
MX 2-3	223.1	87.7				44	0.42	100	0.57	1.77	4.7	2.5			
MX 2-2	291	84.6				79.2	0.76	100	1.75	4.8	42.4	3.7	105.6	6.74	32.3
MX 2-4	217	92.6				92	0.87	45	4	25.6	64.1	2			
MX 2-6	296.1	89.9	113.2	8.1	64.2										
MX 2-7	301.9	88.4				57.7	0.51	100	0.65	1.7	14.9	1.16	102.3	8.01	52.6
MX 2-5	298.4	86.7				92.8	0.82	84	0.66	25.5	51.8	18.6			
MX 2-9	235.6	95.3	110	7.4	66.3										
MX 2-10	227.9	84.7				42.1	0.38	100	0.41	0.54	10.9	1.9	98.5	7.15	55.2
MX 2-8	235	92.7				59.8	0.54	100	0.91	1.26	21.8	8			

Behavior during CU-cyclic loading tests

The term cyclic stress ratio R (= q_{cyc}/ q_{sf}) is defined herein as cyclic shear stress (q_{cyc}) divided by the peak shear stress in a CU-static compression test (pre-cyclic monotonic test, q_{sf}) run at axial strain rate of 1% per hour.

The maximum axial strain with the number of cycles (N), for Series 1 and 2, are shown in Figures 2 and 3 respectively, to various cyclic stress ratios (R). It may be noted that the axial strain response is softer in extension (negative strain) than in compression. This fact is attributed to inherent anisotropy in natural clays, and partly due to the fact that samples were subjected to constant amplitude cyclic load.

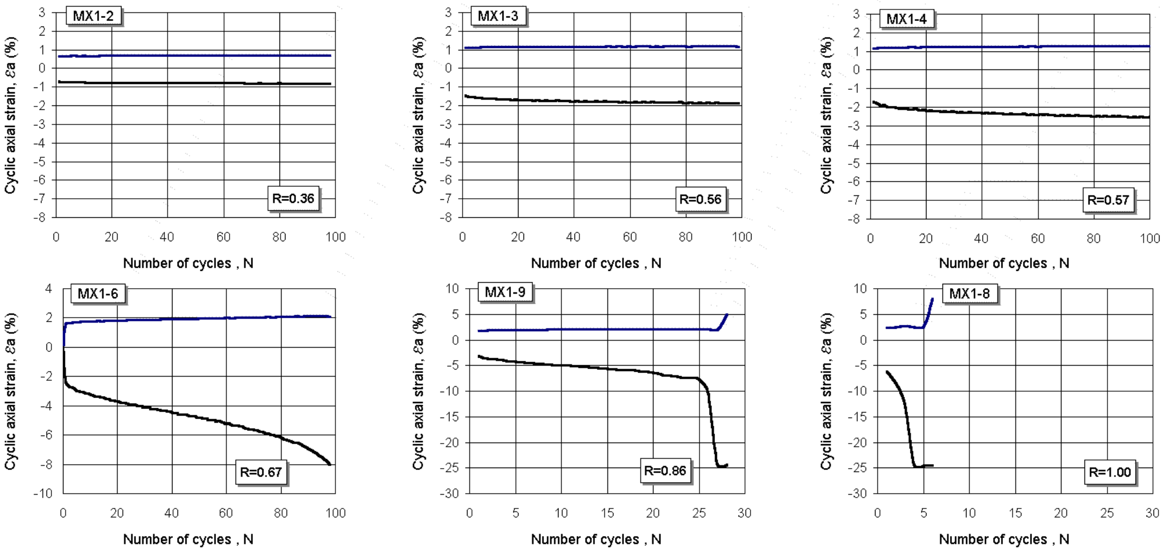


Figure: 2 Development of maximum axial strain during CU cyclic loading to various cyclic stress ratios. Series 1

