Behavior of Cohesive Soils Subjected to Cyclic Loading: An Extensive Review of Pertinent Literature

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Research Report

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Chapter 1

Introductory Remarks

Beginning in the the 1960's, the behavior of cohesive soils subjected to cyclic loading histories has been investigated by many researchers. The goal of these investigations has been to gain a better understanding of

- Changes in the total and effective stress.
- Evolution of pore pressure.
- Degradation of the elastic moduli.
- The shape of stress-strain curves and stress-strain loops.
- Post-cyclic behavior.

This report summarizes the experimental studies that have been performed so as to gain insight into the response of cyclically loaded, saturated cohesive soils. The results of the aforementioned studies are next synthesized. This is followed by a summary of experimental findings related to the post cyclic response of cohesive soils. A set of conclusions completes the report.

1.1 Experimental response of cohesive soils subjected to cyclic loading

The dynamic response of cohesive soils has been rather widely studied using different experimental techniques. These include cyclic triaxial, cyclic direct simple shear, resonant column and bender element tests. The main difference between these tests is the strain amplitude to which a soil sample is subjected. For example, cyclic triaxial and cyclic direct simple shear tests typically correspond to the large strain domain. They are thus particularly well-suited for studying important features such as excess pore fluid pressure generation, stiffness degradation, and post-cyclic shear strength. By contrast, resonant column and bender element tests are used to study the small strain response; they are thus used to evaluate the maximum value of the elastic shear modulus (G_{max}).

The response of cohesive soils subjected to cyclic loading is known to be affected by different factors. The most important of these factors are the soil type, the stress or consolidation history, the specific test conditions.

The type of soil is commonly quantified by the value of the plasticity index (I_p). Chapter 3 presents further details pertaining to the role of the plasticity index in quantifying the response of cyclically loaded cohesive soils.

Regarding the consolidation history, normally consolidated and overconsolidated clays respond differently to cyclic loading. When subjected to cyclic loading, normally consolidated cohesive soils exhibit an apparent overconsolidation, and tend to be more resistant to undrained cyclic loading than overconsolidated ones. This conclusion has been established by subset of investigators [1; 6; 18; 44; 56; 65; 73; 75] that tested cohesive soils over a rather large range of overconsolidation ratios ($OCRs$) that ranged from 1 to 50. In addition, anisotropically consolidated soils are known to behave differently from ones that are consolidated isotropically.

The loading conditions imposed in a given cyclic test on a cohesive soil affect its response. In particular, the cyclic response of such soils depends on the frequency and type of cyclic loading, as well as on the cyclic stress level (or cyclic strain amplitude). Cyclic loading is generally applied under either stress- or strain-controlled conditions. In many past cyclic tests, the loading was applied at frequencies ranging between 0.1 and 0.5 Hz. However, in order to obtain reliable excess pore pressure measurements during cyclic loading, very low axial strain rates (e.g., $0.0002\%/min$ [48] and $0.4\%/hr$ [73]) and low frequencies (e.g., $0.001Hz$ [54; 52]) have been used.

Cyclic loading can be applied using either a one-way (non-reversal) or two-way (full reversal) history. In the former case, only positive deviator stress in the axisymmetric triaxial test or positive shear stress in the direct simple shear test are applied [2; 6; 9; 12; 16; 18; 20; 27; 28; 36; 38; 40; 43; 42; 49; 55; 68; 64; 75]. In the latter case, two-way or reversed loading, consisting of positive (compression) and negative (extension) stresses, is applied to the sample. Two-way loading is typically applied more quickly than, and tends to disturb a sample more than one-way loading [3; 4; 7; 14; 17; 21; 25; 29; 33; 44; 46; 47; 56; 57; 59; 58; 70; 71]. There are also instances where both one and two-way loading have been used in a given testing program [1; 19; 22; 23; 51; 69; 72]. Finally, irregular [45], harmonic [5], and "storm" [65; 66] loading have also been used.

Although, the majority of the experimental investigations were undertaken to study the strength of cohesive soils under cyclic loading, some investigations also focused on the degradation of elastic shear modulus (G) and on the damping ratio due to cyclic loading [59; 58; 60].

Finally, some investigations have been undertaken to better understand the post-cyclic behavior of cohesive soils. This included a study of consolidation due to the dissipation of excess pore pressure developed during cyclic loading[1] [21; 44; 65; 66; 69], and the post-cyclic strength under monotonic undrained axisymmetric triaxial [1; 6; 9; 18; 19; 25; 29; 38; 64; 70; 72], direct simple shear [4], and monotonic torsional loading [11].

1.2 Overview of the Report

This report organized as follows: Chapter 2 summarizes the experimental studies that have been performed so as to gain insight into the response of cyclically loaded, saturated cohesive soils. In Chapter 3 the results of the aforementioned studies, as well as the main conclusions drawn by the various researchers are next synthesized. This is followed, in Chapter 4, by a summary of experimental findings related to the post-cyclic response of cohesive soils. A set of conclusions is given in Chapter 5 and completes the report.

[1] In such cases a drainage period is imposed following an undrained cyclic event so as to allow for the dissipation of excess pore pressure.

Chapter 2

Review of Pertinent Literature

This chapter presents the main information from the most important papers that experimentally investigated the cyclic response of cohesive soils. All information presented herein was directly taken from the respective papers; some paragraphs were not changed in order to maintain the original comments of the authors. In addition, all figures were copied directly from the original papers.

Rather than following a chronological order, the material in this chapter is presented *alphabetically* by the last name of the first author of a given paper. In Chapter 3 the results of the aforementioned experimental investigations, as well as the main conclusions drawn by the various researchers are next synthesized.

2.1 Andersen et al. (1980) [1]

Samples of undisturbed plastic Drammen clay (a marine plastic clay) were tested using cyclic triaxial and cyclic simple shear tests. The physical properties of this clay are a moisture content (ω) of 52%, a specific gravity (G_s) of 2.76, a liquid limit (LL) of 55%, a plastic limit (PL) of 28%, and a plasticity induex (PI) of 27. The clay was consolidated in the laboratory beyond the in situ stresses to a vertical preconsolidation stess of $400kN/m^2$ and was then swelled to various consolidation stresses in order to get attain the desired OCRs. In particular, samples with OCRs of 1, 4, 10, 25 and 50 were tested. Preconsolidation was carried out under conditions of no lateral strain with $K_0 = 0.5$. Monotonic triaxial tests were run at a rate of 3% axial strain per hour, and monotonic shear tests were run at a rate of 4.5% axial strain per hour. Cyclic triaxial tests were run under undrained conditions with one-way and two-way stress-controlled cyclic loading at $0.1Hz$. The cyclic deviator stress amplitude was kept constant throughout each test. After cyclic loading, all samples were subjected to undrained monotonic triaxial loading.

Comparing the results of tests with the same cyclic deviator stress amplitude, overconsolidated samples reached failure after a *smaller* number of cycles as compared to normally consolidated ones. The shear modulus decreased with increasing number of cycles. The higher the cyclic stress, the more pronounced was the decrease of the modulus. However, at low cyclic stress levels the soil stiffness degradation is negligible so the modulus is independent of the number of cycles. In addition, the number of cycles to failure decreased with increasing the cyclic deviator stress for both normally and overconsolidated samples. For normally consolidated samples, cyclic loading caused an apparent overconsolidation of the soil that was attributed to the reduction in the effective stress during cyclic loading. As such, the effective stress reductions during cyclic loading had the same effect on the response as an actual unloading.

Comparing the undrained shear strength before and after cyclic loading, the strength after cyclic loading has decreased by approximately 25% as long as the cyclic shear strains were less than ±3% after 1000 cycles. In addition, the strength reduction increased with increasing cyclic shear strain and number of cycles. In Figure 2.1 it is evident how the stress paths after cyclic loading move towards the origin. For example, the stress path for a normally consolidated sample reconsolidated to $400\,kN/m^2$ attains an effective stress of $95\,kN/m^2$ at the end of the cyclic event; it is quite similar to the response of a sample with $OCR = 4$. For overconsolidated samples, the cohesion intercept (c') and effective friction angle (ϕ') do not seem to be influenced by cyclic loading.

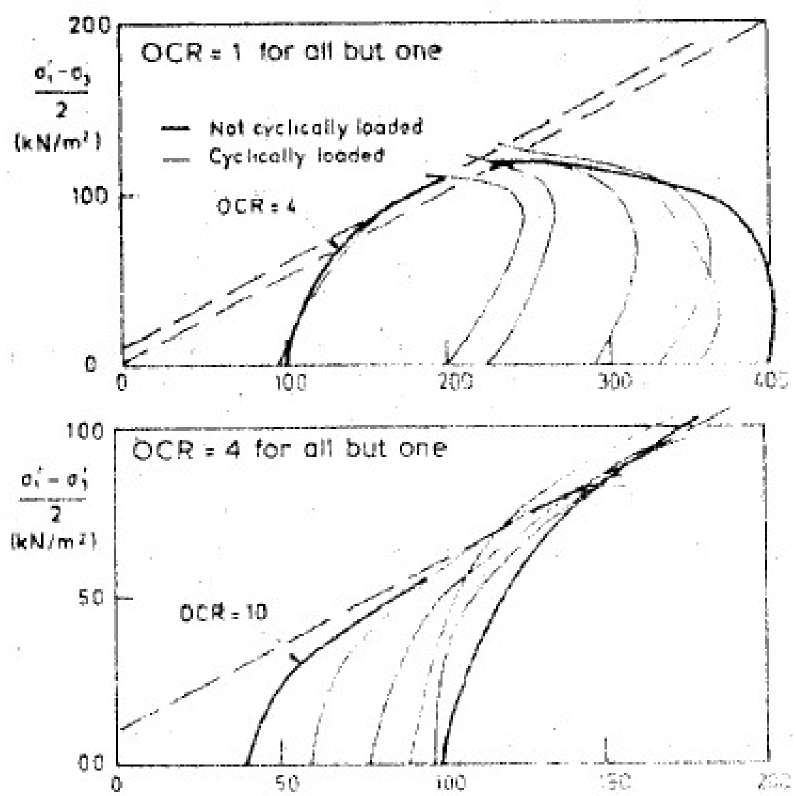


Figure 2.1: Effective stress paths for monotonic loading before and after cyclic loading (after Andersen et al. [1])

If drainage is allowed following cyclic loading, a normally consolidated soil will become stronger and more resistant to further undrained cyclic loading. On the other hand, drainage after cyclic loading in an overconsolidated soil may have a deteriorating effect that will result in lower resistance to further undrained cyclic loading. After large cyclic strain some dilatancy occurs and affects the cyclic pore pressure.

2.2 Ansal and Erken (1989) [2]

Ansal and Erken [2] performed cyclic simple shear tests that measured pore pressure, axial and horizontal deformations for normally consolidated, laboratory prepared clay samples. All samples were prepared under identical conditions using ground kaolinite clay with $G_s = 2.65$, $LL = 65$, and $PL = 25$. The slurry was one-dimensionally consolidated ($\sigma_v' = 100kPa$) and then samples were taken from the clay block. Another set of samples was prepared by remolding the consolidated clay in order to achieve material isotropy. Cyclic tests were carried out at frequencies of $1\,Hz$, $0.5\,Hz$, $0.1\,Hz$, and $0.01\,Hz$.

The pore pressure behavior indicated that there existed a threshold cyclic shear stress below which no excess pore pressure accumulation will occur. It is evident that pore pressure and cyclic shear strain amplitude are interrelated. There is a critical shear stress ratio that can be defined as the critical level of repeated stress below which failure will not occur. This definition was first given by Larew and Leonards [31] as the maximum level of repeated stress that will not lead to failure. Later Sangrey [48], Sangrey et al. [49], France and Sangrey [12], and Sangrey et al. [50] demonstrated the validity of this concept.

Ansal and Erken [2] observed that frequency effects diminish with increasing numbers of cycles and with decreasing shear stress amplitude. The influence of cyclic frequency appears to be significant if a relatively small numbers of cycles are used in evaluating the cyclic behavior, as in the case of earthquakes. Such behavior patterns were observed for both undisturbed as well as remolded samples. Remolded samples appear to be more resistant to cyclic shear stresses. Cyclic shear strain amplitudes developed in these tests are smaller in comparison to shear strains measured in one-dimensionally consolidated samples. The pore pressure accumulations observed for both types of samples are similar, but the remolded samples did show a tendency to yield slightly higher pore pressures.

2.3 Ansal et al. (2001) [3]

Ansal et al. [3] performed undrained stress-controlled, multistage cyclic hollow cylinder (torsional) triaxial tests to evaluate the changes in the stress-strain and shear strength characteristics in terms of threshold cyclic shear stresses and cyclic yield stress.

Tests were conducted on undisturbed samples of Golden Horn clay from Istanbul, Turkey that were were consolidated under K_0 conditions. This soil is classified as a normally consolidated, organic, marine clay (CH/OH).

The maximum shear modulus, G_{max}, as well as the modulus degradation curves for each sample were determined to evaluate the threshold cyclic shear stress level, and the difference between soil samples with different plasticity indices. Then, a series of cyclic simple shear tests at different cyclic shear stress amplitudes was conducted. Some tests were also performed to study the post-cyclic behavior that follows cyclic loading. In these tests the samples were kept under K_0 stress conditions without allowing drainage for $1h$ to ensure stabilization and uniform distribution of pore pressure. The samples were then sheared under undrained strain-controlled conditions. Another group of tests was performed to determine consolidation settlements by allowing the dissipation of pore pressures generated by cyclic shear stresses.

There are *three* possible consecutive stages in the cyclic stress-strain behavior of soils. In the first stage, the soil sample will respond elastically without any significant degradation in its stress-strain and shear strength response. The imposed cyclic stresses are small, so induced cyclic strain amplitudes are negligible.

The elastic threshold is the upper limit for this stage. If the imposed cyclic stress levels are lower than the elastic threshold, the degradation of the dynamic shear strength, as well as the post-cyclic shear strength, will be negligible or, at most, limited. However, once the elastic threshold is exceeded, the soil sample will respond in an elasto-plastic manner. This can be considered as the second stage in the cyclic behavior of soils. During this stage the induced cyclic shear strains would lead to strain softening, particle structure breakdown, pore pressure build-up and a rapid deterioration of stress-strain-shear strength characteristics up to the plastic threshold. After the plastic threshold is exceeded, the soil sample would experience large strain amplitudes due to the accumulated degradation of the dynamic shear modulus. This can be considered as the third stage or plastic flow stage in the cyclic behavior of soils (Figure 2.2).

If a clay soil is in the elastic phase and the induced cyclic shear strain amplitudes are less than the threshold shear strain amplitude, no significant particle structure breakdown occurs and therefore no reduction in post-cyclic shear strength is observed. However, when clay samples are in the elasto-plastic phase, the particle structure breakdown has started. However, since it is still limited, the reduction in the post-cyclic shear strength is also limited. Finally, once the critical shear strain level is exceeded, the clay samples are in a plastic flow phase and the particle structure breakdown, as well as the accumulated pore pressures increase significantly. Consequently, the post-cyclic shear strength reduction rate increases drastically.

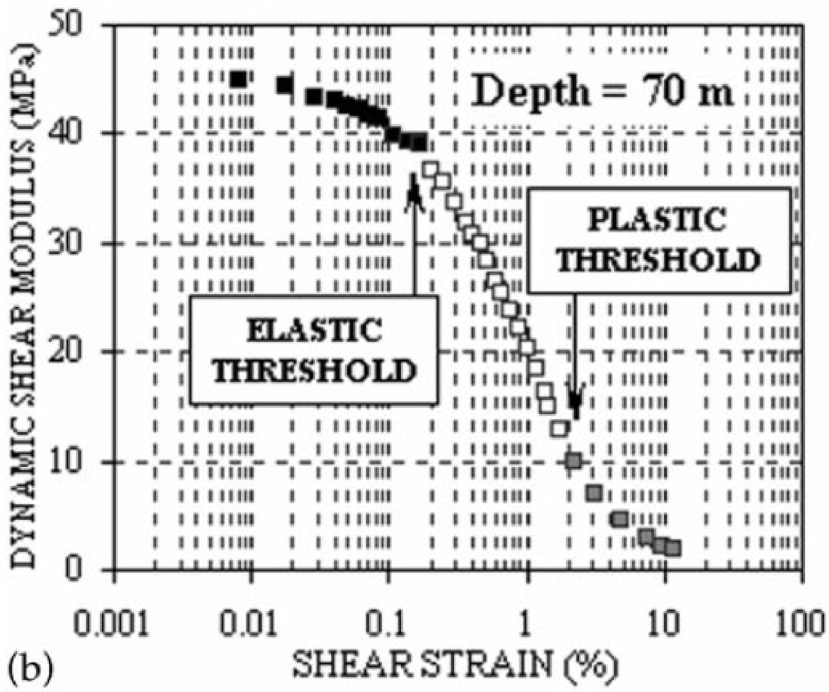


Figure 2.2: Cyclic shear modulus degradation and definition of elastic and plastic thresholds (after Ansal et al. [3]).

2.4 Azzouz et al. (1989) [4]

The testing program undertaken by Azzouz et al. [4] was primarily designed to establish the undrained behavior of normally consolidated to slightly overconsolidated ($OCR = 1.38$ and 2) Boston Blue clay (BBC) samples subjected to uniform two-way symmetric cyclic loading conditions. Stress-controlled cyclic direct simple shear tests with a sinusoidal wave with an amplitude of $\pm\tau_c$ and frequency of $0.1\,Hz$ were performed. Cyclic shear stress ratios (τ_c/C_u) ranging from 0.35 to 0.85 for normally consolidated clay, and cyclic shear stress ratios ranging from 0.56 to 0.85 for overconsolidated samples were applied. Moreover, some samples were taken to failure under monotonic shear stress after the cyclic loading phase.

Undrained cyclic shearing of normally consolidated clays develops positive excess pore pressures and causes a reduction in effective stresses. As a result, an apparent overconsolidation in response to subsequent undrained monotonic shearing was observed. The term "apparent overconsolidation" is used rather than "overconsolidation" because mechanical overconsolidation requires changes in the water content of the clay, whereas undrained cyclic shearing causes a reduction in the effective stresses at constant volume.

2.5 Beroya et al. (2009) [5]

Beroya et al. [5] prepared soil specimens by mixing non-plastic silica silt with commercially available kaolinite, illite and montmorillonite clays in various proportions to obtain different values of plasticity index (PI). Samples were formed by moist tamping using the undercompaction procedure; this was shown to produce samples of fairly uniform density. The specimens were then subjected to a harmonically varying cyclic load using an electromechanical loader. The cyclic load was applied at a frequency of $1\,Hz$. The intensity of the cyclic load was varied in such a way as to produce a wide range of cyclic stress ratios ($CSR = \sigma_c/2\sigma'_{3c}$) and corresponding number of cycles required to cause cyclic failure. The cyclic strength was evaluated as the cyclic stress ratio required to cause failure in 20 cycles (i.e., for $N = 20$).

Results showed that the cyclic behavior of silty clay mixtures is greatly influenced by the predominant clay mineral in the soil. It was demonstrated that given the same amount of clay mineral and/or same value of PI, the montmorillonite soils had the highest cyclic strength, followed by the illite soils, and then by the kaolinite soils. Moreover, the rate of increase in cyclic strength with increasing percent clay mineral and PI is again the highest in the montmorillonitic soil, lowest in the kaolinitic soil, and intermediate in the illitic soil. It was confirmed that the relationships between percent clay fraction, percent clay mineral, and PI with cyclic strength is not unidirectional, and that these parameters cannot be used as reliable indicators of susceptibility to cyclic failure without considering the type of clay minerals involved. There are some aspects of clay mineralogy that the PI can not quite capture (e.g., differences in clay mineral coating, water-orienting ability). According to the results in this study, the use of percent clay fraction and/or PI values as a criterion for determining the cyclic behavior and liquefaction susceptibility of fine-grained soils (or even coarse-grained soils) may need to be reviewed.

2.6 Brown et al. (1975) [6]

Brown et al. [6] performed undrained repeated load triaxial tests on reconstituted samples of Keuper Marl silty clay ($G_s = 2.74$, $PL = 18\%$, $PI = 14\%$). Beginning with slurry form, the soil was consolidated under one-dimesional conditions at $150\,kPa$, and samples with OCRs of 1, 2, 4, 10, and 20 were created. Cyclic loading was applied sinusoidally with a frequency of $10\,Hz$. All tests were carried out under undrained conditions with the excess pore pressure measured at the bottom of the specimen. The pore pressure measurements thus corresponded to the mean values and did not represent the changes that took place during the respective cycles. This limitation inhibited proper interpretation of resilient behavior and arises from the difficulty of measuring cyclic changes in pore pressure in fine-grained soils. Moreover, monotonic loading tests were performed (strain controlled at a rate of0.076 mm/min) in order to compare strength before and after cyclic loading.

In this work the critical cyclic stress level or cyclic threshold where below this limit the soil reach an equilibrium state was not observed. In general, strain was continuing to build up even after 10^6 stress cycles, and an apparent failure condition was only obtained for the highest cyclic stress levels on the overconsolidated samples ($OCR = 2$ 4, 10 and 20). These highest cyclic stress levels were 93%, 94%, 82% of the monotonic undrained shear strength, respectively. Figures 2.3 and 2.4 show the strain accumulation for different cyclic deviator stresses corresponding to samples with OCR =2 and 4, respectively.

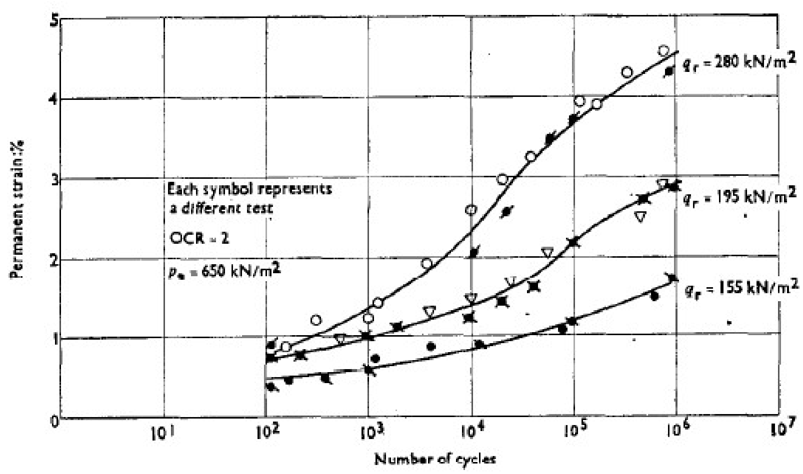


Figure 2.3: Permanent strain during cyclic loading, $OCR = 2$ (after Brown et al. [6]).

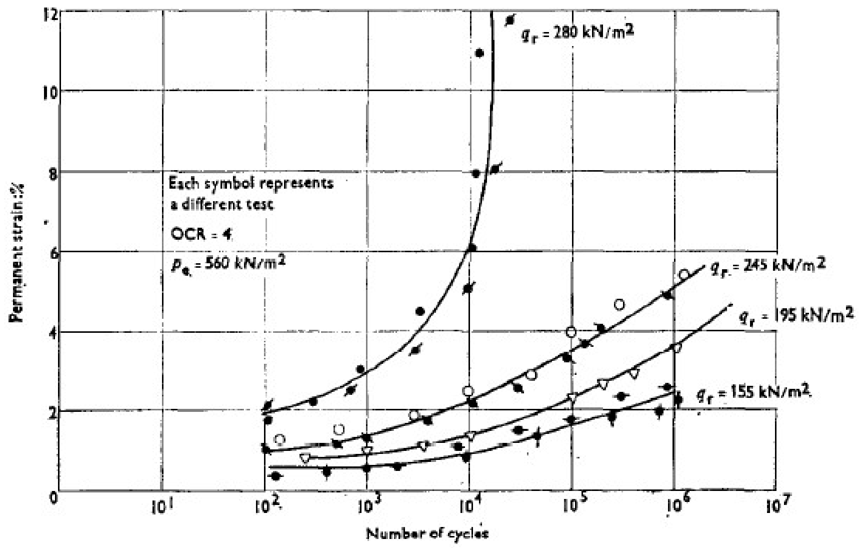


Figure 2.4: Permanent strain during cyclic loading, $OCR = 4$ (after Brown et al. [6]).

2.7 Cao and Law (1992) [7]

Cao and Law [7] performed undrained cyclic triaxial and resonant column tests at small strain amplitudes in order to measure the initial shear modulus for undisturbed normally consolidated and overconsolidated samples of sensitive Champlain Sea clay from the Ottawa River valley region. The cyclic triaxial tests were run at a frequency of $1\,Hz$, with uniform load amplitude and two-way loading. Failure was assumed to have happened when the double amplitude strain reached 10%.

During a deformation process resulting from a dynamic loading, the energy is dissipated through hysteric damping and plastic deformation of the soil. The first is represented by the area under a hysteresis loop. Plastic deformation occurs when permanent deformation is developed during the loading process. Based on this energy concept, a new energy concept was introduced in order to analyze clay soil behavior under cyclic loading. This proposed method plots excess pore pressure and dynamic strength as a function of the dissipation energy that is developed during the dynamic test. This facilitates data analysis and interpretation of soil behavior under cyclic loading. Cao and Law's results showed that a unique relation existed between the dissipation energy and the dynamic soil behavior expressed in terms of dynamic strength, excess pore pressure, shear modulus, and permanent strain. The excess pore pressure generated during a dynamic test can be expressed by two components, namely a residual value and an amplitude of the pore pressure oscillating about the residual value. The residual value is the mean excess pore pressure for a complete load cycle.

2.8 Diaz-Rodriguez (1989) [9]

Diaz-Rodriguez [9] performed cyclic triaxial tests on undisturbed samples from a Mexico City Lacustrine silty clay deposit ($G_s = 2.24$) of very high water content ($\omega = 395\%$) and soft consistency ($LL = 480\%$, $PL = 76\%$) with void ratios from 2 to 12. The average pre consolidation pressure (σ'_p) was $68.6\,kPa$. Samples of the clay were consolidated isotropically under $\sigma'_3 = 60\,kPa$ in a triaxial cell. Then, specimens were then subjected to an additional static axial stress under drained conditions to simulate the in situ shear stress that can exist in a soil element beneath a structure. Anisotropically consolidated samples were subjected to undrained cyclic loading with stress-control at different stress levels. Sinusoidal cycles at $0.5\,Hz$ were applied until 100 cycles were reached. Finally, if samples did not fail due to cyclic loading, undrained stress-controlled post-cyclic monotonic loading was initiated. Drainage was not allowed during any stage of the cyclic and post-cyclic tests.

An equilibrium state was achieved in which the stress-strain curves followed closed hysteresis loops with no additional changes in pore water pressure or strain occurring with load repetition. This state corresponded to the state defined by Larew and Leonards [31], and called by Sangrey at al. [49] "non-failure equilibrium". After 100 cycles of repeated loading, the clay did not accumulate additional plastic strain, implying that it exhibited essentially elastic response. According with the results, if the cyclic stress amplitude is small, the post-cyclic strength will not decrease significantly as compared to the monotonic strength. If, however, the cyclic stress amplitude increases, the reduction in the strength will decrease significantly.