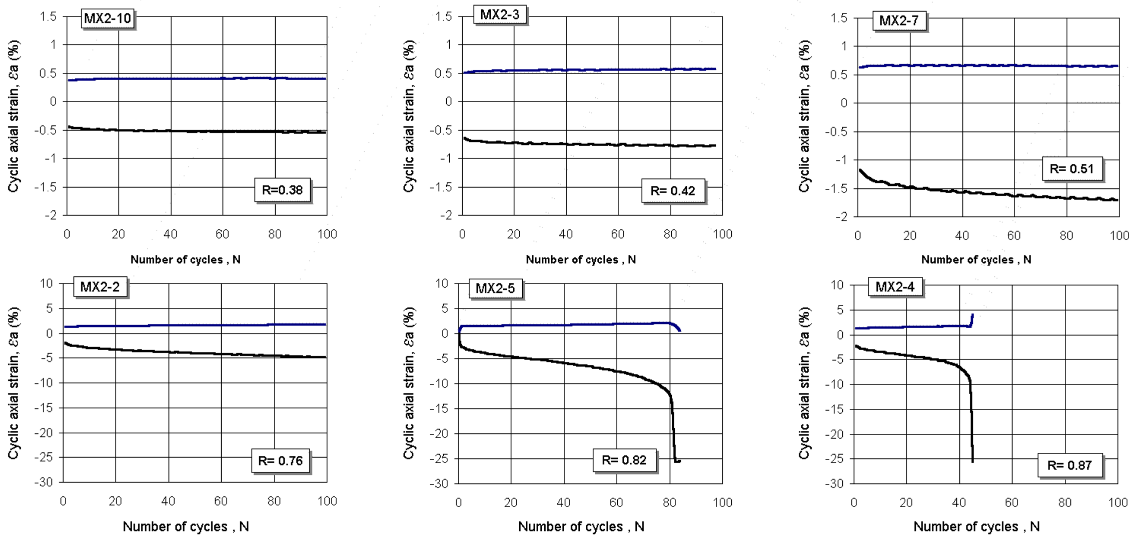


Figure: 3 Development of maximum axial strain during CU cyclic loading to various cyclic stress ratios. Series 2

During the extension phase the net area of the sample decreases whereas it increases in compression. It appears that beneath a R = 0.80 the compression strain development seems to be almost constant with the number of cycles.

The developed of maximum pore water pressure with the number of cycles, for Series 1 and 2 are shown in Figures 4 and 5 respectively, to various cyclic stress ratios (R). It may be noted that although the general pattern of pore water pressure response is similar between Series 1 and 2, the levels of pore water pressure generated are different, mainly in negative pore pressure.

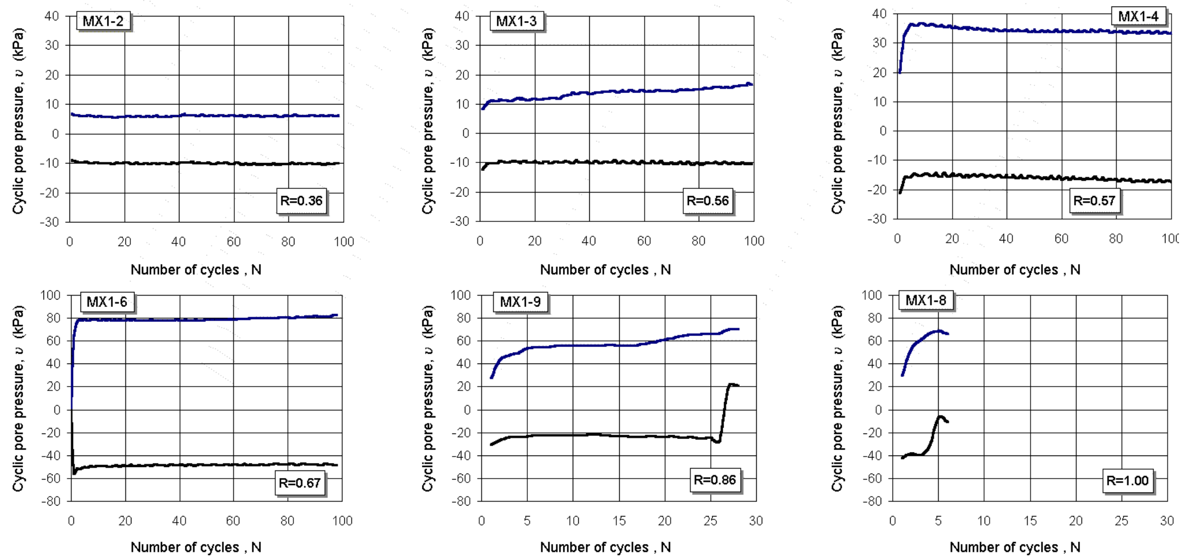


Figure: 4 Development of maximum pore pressure during CU cyclic loading to various cyclic stress ratios. Series 1