2.9 Erken and Ulker (2007) [11]

Erken and Ulker [11] performed torsional shear tests on hollow specimens of undisturbed and reconstituted (prepared air pluviation) fine-grained soils (silt and silty clay) from different Turkish cities. The PI of Adapazari soils varied between 2 and 33. The Gumudere soil is a low plastic silty clay, having a PI equal to 18. The PI of Izmir soils varied between 5 and 9. The specimens varied between 70 and 100 mm in outer diameter, 30 and 60 mm in inner diameter, and 140 and 200 mm in height. Stress-controlled dynamic tests were performed under different cyclic shear stress ratios (τ_d/σ_c) between 0.14 and 0.23 at a frequency of $0.1\,Hz$. Isotropic consolidation at an effective stress of 100 kPa was used for all samples. Some of the cyclic tests were carried out until failure. Other sets of cyclic tests were followed by monotonic loading until failure. Monotonic torsional loading was applied at a rate of $0.5\,mm/s$, and lasted until the soil specimens reached a shear strain of approximately 10%.

A critical cyclic shear strain level that triggers an excessive increase in shear strain was observed. Cyclic yield shear strain levels of $\gamma_{cr} = \pm 0.75\%$ and $\gamma_{cr} = \pm 0.50\%$ were determined for reconstituted and undisturbed fine-grained specimens, respectively. The difference in these magnitudes was attributed to differences in the soil's microstructure. The number of cycles at γ_{cr} increased with increasing PI of the reconstituted and undisturbed fine-grained soils. It was concluded that as the cohesive forces increase with PI, they tend to decrease the soil's permeability. Thus, cyclic loads can be partly transferred to the water trapped within a skeleton of soil with a high PI. For this reason, plastic clays develop limited shear strain with respect to the low plasticity silts and clays (See Figures 2.5 and 2.6).

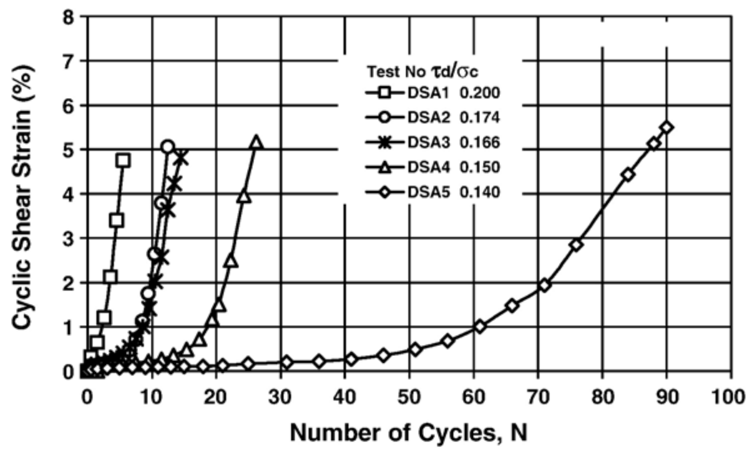


Figure 2.5: Shear strain of the reconstituted soil under different cyclic shear stress, silty soil (after Erken and Ulker [11]).

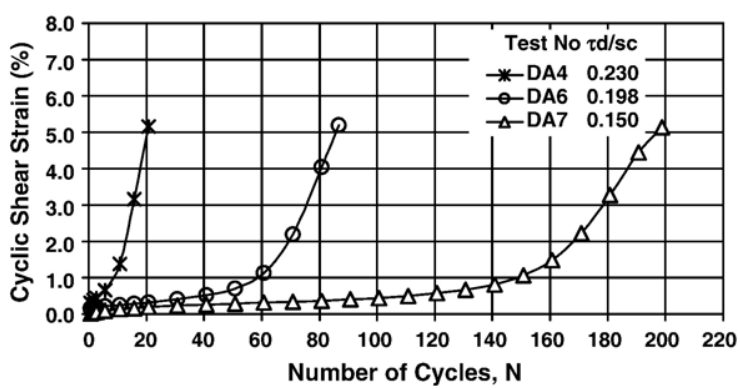


Figure 2.6: Shear strain of the reconstituted soil under different cyclic shear stress, clay soil (after Erken and Ulker [11]).

The monotonic undrained shear strength of reconstituted and undisturbed specimens decreased due to the cyclic shear stress history. This decrease depended on the previous number of loading cycles. However, this reduction was significant when the soil specimen exceeded a certain yield strain level under the same shear stress amplitude prior to monotonic undrained test and reaches nearly 40% in silty soil.

2.10 France and Sangrey (1977) [12]

In this work a descriptive model for the response of clay soils to repeated loading with drainage periods was presented. Undrained cyclic compression tests with stress control were carried out on reconstituted normally and overconsolidated illitic clay. The physical properties of this clay were $LL = 57\%$, $PL = 26\%$, $PI = 31\%$, $\omega = 39.5\%$, and $G_s = 2.78$. The clay was one-dimensionally consolidated under $\sigma_v' = 110\,kPa$. Semi-drained cyclic compression test were performed on isotropically and anisotropically normally consolidated and overconsolidated specimens. Undrained load-unload cycles of constant deviator stress were followed by cycles where drainage was allowed during the unloaded periods for each cycle. Cyclic loading was applied at two or more successively higher levels of deviator stress until either 1) failure occurred or 2) a state of non-failure equilibrium was reached. In unloading cycles the load never was zero; a minimum deviator stress of $4.9kPa$ was always maintained.

For normally consolidated soils that exhibit contractive tendencies, the dissipation of residual positive pore pressures lead to a water content decrease and an increase in undrained strength. As a result, when loaded to failure after the 19^{th} cycle, specimens had a compressive strength higher than its original undrained strength. In other words, if drainage is allowed after the cyclic event, the water content decreases and the undrained shear strength increases. This trend was observed for isotropically and anisotropically normally consolidated specimens. However, for heavily overconsolidated specimens (that exhibit dilative tendencies) with drainage allowed between cycles, residual negative pore pressure was left after the first undrained load-unload cycle and the specimen drew in water to dissipate this pressure. The process was repeated in the second cycle, but a smaller negative residual pore pressure was measured. After the second cycle, the specimen had drawn in sufficient water to reduce its undrained compressive strength below the cyclic deviator stress level. In the third cycle, the specimens collapsed.

2.11 Fujiwara et al. (1985 & 1987) [13; 14]:

Fujiwara et al. [13; 14] performed monotonic and repeated loading odometer tests on remolded samples of Kudamatsu clay ($LL = 65\%$, $PL = 30\%$, $PI = 35\%$, $G_S = 2.648$) and Arike clay ($LL = 115\%$, $PL = 57\%$, $PI = 58\%$, $G_S = 2.65$). Samples $20\,mm$ in height and $60\,mm$ in diameter were tested. The equipment was modified in order to apply loading and unloading cycles, thus simulating the consolidation loading history associated with an oil storage tank. The soil was first consolidated under the load of the tank. The tank was next filled with oil and the soil consolidated under both the weight of the tank and the oil. Finally, the tank is emptied.

Fujiwara et al. [13; 14] simulated this filling and emptying process. First, the sample was consolidated under an overburden pressure, p during 48 hours. Then the sample was consolidated under both static pressure and repeated pressure, Δp, simultaneously for 96 hours. Finally, it was consolidated under the static pressure, $p + \Delta p$ for 48 hours.

Settlement caused by repeated loading is larger than that caused by static loading. The factors influencing the settlement under repeated loading are concluded to be the magnitude of total load, the loading period, the load increment ratio and the initial soil particle structure. In addition, strains caused by repeated loading are larger than those caused by static loading. This fact can be attributed to the dominant value of secondary compression in repeated consolidation. The coefficient of secondary compression calculated under repeated loading is larger than that calculated under static loading.

2.12 Hanna and Javed (2008) [16]

Hanna and Javed [16] performed undrained and partially drained cyclic triaxial tests were performed on normally consolidated sensitive Champlain clay from the City of Rigaud, Quebec, Canada. This clay is classified as being highly plastic (CH), with water content between 44 and 51%, LL = 69%, PL = 25%, liquidity index (LI) in the range between 0.43 and 0.59, $PI = 44$, and $G_s = 2.74$. Prior to cycling, samples were consolidated under a confining pressure equivalent to the preconsolidation pressure of $207\,kPa$. In this investigation, samples were subjected to cyclic deviator stresses equivalent to 33%, 35% or 67% of the deviator stress at failure ($88\,kPa$) from a monotonic triaxial compression test. The cyclic deviator stress was applied at a rate of 4 cycles per hour ($0.001\,Hz$). This rate represents the frequency that wind and wave action impose on tall structures.

In general, compared to samples tested under fully undrained conditions, those tested under drained conditions required a larger number of cycles and higher stress deviator ratio to reach failure. Furthermore, for both undrained and drained tests, the higher the cyclic deviator stresses, the fewer the cycles required to reach failure. For relatively low values of the cyclic deviator stress, the clay may reach a quasi-elastic resilient state without reaching failure. Moreover, the excess pore pressure increases faster and attains higher levels for undrained tests. By contrast, in drained tests the excess pore pressure starts to increase after the 10^{th} cycle.

Cyclic loading on sensitive clay causes remolding that facilitates the dissolution of the bonding agents between particles by the pore water. This leads to the destruction of soil structure and to the loss of shear strength and perhaps liquefaction of the soil. Accordingly, foundations built on this clay would suffer extensive settlement and significant loss of bearing capacity, or perhaps catastrophic failure when loaded under cyclic conditions.

2.13 Hirao and Yasuhara (1991) [17]

Hirao and Yasuhara [17] investigated the effect of preconsolidation periods on the cyclic strength of Reconstituted Ariake clay. This highly plasticity marine clay had the following physical properties: $G_s = 2.58$, $LL = 115\%$, $PI = 75$. Two-way cyclic stress-controlled tests at $0.1\,Hz$ were run at different cyclic stress levels. Failure was considered to have occurred when the double amplitude of cyclic strain reached 5%.

Preconsolidation times of 0, 0.5, 1, 4, 8, 24 hours were allowed during the preconsolidation proccess, which took place before the cyclic loading. After the cyclic loading, a re-consolidation stage was allowed until the complete dissipation of the excess pore pressure had taken place (See Figure 2.7).

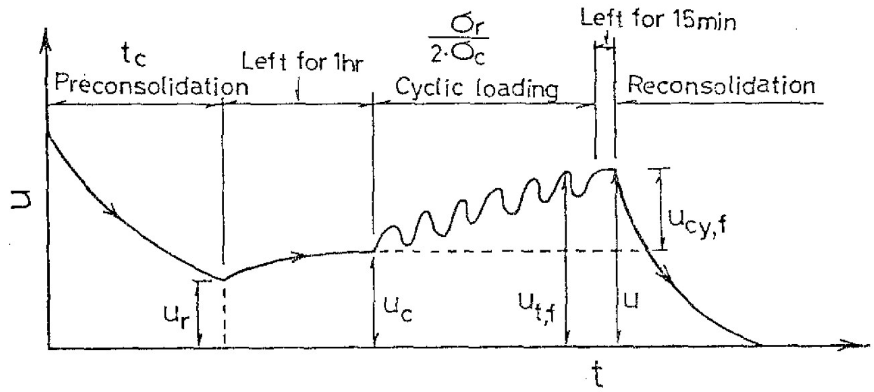


Figure 2.7: Loading scheme used by Hirao and Yasuhara [17].

Cyclic strength of a preconsolidated clay is governed by an increase in effective stress that is developed during the preconsolidation stage. Thus, cyclic strength increases with preconsolidation time. This is explained by the fact that increasing preconsolidation time leads to greater decreases in void ratio. As a result, the sample becomes stronger. Samples consolidated for 8 and 24 hours reached a 100% degree of consolidation (i.e., $U = 100\%$); samples consolidated less than 8 hours did not complete all their primary consolidation.

2.14 Hyde and Ward (1986) [18]

Hyde and Ward [18] performed undrained cyclic triaxial tests in order to study the post-cyclic undrained shear strength of Keuper Marl silty clay ($G_s = 2.80$, $LL = 36\%$, $PL = 19\%$). Samples of the soil were reconstituted in the laboratory under one-dimensional conditions. Normally consolidated and overconsolidated specimens (with $OCR = 1$, 4, 10, and 20) were created. Up to $10,000$ cycles of sinusoidal cyclic stress was applied at a frequency of at $0.1\,Hz$. Cyclic stress ratios (q'/p_e) ranging from 0.13 to 42 were used (here p_e is the equivalent pressure on the isotropic virgin consolidation line that is at the same specific volume as the sample itself). Following completion of the cyclic tests, monotonic triaxial tests were performed. No drainage was allowed between the cyclic and monotonic tests.

Undrained cyclic loading on normally consolidated and lightly overconsolidated soil causes an apparent degreee of overconsolidation, as shown in Figure 2.8. Reduction in strength after cyclic loading is greater for normally consolidated and lightly overconsolidated samples than for heavily overconsolidated ones. Heavily overconsolidated samples did not generate large pore pressures under cyclic loading and did not, therefore, exhibit large changes in post-cyclic monotonic strength.

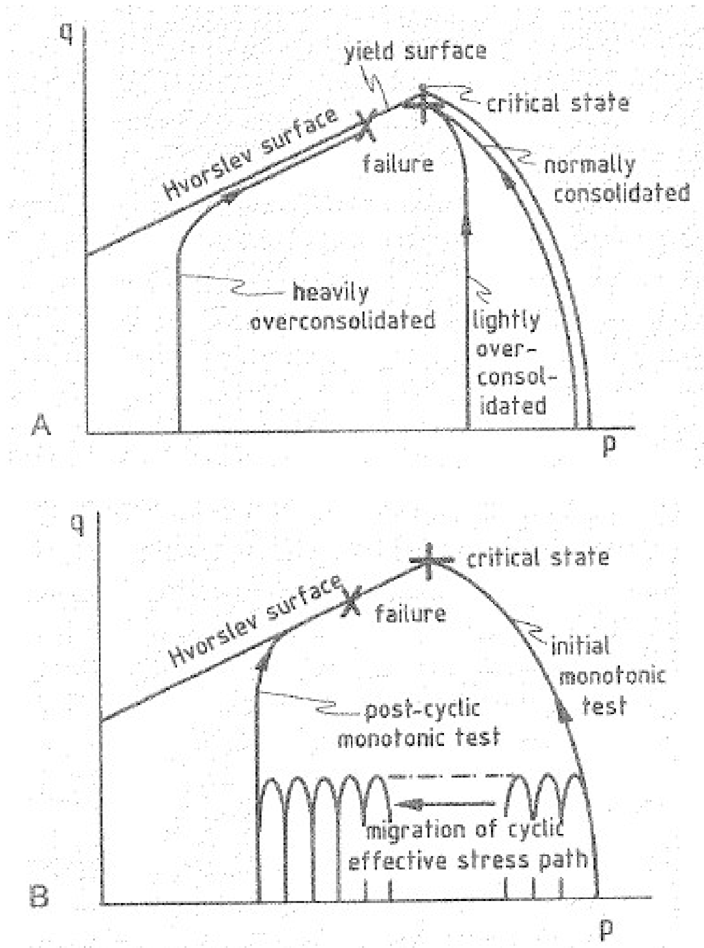


Figure 2.8: Effective stress path for normally consolidated clay, a) No cyclic loading b) Following cyclic loading (after Hyde and Ward [18]).

2.15 Hyde et al. (2007) [19]

Hyde et al. [19] investigated the post-cyclic recompression of samples that occurred following a drained rest period, as well as the results of post-drainage monotonic and cyclic triaxial tests with regard to strength and stiffness. The effect of initial anisotropic consolidation under a static drained deviator stress was investigated. The silty soil tested had the following properties: $G_s = 2.71$, $LL = 24\%$, $PL = 18\%$, and $PI = 6$. Samples were isotropically and anisotropically consolidated under a mean normal effective stress of $p'_c = 100\,kPa$. Several initial sustained deviator stress ratios ($q_s/p'_c = 0$, 0.25, 0.5, and 0.75) were employed. Cyclic loading was applied at various stress levels, with q_{cyc}/p'_c ranging from 0.099 to 0.491, where q_{cyc} is defined as the single amplitude cyclic deviator stress. The frequency of loading used was $0.1\,Hz$. Two-way and one-way cyclic triaxial tests were carried out. Once the failure strain was reached (5%), the cyclic loading was immediately terminated and the samples were allowed to reconsolidate during a rest period in order to simulate a calm period after shaking. Subsequently, the samples were subjected to further monotonic or cyclic triaxial loading under undrained conditions.

The compressive strength of post-drainage samples increased due to reconsolidation during the rest period, particularly for those samples with lower degrees of anisotropy in consolidation. The initial stiffness of isotropically consolidated samples was found to have decreased after post-cyclic recompression; for anisotropically consolidated samples the stiffness increased. For isotropically consolidated samples with post-cyclic recompression, the axial strain began to develop at a much earlier stage in the test due to the lower initial stiffness compared to those without a preloading history. Irrespective of stress reversal or non-reversal conditions, the cyclic strength for the second post-drainage loading increased with increasing magnitude of initial sustained deviator stress.

2.16 Hyodo et al. (1988 & 1992) [20; 21]

The purpose of these two research programs was to develop an analytical model to predict the generation and dissipation of pore pressure and volumetric strain in clay during undrained and partially drained cyclic loading, respectively. Undrained and partially drained cyclic triaxial tests, followed by drainage on reconstituted samples of Ariake marine clay ($G_s = 2.65$, $LL = 123\%$, and $PI = 69$) were performed. Samples were isotropically consolidated under different confining pressures. Then, one-way compression cyclic loading was applied at $0.1\,Hz$. In drained cyclic tests the drainage line was kept open throughout the duration of cyclic loading.

In the undrained cyclic triaxial tests, excess pore pressure built up and shear strain amplitude developed with the number of load cycles. In certain cases, the specimens reached cyclic failure. Under drained cyclic loading conditions, excess pore pressure within a clay specimen is not uniformly distributed since the generation and dissipation of excess pore pressures occurs simultaneously during cycling depends on the distance from the drainage boundary. So, for small cyclic stress amplitudes the excess pore pressure builds up in the first cycles, and then diminishes. The same behavior was observed for the axial strain. However, for large cyclic stress amplitudes there is little dissipation of the excess pore pressure, and the axial strain continuously increases with the number of loading cycles. From these results an analytical model was proposed for evaluating the deformation of clay due to undrained shear and partial drainage under long-term cyclic loading. The proposed model is a useful and practical tool to predict clay behavior under both undrained and partial drained cyclic loading.

2.17 Hyodo et al. (1993 & 1994) [22; 23]

Hyodo et al. [22; 23] investigated the effect of an initial non-zero deviator stress (q_s) on the cyclic response of resedimented Itsukaichi clay. This highly plasticity marine clay had the following properties: $G_s = 2.552$, $LL = 124.2\%$, $PL = 51.4\%$, and $PI = 72.8$. Reconstituted [22] and undisturbed and remoulded [23] samples of Itsukaichi clay were used.

The slurry was reconstituted in the laboratory with a vertical pressure of $50\,kPa$ under one-dimensional conditions. Specimens were initially consolidated isotropically and then consolidated anisotropically under a constant mean principal stress of $p_c = 200\,kPa$. Different values of q_s were employed, ranging from 0 to $240\,kPa$. Reversal and non-reversal cyclic axial load was applied at a frequency of $0.02\,Hz$. Cyclic failure was defined as a cumulative peak axial strain value of $\epsilon_p = 10\%$.

As shown in Figure 2.9, the cyclic shear strength was observed to decrease with increasing initial static deviator stress and with the number of cycles. In addition, both the excess pore pressure and strain are affected by the initial static deviator stress.

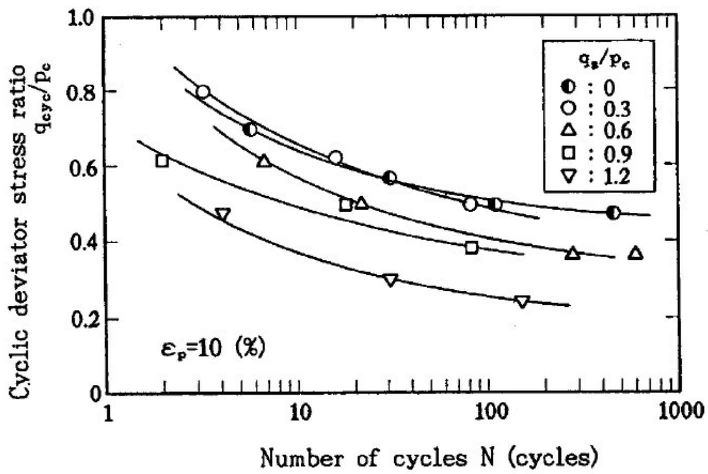


Figure 2.9: Cyclic deviator stress for different q_s/p_c ratios (after Hyodo et al. [22; 23]).

Furthermore, in isotropically consolidated conditions, the cyclic strength for Itsukaichi clay was about two or three times as high as that for Toyoura sand (Figure 2.10a). Cyclic strength for the clay was certainly greater than that for sand. However, Itsukaichi clay was more unstable then Toyoura sand in the non-reversal cyclic stress condition. As shown in Figure 2.10b, the cyclic shear strength decreased with increasing initial static deviator stress. By contrast, the cyclic shear strength for sand increases with increasing initial deviator stress. Thus, it appeared that Itsukaichi clay, subjected to initial static deviator stress, is more unstable than Toyoura sand.

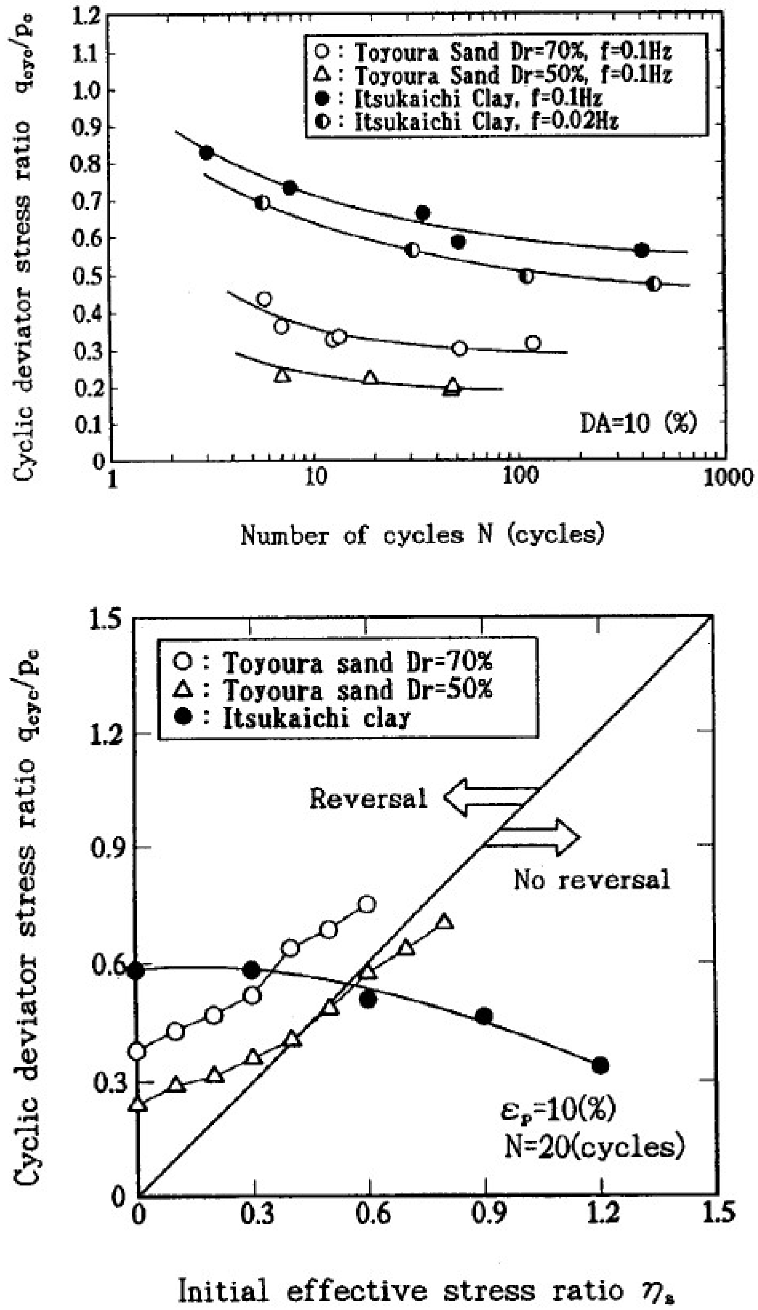


Figure 2.10: a) Cyclic deviator stress ratio for isotropically consolidated condition. b) Cyclic deviator stress ratio vs. initial deviator stress (after Hyodo et al. [22; 23]).

2.18 Hyodo et al. (1999) [24]

In this work the authors not only examined the influence of the initial deviator stress (q_s) in anisotropically consolidated samples, but also the influence of plasticity index (I_p) on the undrained cyclic strength of clays. They found that as the plasticity index increases from non-plastic state to high plasticity clays (i.e., $I_p \approx 70$), the cyclic strength increases as shown in Figure 2.11.

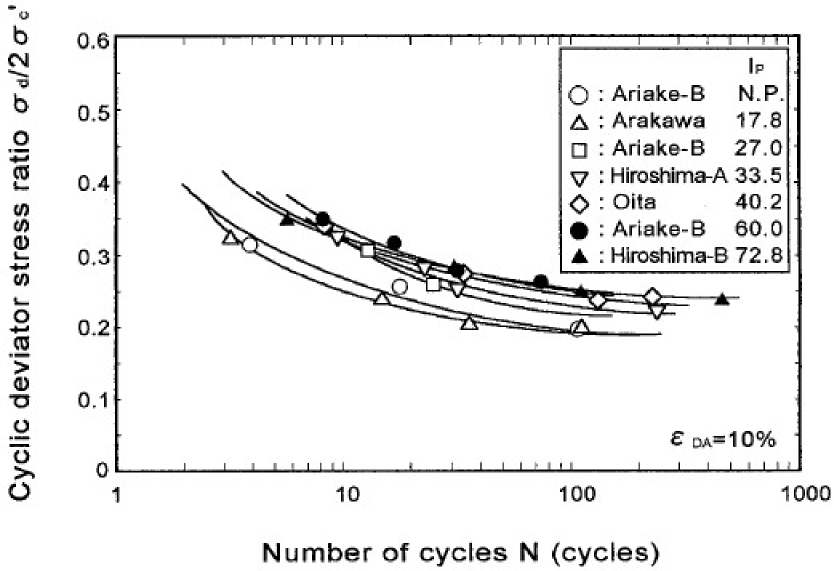


Figure 2.11: Effect of plasticity index on cyclic strength for normally consolidated clays (after Hyodo et al. [24]).

In addition, tests on samples with initial drained shear stress failed as a result of increasing peak axial strain. In the case of low plasticity clay, the increase in initial drained shear stress caused a slight increase in cyclic strength. On the contrary, in the case of high plasticity clay, the cyclic strength decreased markedly as the initial drained shear stress increased. Figure 2.12 shows the cyclic deviator stress ratio required to cause failure after 20 cycles plotted against the initial drained shear stress ratio. It is evident that for clays the cyclic strength defined in terms of the peak axial strain decreases at higher initial drained shear stress ratios, while for sands, liquefaction is more critical under low initial shear stress ratios. Hyodo et al. [24] found that the response of the non-plastic clay is similar to that for sands. It was also observed that the cyclic stiffness of the clays increased with increasing plasticity.

In order to evaluate the the effect of the overconsolidation ratio, some triaxial test were performed on normally consolidated and overconsolidated samples. Based on the results shown in Figure 2.12, the cyclic strength seems to increases with the overconsolidation ratio.