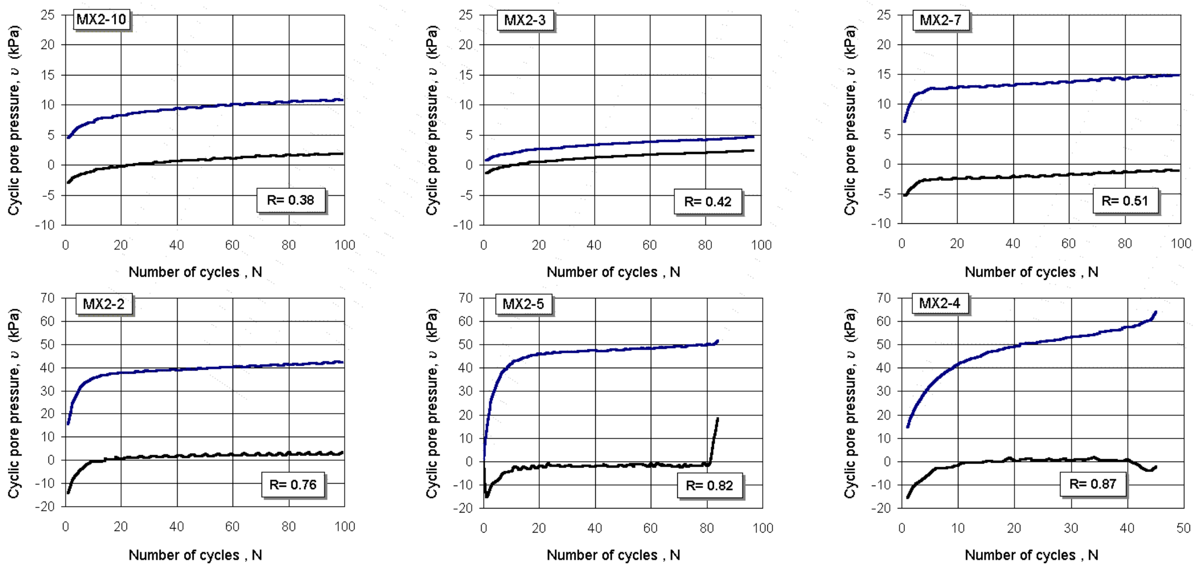


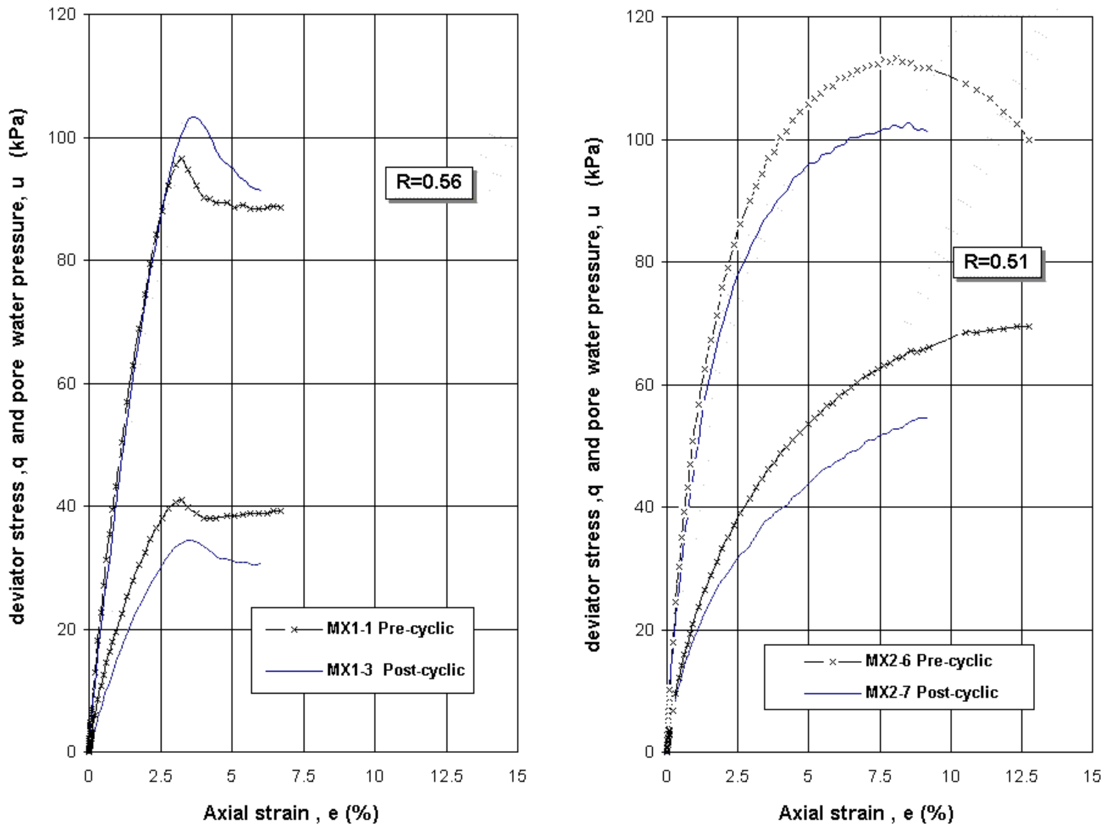
Figure: 5 Development of maximum pore pressure during CU cyclic loading to various cyclic stress ratio. Series 2

This fact is attributed to inherent anisotropy in natural clays and partly due to the effect of soil structure; in Series 2 the confining pressure is closer to the yielding stress, thus, it is almost at the beginning of the destructured branch.