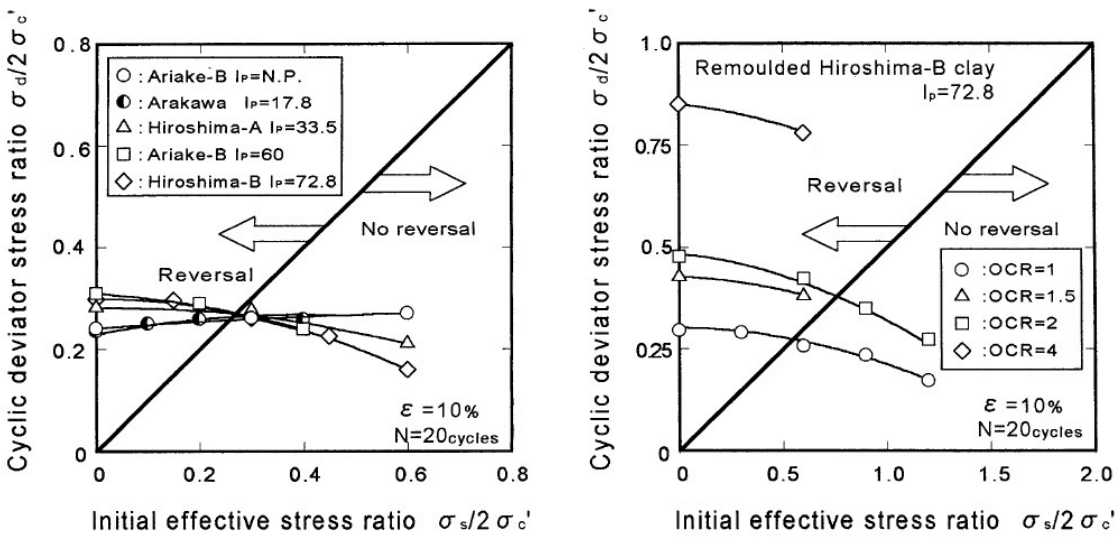


Figure 2.12: Variation of cyclic strength with initial drained shear stress, plasticity, and overconsolidation ratio (after Hyodo et al. [24]).

2.19 Idriss et al. (1978 & 1980) [25; 26]

Idriss et al. [25; 26] developed a new stress-strain degradation model for soft clays subjected to one-dimensional undrained shear cyclic loading. This model was formulated in terms of total stresses and was based on the Ramberg-Osgood backbone degradation method. The model was evaluated using experimental results for San Francisco Bay Mud [25], and Icy Bay marine clay (from Alaska) [26].

Strain-controlled undrained reversed cyclic triaxial tests were performed on undisturbed specimens of San Francisco Bay Mud ($\omega = 89 - 95\%$, $LL = 81\%$, and $PI = 49$). One hundred cycles were applied, with total shear strains of $\gamma = 0.6\%$, 1%, 1.5%, 4%. Changes in pore pressure were not measured during the undrained cyclic tests. After cyclic loading, monotonic undrained triaxial tests were carried out. In addition, cyclic direct simple shear test on normally consolidated Icy Bay marine clay in stress-controlled conditions were carried out. Cyclic loading at frequencies of $1\,Hz$ for earthquake simulations and $0.05\,Hz$ for wave simulations were performed.

Idriss et al. [25; 26] concluded that degradation is strongly dependent on the cyclic strain level applied to the sample. Thus, cyclic loading with large cyclic strain produces a substantial soil degradation. In addition, for the first cycle of loading, the area under the hysteresis loop increases and the loop becomes flatter as the cyclic strain amplitude increases. The unloading and reloading branches of the loops are essentially symmetric for small values of applied strain $\varepsilon_{cyc} \leq 0.75\%$. However, for large cyclic strain amplitudes the loops are not symmetric any more getting the known "S"-shape. Post-cyclic monotonic tests indicated that the modulus E decreases significantly, more than the peak monotonic undrained strength.

There were some tests in which the peak undrained strength was essentially unchanged due to prior cyclic loading.

2.20 Ishihara and Yasuda (1980) [27]

Ishihara and Yasuda [27] performed undrained cyclic triaxial tests on undisturbed samples of two cohesive soils: alluvial clay ($G_s = 2.44 - 2.65$, $LL = 80 - 111$, $PL = 39 - 64$, and $PI = 41 - 47$) and loam from Tama City, Japan ($G_s = 2.57 - 2.69$, $LL = 47 - 100$, $PL = 29 - 60$, $PI = 18 - 40$). Samples were first isotropically consolidated to $\sigma'_c = 100\,kPa$. They were then subjected to an initial static axial stress under drained conditions. Sequences of 30 cycles of constant cyclic load at $1\,Hz$ were applied; the amplitude of each sequence was increased successively until the specimen deformed to a determined failure strain. Monotonic and cyclic strengths were determined by the peak deviator stress corresponding to 15% axial strain.

As shown in Figure 2.13 and Figure 2.14, Ishihara and Yasuda [27] found that the cyclic strength increases with an increase of the initial static deviator stress, σ_s. The cyclic strength increased over the initial shear strength when the initial static stress was approximately half the initial shear strength. However, when the cyclic stress was applied without the initial static stress, the cyclic strength was observed to be below the initial shear strength.

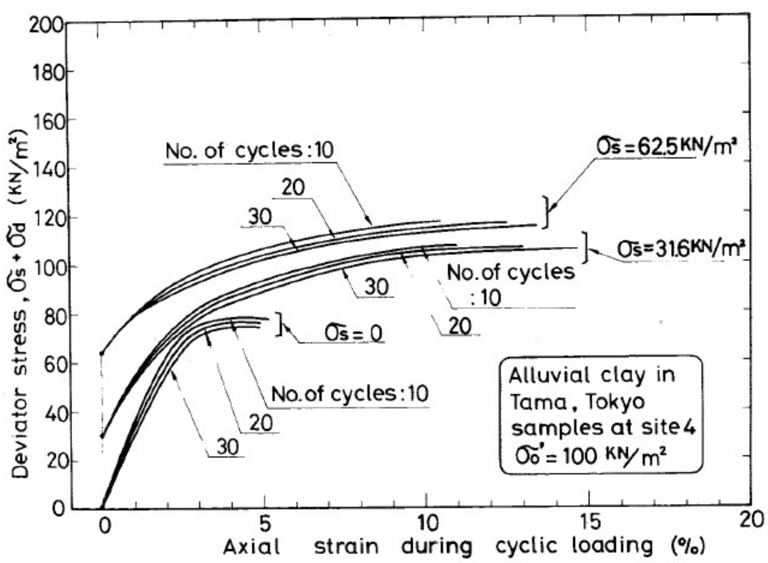


Figure 2.13: Stress strain curves in cyclic loading (after Ishihara and Yasuda [27]).

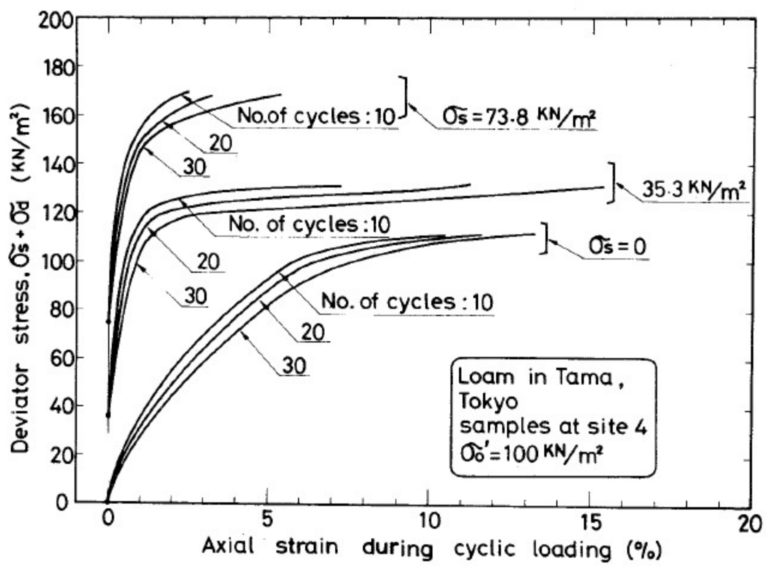


Figure 2.14: Stress strain curves in cyclic loading (after Ishihara and Yasuda [27]).

2.21 Konrad and Wagg (1993) [28]

Konrad and Wagg [28] performed undrained cyclic triaxial tests on reconstituted kaolin clay-silica silt mixtures. Slurry was reconstituted under K_0 conditions with an effective vertical stress $\sigma'_v = 100\,kPa$. Mixes from pure silt to silty clay (40% of kaolinite) were used. Samples were anisotropically consolidated in the triaxial cell, then loaded with one-way (no reversal) cyclic shearing at $0.5\,Hz$. Cyclic stress ratios ($csr = \sigma'_d/\sigma'_c$ where $\sigma'_c = 60\,kPa$) of 0.38, 0.29, and 0.19 were used during the cyclic loading. In addition, in order to check the influence of the strain rate on the cyclic response of clay mixtures, two specimens were tested with an effective stress ratio of 1.67 and loaded at cyclic frequencies of 0.2 and $0.05\,Hz$.

After several loading cycles, the specimens reached a constant rate of deformation termed the average resultant cyclic strain rate. Strain rate effects in cyclic triaxial tests may induce non-uniform pore pressure distributions within a sample and affect the position of the dynamic failure envelope. Clay fraction is also an important factor controlling the behavior of anisotropically consolidated clay-silt mixtures subjected to cyclic triaxial loading without shear stress reversal. For a given cyclic stress ratio and a given static shear stress, the average cyclic strain rate increased with increasing clay content.

2.22 Koutsoftas (1978) [29]

Koutsoftas [29] tested normally consolidated and overconsolidated specimens of two different marine clays: a plastic clay with $PI = 40$ and a silty clay with $PI = 18$. Undrained cyclic triaxial tests were carried out under stress-controlled conditions with two-way (reversal) loading at $1\,Hz$. Cyclic shear stress ratios of $\tau_{cyc}/s_u = 0.75$ for normally consolidated samples and 0.5 for overconsolidated ones were applied. Here $\tau_{cyc} = 1/2(\sigma_1 - \sigma_3)$ and s_u is the undrained shear strength from ICU triaxial test. After cyclic loading, specimens were left under zero shear stress for one day (at constant volume) to allow the excess pore pressure to equalize within the specimen. The specimens were then monotonically sheared to failure under undrained conditions.

Qualitatively, the behavior exhibited by the silty clay after being subjected to cyclic loading was very similar to the behavior of the plastic clay. The undrained shear strength after cyclic loading decreases and the strain at failure increases as the strain at the end of the cyclic loading increases. For double amplitude cyclic strains less than 5%, the loss in undrained shear strength was less than 10% of the initial undrained shear strength. For the plastic clay, the strength loss for overconsolidated specimens was smaller than that of normally consolidated specimens; this was consistent with the smaller excess pore pressure generated in the overconsolidated specimens. Cyclic loading lead to softening of the specimens; the decrease of the undrained modulus (E_u) after cyclic loading was larger than the decrease of strength.

2.23 Kvalstad and Dahlberg (1980) [30]

Kvalstad and Dahlberg [30] performed cyclic triaxial tests on overconsolidated samples ($OCR = 4$) of Drammen clay. These tests were carried out in order to study the effects of anisotropic consolidation, simultaneous variation in vertical and horizontal stresses, and random cyclic loading. Samples were first anisotropically consolidated and then unloaded to isotropic effective stress conditions to produce the desired overconsolidated specimens. Then the random cyclic loading was applied; it consisted of 1250 cycles of storm loading, followed by monotonic loading to failure.

For cases with non-symmetrical loading; i.e., high permanent stress levels, permanent strains developed. This affects the development of pore pressure, as well as cyclic shear strain. Kvalstad and Dahlberg [30] established an upper limit for the maximum shear stress in a soil element beyond which very large and unacceptable strains will occur. For monotonic loading tests that followed cyclic loading, the effective shear strength parameters seem to be unaffected by cyclic loading. The undrained shear strength is, however, reduced. This reduction increases with the intensity and duration of the storm loading.

2.24 Lee and Focht (1976) [35]

The paper by Lee and Focht [35] presented a review of the literature related to the behavior of clays subjected to cyclic loading. Included in this paper was information from Larew and Leonards [31], Seed and Chan [51], Sangrey [49], and Thiers and Seed [57]. Some of the main conclusions are listed below.

- Soft samples are weaker under cyclic loading than stiff samples, this is because every load cycle on a soft sample causes more strain and hence more remolding than for a stiff sample.

- Cyclic loading conditions that produce the longest duration of high sustained load per cycle will lead to the greatest amount of cyclic strength deterioration. Thus, low frequencies and flat top load forms lead to the lowest cyclic strengths.

- On an effective stress ratio basis overconsolidated clays are stronger than normally consolidated clays and normally consolidated clays have cyclic strength similar to saturated sands. However, on a cyclic/static strength ratio basis, overconsolidated clays appear to suffer more rapid cyclic strength deterioration than normally consolidated clays.

- In monotonic loading after cyclic test, the strength decreases and the strain to failure is greater than for the same soil before cyclic loading.

2.25 Lee and Sheo (2007) [34]

The main purpose of Lee and Sheo's [34] work was to propose a stiffness degradation model to quantify the degradation of the secant modulus during cyclic loading. The Ramberg-Osgood equation was used with a proposed cyclic stiffness degradation equation and a proposed equation for evolution of the damping ratio. Undrained cyclic strain-controlled triaxial tests were performed to investigate the non-linear dynamic behavior of Taipei silty clay ($\omega = 41\%$, liquid limit $LL = 38\%$, plastic index $PI = 21$). Sinusosoidally applied cyclic strain at a frequency $0.1\,Hz$ was employed. The first test series consisted of a total of six strain-controlled cyclic triaxial tests; each test consisted of several phases of different consecutive cyclic strains. The amplitude of the cyclic strain was kept constant in each phase, but was varied between loading phases. Each phase consisted of 100 cycles. No failure was allowed between any two phases or any two cycles. In the second series, each cycle had different strain amplitudes. The results of this series were used to investigate the effects of non-uniform cyclic strain on the softening or hardening behavior of the clay.