Strength after CU-cyclic loading tests

Post-cyclic behavior of clay is linked to the parameters of cyclic loading (e.g. confining pressure and cyclic stress ratio). Taylor and Bacchus [15] and Matsui and Abe [11] have reported that soil previously subjected to cyclic loading generally exhibits softer stress-strain response when compared to the pre-cyclic behavior.

Typical results of post-cyclic monotonic tests on samples previously subjected to cyclic loading for Series 1 and 2 are presented in Figure 6. For Series 1 and cyclic stress ratio below 0.80, less pore water pressure was generated during post-cyclic monotonic tests than during pre-cyclic monotonic tests. It can be seen that the stress-strain curve displays a drop after the peak at an axial strain of 3.6%. Comparing the curves presented in the Figure 6b show a reduction in strength due to cyclic loading and an increase in the axial strain to 7.7%.



a) b)

Figure: 6 comparison between pre-cyclic and post-cyclic cu strength

In a previous paper (Díaz-Rodríguez, [4]), a non-dimensional plot was defined, plotting the failure ratio R_f (=q_{cf} / q_{sf}) defined as the deviator stress at failure after repeated loading, q_{cf}, divided by the static strength, q_{sf}, versus cyclic stress ratio, R. For this paper, the experimental points for both Series are shown in Figure 7. It can seen that it is possible to identify two different patterns of behavior: (a) for R < 0.80 an state of equilibrium is reached during cyclic loading, R_f has an average of 105% for Series 1 and 96.5% for Series 2. (b) for R > 0.80 failure is reached in both series during the cyclic loading.

CONCLUSIONS

The present study includes a battery of CU-cyclic triaxial tests and CU-static triaxial tests on Mexico City sediments. The influence of factors, such as, the yielding stress, confining pressure, cyclic stress level and number of cycles was examined. Then the main conclusions can be summarized as follows:

1. The Mexico City sediments exhibit elastic behavior in spite of its very high water content, if the cyclic stress ratio R (= q_{cyc}/ q_{sf}) < 0.80.
2. It is possible to identify two different patters of behavior: a) If R < 0.80 a state of equilibrium is reached during the cyclic loading and the strength loss is less than 5 % and (b) if R > 0.80 all the samples failure during the cyclic loading.
3. The potential for strength loss and failure of Mexico City sediments under cyclic tests depends on the cyclic stress ratio applied.
4. Then, is possible to define a threshold cyclic stress ratio, $R_D \approx 80\%$ for Mexico City sediments.

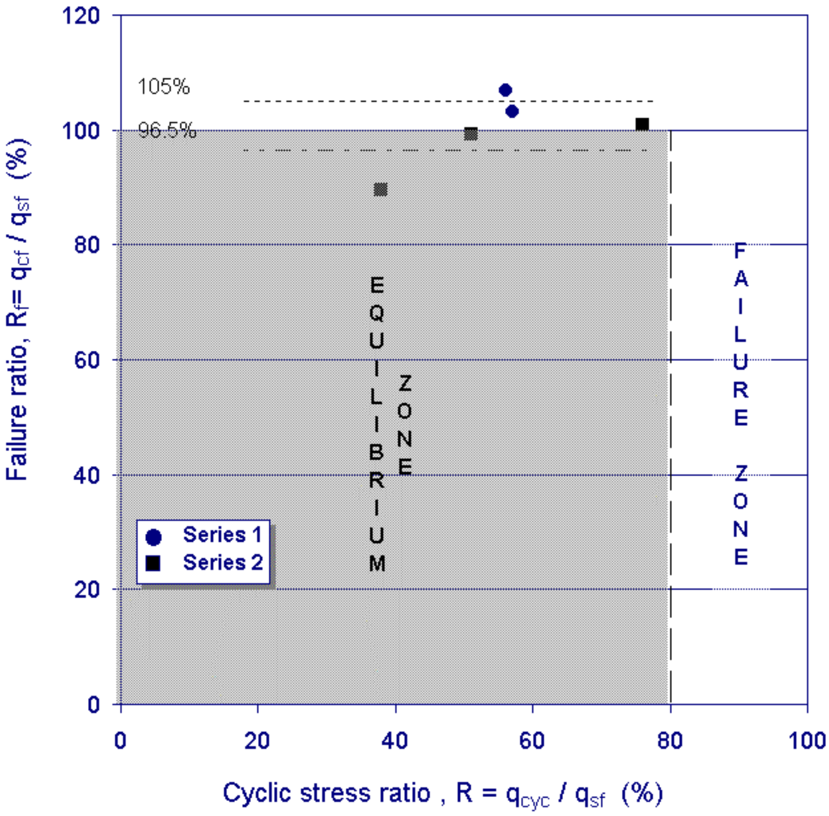


Figure: 7 failure ratio vs cyclic stress ratio

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