Lee and Sheo's [34] paper introduced a modified stiffness degradation model and a damping ratio evolution model to describe the cyclic behavior of normally consolidated cohesive soils under both uniform and irregular cyclic loading. According to measured values, the Young's modulus was degraded with increases in the number of cycles. Moreover, the higher the cyclic axial strain amplitude, the higher was the rate of modulus degradation. When a large amplitude cyclic strain is followed by a small one, hardening will occur. In contrast, when a small amplitude cyclic strain is followed by a large one, softening behavior will occur.

2.26 Lefebvre and LeBoeuf (1987) [32]

Lefebvre and LeBoeuf [32] studied rate effects on three undisturbed sensitive clays from eastern Canada. The clays were from lacustrine or marine origin and are characterized by the following index properties: $LL = 32-38\%$, $PL = 22-24\%$, and $PI = 10-14$. Lefebvre and LeBoeuf [32] performed monotonic and cyclic triaxial tests on isotropic and anisotropic ($\sigma_3'/\sigma_1' = 0.55$) specimens. Strain and stress-controlled tests were carried out on structured and restructured samples at a frequency of $0.1\,Hz$. Although cyclic loading histories were applied to the clay samples, the work focused more on rate effects.

For the cyclic tests performed on overconsolidated samples (structured domain), cyclic loading generated very small pore pressures before reaching failure. However, cyclic tests on the normally consolidated samples (destructured domain) indicated a different behavior. Large pore pressures were generated during cyclic loading, bringing the stress path to the static maximum deviator envelope. The stress path then ended in failure above this envelope, very close to the locus defined by the monotonic stress paths at large deformations.

Lefebvre and LeBoeuf [32] also emphasized that when comparing monotonic and cyclic undrained shear strengths, it is important to consider the difference in strain rate. Significant strain rate effects in saturated clays cause the cyclic strength mobilized at high frequencies to be higher than the monotonic strength measured at standard strain rates. As a first approximation, the cyclic strength of a clay at a given frequency could be evaluated from a monotonic test by first applying a correction for the strain rate effect to obtain the monotonic strength at the given frequency, and then applying a degradation function to obtain the cyclic strength that can be mobilized for a given number of cycles.

2.27 Lefebvre and Pfendler (1996) [33]

Lefebvre and Pfendler [33] performed cyclic direct simple shear tests on undisturbed sensitive clay samples from the west of Quebec City, Canada. Index properties for the clay were as follows: $\omega = 55 - 57\%$, $LL = 41\%$, $PL = 21\%$, and $PI = 20$. After sampling, the clay was reconsolidated to the in-situ stress state with an overconsolidation ratio of $OCR = 2.2$. Cyclic stress-controlled tests with a sinusoidal cyclic loading form at a frequency $0.1\,Hz$ under two-way loading conditions (i.e., with loading reversal) were performed. An initial static shear stress (pre-shear) was applied before beginning cyclic loading. This initial stress was applied under undrained conditions; i.e., at constant volume. The presence of an initial static shear stress decreases the cyclic resistance but increases the total undrained shear resistance. This behavior is attributed to the fact that the strain reversal condition partially or totally disappears. Moreover, this initial static shear stress prior to cyclic loading significantly reduces the rate of strength degradation with the number of cycles.

The direct simple shear test program confirmed the significant increase in undrained shear strength with the strain rate as observed in previous studies using triaxial tests. At a cyclic strain rate equivalent to $0.l\,Hz$, the sensitive clay tested in this study can mobilize an undrained shear strength, which is about 40% higher than that determined at a standard strain rate.

2.28 Li et al. (2011) [36]

Li et al. [36] performed undrained cyclic triaxial tests on soft clays, of marine origin, from Wenzhou city, China. The in-situ clay is normally consolidated with the following index properties: $\omega = 67.5\%$, $LL = 63.4\%$, and $PI =$. Samples were consolidated under K_0 conditions, with an average value of $K_0 \approx 0.55$. A cyclic sinusoidal wave loading was applied in the vertical direction, while the cell pressure was kept constant. Only one-way cyclic loading was used in experimental program. To study the effect of loading frequency on cyclic behavior, the frequencies chosen to work with were 1.0, 0.10, and 0.01 Hz. Various test series were then performed. In the first series, samples were cyclically sheared until failure; the cumulative behavior and the effect of loading frequency were studied. In the second series, cyclic undrained tests, followed by strain-controlled monotonic compression tests were carried out, with drainage prevented after the cyclic loading.

In general, for a given number of cycles, larger shear strains and pore pressures were generated at lower frequencies. As shown in Figure 2.15, when the frequency decreases from 1 Hz to 0.01 Hz, the number of cycles required to reach failure decreases drastically. Figure 2.16 shows the accumulative pore pressure corresponding to Figure 2.15.

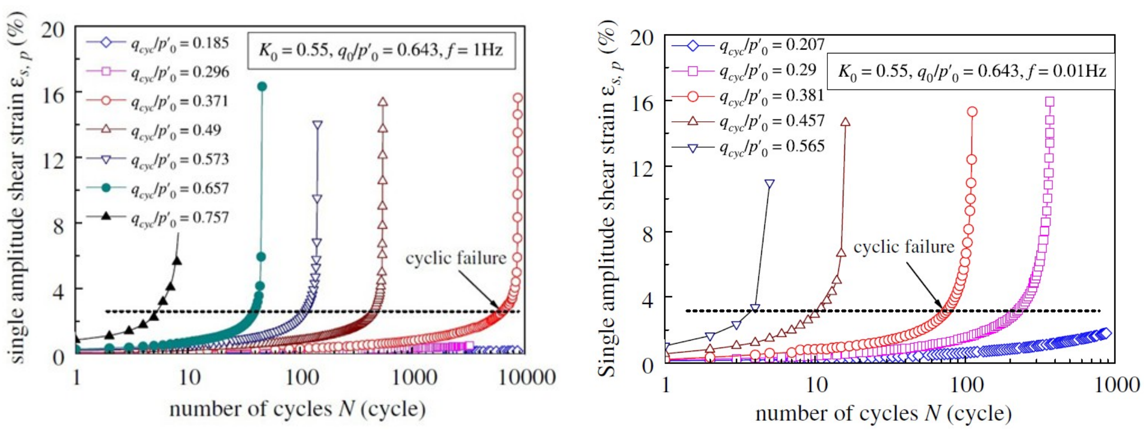


Figure 2.15: Cumulative shear strain and number of cycles for $f = 1\,Hz$ and $f = 0.01\,Hz$, respectively (after Li et al. [36]).

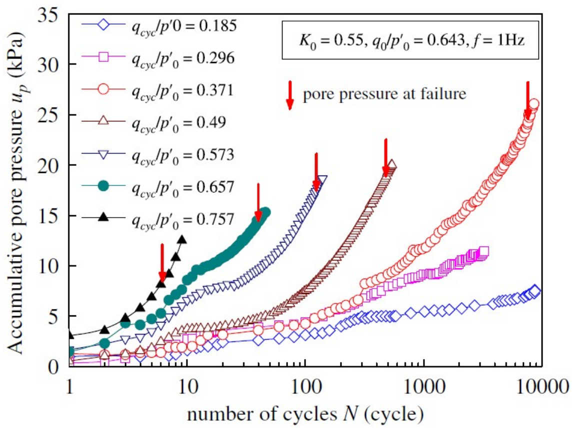


Figure 2.16: Accumulative pore pressure and number of cycles for $f = 1\,Hz$ and $f = 0.01\,Hz$, respectively (after Li et al. [36]).

The reduction of peak strengths was found to depend on the cumulative behavior induced by cyclic loads. The larger the cumulative shear strain and pore pressure induced by cyclic loading, the higher the degree of degradation in post-cyclic strength (Figure 2.17). The deviator stress at the critical state is approximately equal to that obtained from static tests without cyclic loading. The final post-cyclic pore pressures from all the tests are approximately the same and are larger than those measured in the tests without cyclic loading.

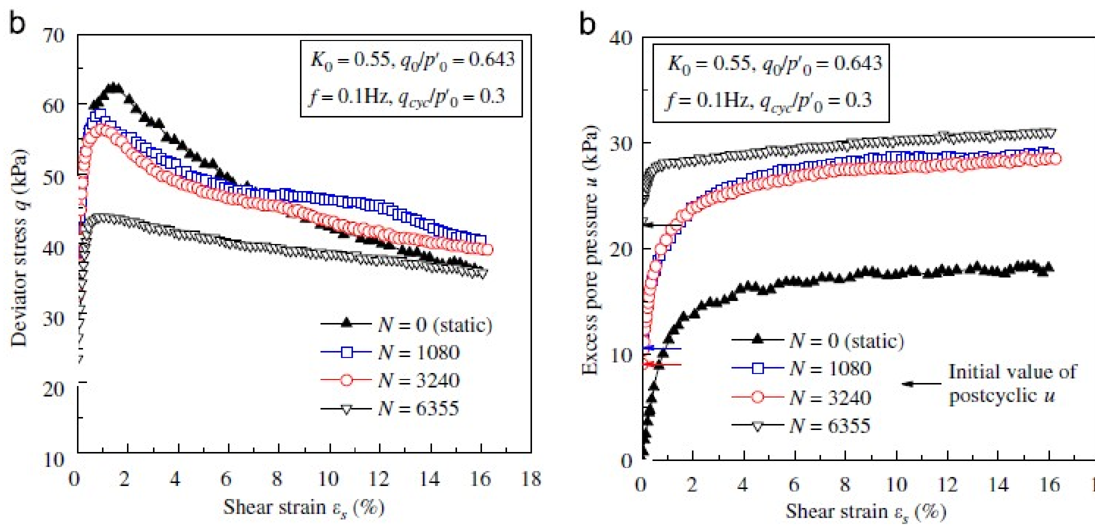


Figure 2.17: Measured stress strain and pore pressure strain relationships of post-cyclic undrained monotonic shear test after N cycles, $f = 0.01\,Hz$ (after Li et al. [36]).

2.29 Matsui et al. (1980) [38]

Matsui et al. [38] performed undrained triaxial cyclic loading tests on saturated normally and overconsolidated (OCR = 1.5 to 4) samples of Senri clay (LL = 93%, PL = 38%, and G_s = 2.68). Compressive and extensional stresses were applied under constant mean total principal stress. Cyclic loading was applied sinusoidally by varying the axial and radial stresses. Due to this condition of loading, the excess pore pressure generated by the cyclic deviator stress component could be mainly constant, since the isotropic stress component does not change under such conditions. Frequencies of $0.02\,Hz$ and $0.5\,Hz$ and cyclic shear stress ratios $\tau_d/\tau_f =$ between 0.40 and 0.65 were employed. Here τ_d is the single amplitude cyclic shear stress and τ_f is the undrained shear strength. After cyclic loading, undrained monotonic tests were carried out. In some tests, the excess pore pressure after cyclic loading was allowed to dissipate, while other tests were run without dissipation of the excess pore pressure.

Effects of factors as the loading frequency, effective confining pressure, cyclic shear stress level and overconsolidation ratio on the excess pore pressure during cyclic loading were explained. Excess pore pressure and the axial strain increase with the number of cycles. As seen from Figure 2.18, for a given number of cycles higher excess pore pressures and axial strains are generated at lower frequencies. Larger strains due to low frequencies could occur mainly from the difference of loading time, due to the viscous behavior of the soil. Where the cyclic shear stress level is high, the excess pore pressure increased more rapidly than in proportion to the number of cycles. When the cyclic shear stress level was low, the excess pore pressure increased gradually with the number of cycles. For overconsolidated specimens, at the beginning of cyclic loading a negative excess pore pressure was measured; subsequently the pressure increased and eventually became positive. The higher the overconsolidation ratio, the greater the negative residual pore pressure after small number of load repetitions and the smaller the positive one after large number of cycles (Figure 2.19).

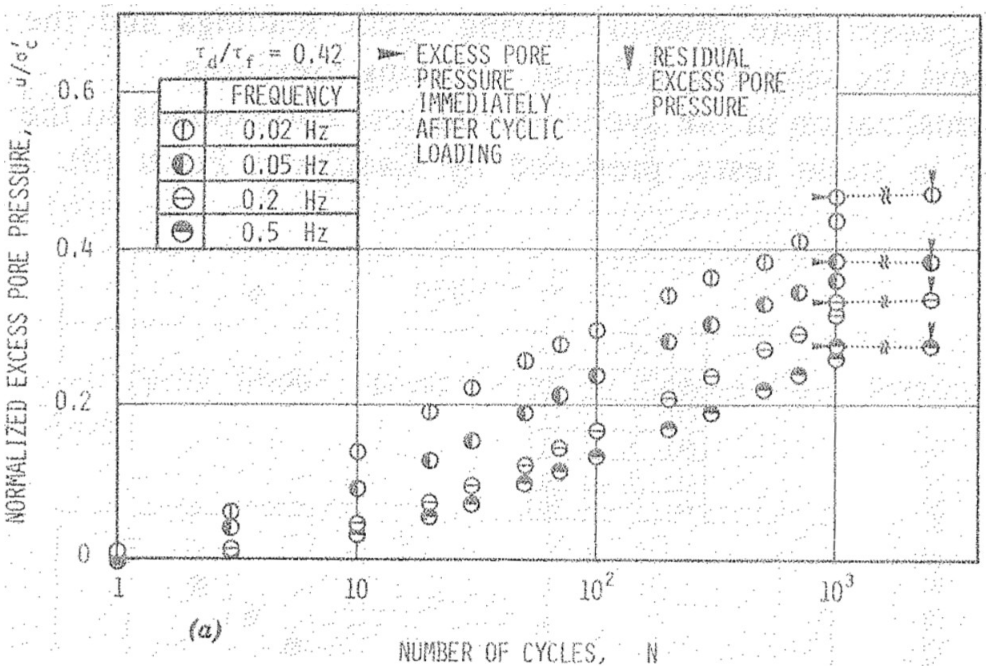


Figure 2.18: Effect of loading frequency on excess pore pressure (after Matsui et al. [38]).

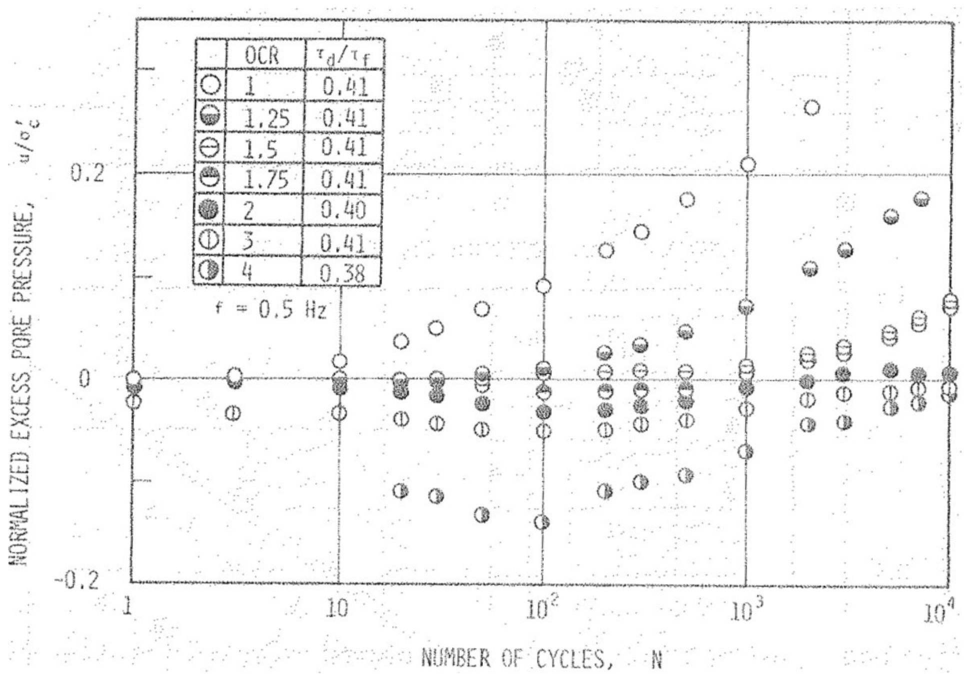


Figure 2.19: Effect of overconsolidation ratio on excess pore pressure (Matsui et al. [38]).

2.30 Matsui et al. (1992) [39]

The work of Matsui et al. [39] focused on the degradation of soil after cyclic loading. Undisturbed specimens of marine Osaka clay (Ma12 or Ma13 clay) and remolded specimens of Crown clay ($\omega = 60\%$, $LL = 99\%$, $PL = 43\%$, and $PI = 56$) were tested under axisymmetric undrained triaxial conditions. Normally consolidated and overconsolidated samples ($OCR = 1$, 1.3 and 4.0) were used. The cyclic strain levels were within the range of small cyclic axial strains that do not produce failure due to cyclic loading (i.e., ε_{cyc} between 0.320 and 1.922). Strain-controlled tests were carried out at $0.5\,Hz$ using a sinusoidal wave pattern and two-way loading. First, 100 cycles were applied to the sample. Then the the pore pressure in the sample was allowed to equalize. Finally, the sample was loaded to failure under monotonic loading.

For normally consolidated specimens, the excess pore pressure generated during cyclic loading caused an apparent overconsolidated state. Thus, the effective stress path corresponding to monotonic loading after cyclic loading looks like the stress path for an overconsolidated specimen (Figures 2.20 and 2.21) for different cyclic axial strain amplitudes. This response is not seen in overconsolidated samples, because there is first negative pore pressure increase at the early part of cyclic loading, followed by positive excess pore pressure depending on cyclic strain level. Consequently there is no big difference in the residual excess pore pressure before and after cyclic loading.

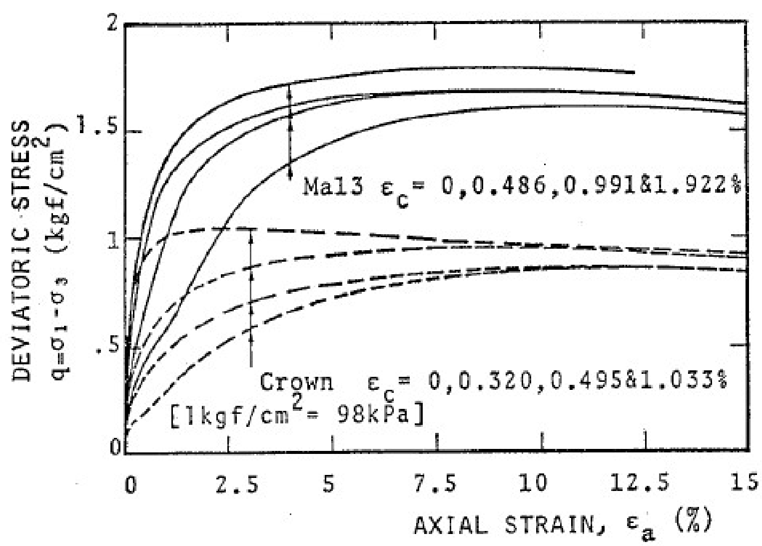


Figure 2.20: Stress strain relation before and after cyclic loading at different cyclic axial strain, Ma13 and Crown clay, $OCR = 1$ (after Matsui et al. [39]).

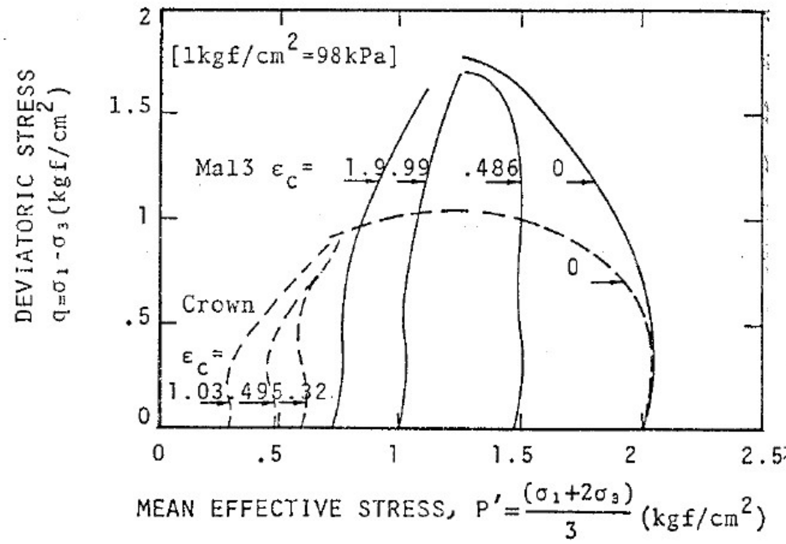


Figure 2.21: Effective stress paths before and after cyclic loading at different cyclic axial strain, Ma13 and Crown clay, $OCR = 1$ (after Matsui et al. [39]).

In terms of strength degradation, for both normally consolidated and overconsolidated specimens the undrained shear strength decreases as the cyclic axial strain increases. In addition, cyclic loading has a significant effect on decreasing the tangent and secant deformation moduli. As a result of shear strength degradation, the strain at failure may increase due to the effect of cyclic loading. When the cyclic axial strain is very small there is no significant effect on the strength degradation (Figure 2.22 and 2.23). The reduction in the undrained strength is not significant, while the reduction in the deformation modulus and the increase of failure strain is.

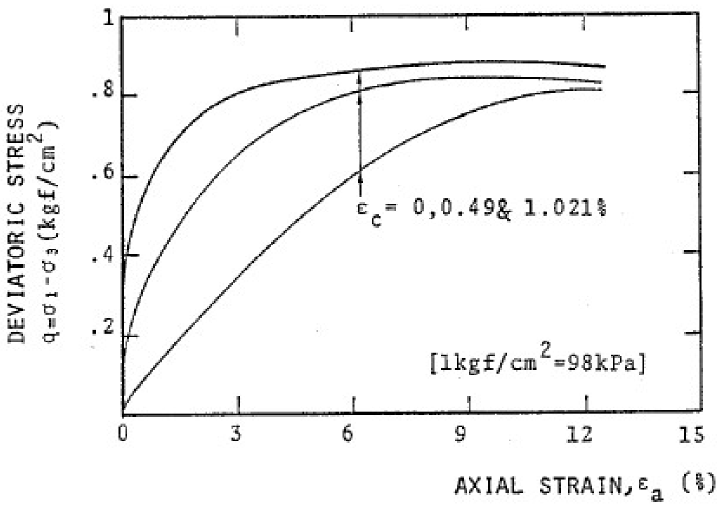


Figure 2.22: Effect of cyclic loading on undrained stress strain behavior of Crown clay, $OCR = 4.0$ (after Matsui et al. [39]).

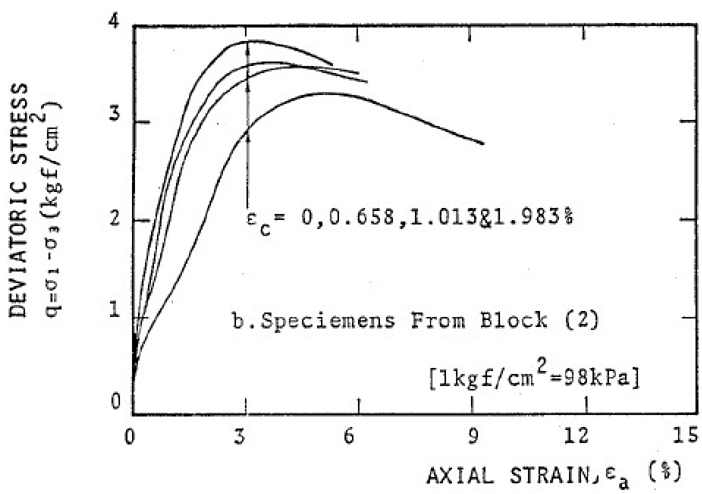


Figure 2.23: Effect of cyclic loading on undrained stress strain behavior of Osaka clay (Ma12), $OCR = 1.3$ (after Matsui et al. [39]).

2.31 Meimon et al. (1980) [40]

Meimon et al. [40] performed undrained cyclic triaxial tests on Kaolinite ($LL = 70\%$ and $PL = 40\%$) and Bentonite ($LL = 105\%$ and $PL = 51\%$) clay soil. Specimens with $OCR = 1$ and $OCR = 4$ were tested. One-way sinusoidal loading with a frequency of $0.1\,Hz$ was employed. An initial deviator stress was applied (q_m), followed by drainage or no drainage before the cyclic loading event. If drainage was allowed, 24 to 48 hours were allowed until the excess pore pressure dissipates; if drainage was not allowed, the sample was left only for one hour in a rest period. Tests were generally stopped at 2000 cycles and the cyclic deviator stress (q_c) was constant during the test.

The reduction in the post-cyclic undrained strength after cyclic loading does not depend on the overconsolidation ratio. In normally consolidated clays, the effective stress path followed after the cyclic loading corresponds to that of an overconsolidated clay, while for overconsolidated ones it does not vary. The main effect of the undrained rest period is a degradation of the initial structure greater than that in a standard monotonic loading triaxial test. The undrained rest period disturbs the initial structure of the clay. For drained tests there is a period of volumetric strain hardening due to the drainage and the sample becomes stronger.

2.32 Moses et al. (2003) [43] & Moses and Rao (2007) [42]

Moses et al. [43] and Moses and Rao [42] performed monotonic and cyclic triaxial tests to study the response to cyclic loading of a cemented marine clay from the East coast of India. The index properties of this clay are $\omega = 80 - 85\%$, $LL = 88\%$, and $PL = 28\%$. Two different testing programs were carried out. In the first, cyclic triaxial tests on isotropically normally consolidated specimens were performed [43]. In the second one, an initial static stress was applied, under undrained conditions, prior to the cyclic loading [42]. Following the application of the sustained static load, a waiting period ensued while the deformation rate had reached a negligible value. Tests were carried out at various cyclic stress ratios, which is expressed as the ratio between the vertical cyclic deviator stress and monotonic undrained strength ($CSR = \tau_{cyc}/s_u$), ranging from 0.20 to 0.57. Tests were conducted under one-way cyclic loading imposed in the vertical direction, with a loading pattern consisting of a half sine wave. The direction of the principal stresses is unchanged during testing, so degradation caused by the continuous change of principal stress direction is not an issue. Loading frequencies of 0.17, 0.083 and 0.05 Hz were chosen for the cyclic tests. Post-cyclic monotonic triaxial tests were conducted to qualitatively evaluate the damage caused by cyclic loading.

According to Moses et al. [43], at low values of cyclic stress ratios, cyclic loading did not have significant influence on deformation and pore water pressure response, irrespective of the number of loading cycles. All these clearly indicated that cementation bonds have a major effect on the endurance limits. The safe cyclic stress level to be used in design should thus consider all of these aspects of cementation, stress history and wave loading frequency. With the increase in number of cycles, the influence of the cyclic stress ratio and frequency of loading become less important. Besides, it can be seen that under post-cyclic monotonic shear loading, the stress-strain curves are so steep and within a very small strain, more than 90% of the ultimate deviatoric stress were reached. Moreover, these steep curves have motivated earlier investigations to suggest that the cyclic load introduced brittleness into the system. Another interesting point to be noted is that there is no significant rise in the excess pore water pressure. However, degradation of the soil compression moduli (E_s) confirms the soil softening.

On the other hand, the same trend was observed on samples that were first loaded with an initial sustained axial load, at low values of cyclic stress ratios. Strain and excess pore pressures reached a stable state irrespective of the number of load cycles. Under the combined action of the initial sustained axial load and subsequent cyclic loading, the soil response is well improved below certain range of sustained stress ratio (SSR). If the sustained stress ratio level is kept lower than this limiting value, it is possible that even at a high cyclic loading, the soil becomes stronger. The initial sustained stress ratio controls the failure in the soil. Finally, the reduction in post-cyclic strength became higher with increased cyclic straining. That is, the higher the cyclic stress ratio, the greater the post-cyclic strength degradation.

2.33 Ohara and Matsuda (1988) [44]

Ohara and Matsuda [44] performed cyclic direct simple shear tests under undrained conditions on normally and overconsolidated specimens ($OCR = 2,\ 4,\ 6$) of Kaolinite clay ($G_s = 2.7$, $LL = 53.8\%$, and $PL = 28.5\%$). Two-way, strain-controlled cyclic loading with a sinusoidal wave form and frequency of $0.5\,Hz$ was used. After the cyclic loading, the excess pore pressure was allowed to dissipate in order to measure the volume change (settlement) built up due to cyclic loading.

In normally consolidated specimens positive excess pore pressure was measured due to cyclic loading. This positive increment started in the first cycle of cyclic loading.

In overconsolidated specimens, there was, however, a negative excess pore pressure in the initial cycles. The excess pore pressure then began to be positive. Excess pore pressure generated during cyclic shear was found to depend on the amplitude of cyclic shear strain, the number of cycles, and on the overconsolidation ratio. The volumetric strain or settlement depends on the excess pore pressure accumulated during cyclic shear and on the overconsolidation ratio. From Figure 2.24 it is evident that the higher the overconsolidation ratio, the lower the volumetric strain that will result compared at the same shear strain amplitude.

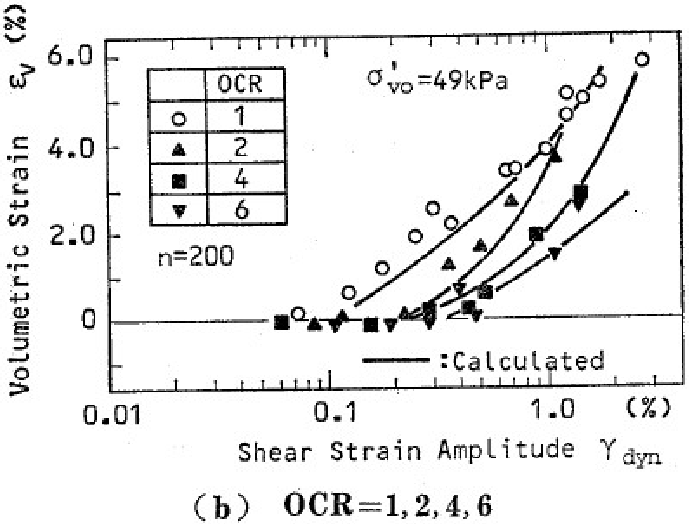


Figure 2.24: Volumetric strain induced by cyclic shear strain (after Ohara and Matsuda [44]).

2.34 Okur and Ansal (2007) [45]

Okur and Ansal [45] performed stress-controlled cyclic triaxial tests on normally consolidated and slightly overconsolidated samples.

The first part of the experimental program consisted of stress-controlled cyclic triaxial tests under different stress amplitudes that were performed to estimate the modulus reduction and thresholds between nonlinear elastic, elasto-plastic and viscoplastic behavior. The second part of the experimental program consisted of an investigation of the undrained stress-strain behavior of fine-grained soils subjected to irregular cyclic loadings. The soil samples used in this research ($\omega = 25 - 52\%$, $LL = 38 - 70\%$, $PI = 9 - 40$) were taken from various sites in Turkey after the 1999 Kocaeli Earthquake, as a part of post-earthquake investigation. Samples were isotropically consolidated in the triaxial cell, the two-way axial cyclic load was then applied to the sample under undrained conditions with frequency of $0.5\,Hz$. Samples were cyclically loaded step-by-step with gradually increasing stress amplitudes covering a wide strain range of $10^{-3} - 10^{1}\%$ under a constant confining pressure. Failure was defined as 10% of the double-strain amplitude.

The results confirmed that the plasticity index (PI) is one of the governing factors in dynamic shear modulus degradation. The strain-dependent change of dynamic shear modulus ratio, G/G_{max} and damping ratio, D, were strongly related to the plasticity index. It was also observed that there is a lower limit below which cyclic degradation will be negligible and that this lower threshold is dependent on the pore pressure build-up. The influence of PI also showed that the value of this threshold generally increases with PI. The modulus degradation observed was due to the rapidly increasing levels of cyclic strain that lead to particle structure breakdown and related softening. Below the elastic threshold, however, the engineering properties of soil remain practically unaffected. The reduction of stiffness starts to become important once the elastic threshold is exceeded. It is also possible to define a second critical strain level defined as flow threshold, where the soil sample has reached the steady state conditions and starts to behave as viscoplastic material. Thus, as shown in Figure 2.25, as the number of cycles increases, so does the cyclic strain. As a result, the stiffness degrades.

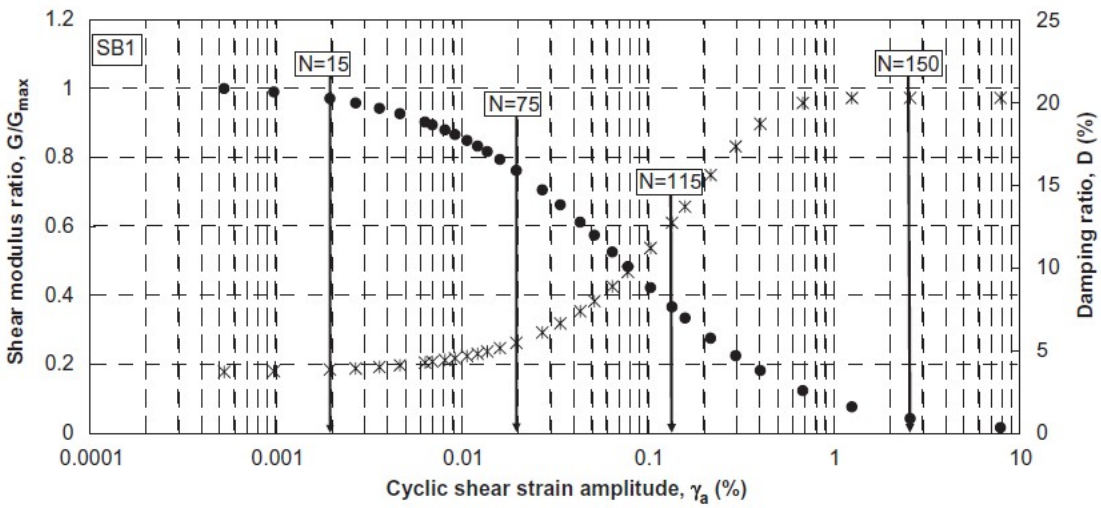


Fig. 10. Variation of shear modulus and damping ratio for different cycles.

Figure 2.25: Variation of the shear modulus and damping ratio for different cycles (after Okur and Ansal [45]).

2.35 O'Reilly et al. (1991) [46]

O'Reilly et al. [46] performed a series of undrained cyclic triaxial tests in order to study the response of a normally consolidated silty clay subjected to cyclic loading. Reconstituted (one-dimensional consolidation) Keuper Marl silty clay was used ($\omega = 32\%$ and $PI = 17$). Specimens were subjected to K_0 consolidation representing the deposition history of a soil element in situ. The subsequent loading regime and drainage periods were designed to simulate a period of reversing (i.e., two-way) undrained cyclic loading at $0.1\,Hz$, followed by periods of drainage under conditions simulating those experienced in the ground. Three amplitudes of cyclic deviator stress were defined as percentages of the undrained strength during monotonic loading (i.e., 30%, 50%, and 70%). After anisotropic consolidation, each specimen was subjected to a series of five periods of undrained cyclic loading up to six hours (2,160 cycles). Each loading period was separated by drained rest periods that lasted between 18 and 66 hours, allowing at least 90% of consolidation to take place. Four distinct test types were performed. The first three consisted of cyclic loading at a constant cyclic stress amplitude (either 30%, 50%, or 70% stress levels). The fourth, which is herein termed a mixed loading test, involved blocks of deviator stress at increasing amplitudes followed by decreasing amplitudes.

The results from these tests showed that anisotropically normally consolidated silty clay, when subjected to undrained cyclic loading, develops pore pressures that move the effective stress state towards failure. The number of cycles required to reach failure decreased with increasing cyclic deviator stress level. The inclusion of drainage periods produced a strengthening effect. During subsequent loading, more cycles at a given stress level were required to develop the same shear strain or pore pressure as that prior to the drainage period. The soil became stronger due to the consolidation periods.

2.36 Procter and Khaffaf (1984) [47]

Procter and Khaffaf [47] performed stress-controlled cyclic triaxial tests on saturated remolded samples of Derwent clay ($PI = 26$). The maximum cyclic shear stress was applied as a proportion (from $\pm 10\%$ to $\pm 60\%$) of the monotonic undrained shear strength. For displacement controlled tests, specimens were subjected to symmetrical sinusoidal cyclic axial strain from $\pm 1\%$ to $\pm 5\%$ of the initial sample height and the deviator loads that were required to achieve these displacements were measured. Frequencies about 0.008, 0.05, 0.2, and $1\,Hz$ were employed.

As seen form Figure 2.26, for the same number of cycles, samples tested at high frequencies exhibited higher cyclic strength. At low test frequencies there a creep strain was measured and a softening process took place. There was clear evidence that the cyclic strength depends upon the test frequency. According to Procter and Khaffaf [47], there is a weakened state that is limited by a “weak” stress; this “weak” stress represents a minimum stress level below which cyclic loading is assumed to have a negligible weakening effect on the soil.

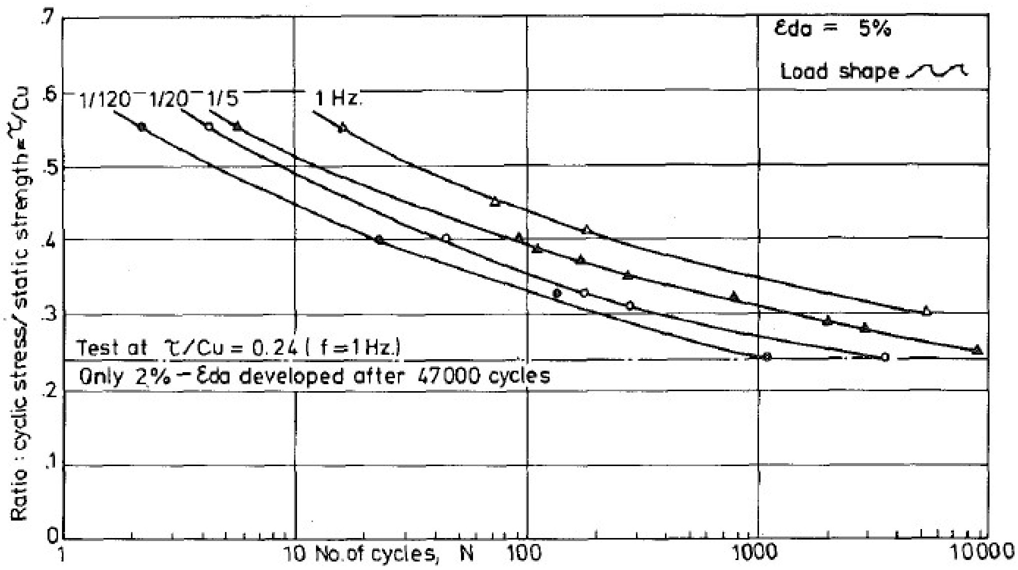


FIG. 3.—Frequency Response of Cyclic Stress Ratio Causing 5% Double Amplitude Strain (Reversed Load-Controlled Tests)

Figure 2.26: Cyclic stress ratio causing 5% double amplitude strain versus the number of cycles (Procter and Khaffaf [47]).

2.37 Sangrey (1968) [48] & Sangrey et al. (1969) [49]

The work of Sangrey and co-workers represents some of the earliest studies of the effects of cyclic loading on cohesive soils. Tests were performed on specimens of clay ($PI = 10$) from the face on a landslide scarp in Newfield, New York. The tests were carried out at 0.0002% per minute in order to ensure adequate stabilization of excess pore pressure. There were continuous loading and unloading cycles until either the sample failed or a state of equilibrium was reached. The cycles were applied at constant stress ($\sigma_1 - \sigma_3$). Different sets of tests were performed. In the first set, samples were normally consolidated under isotropic consolidation stress. In the second set, samples were normally consolidated under anisotropic consolidation stress. Finally, overconsolidated samples were tested. Before cyclic loading a simple load test was carried out until failure. The maximum shear stress ($\sigma_1 - \sigma_3$) was $380kPa(54psi)$.

It was observed that there exists a critical stress level beneath which the soil behaves elastically upon repeated loading. For isotropic normally consolidated clay, the critical stress level was about 2/3 of the maximum undrained strength as determined in a monotonic loading test. The response of anisotropically overconsolidated samples was similar to the normally consolidated ones. Thus, for any particular consolidation history, there exists a critical stress level. In Figures 2.27 and 2.28 are shown the results for deviatoric cyclic stress levels above and below the critical stress level, respectively. From Figure 2.28, where the cyclic deviator stress was around 50% of the initial undrained shear strength, it is evident that the sample reached an equilibrium state that is displayed by a closed hysteresis loop. There is no more plastic strain generated; the excess pore pressure increases during loading but decreases during unloading. The sample did not fail, although as many as one-hundred cycles were applied. The findings were in agreement with the conclusions reached by Larew and Leonards [31] about the existence of a critical level of repeated stress.

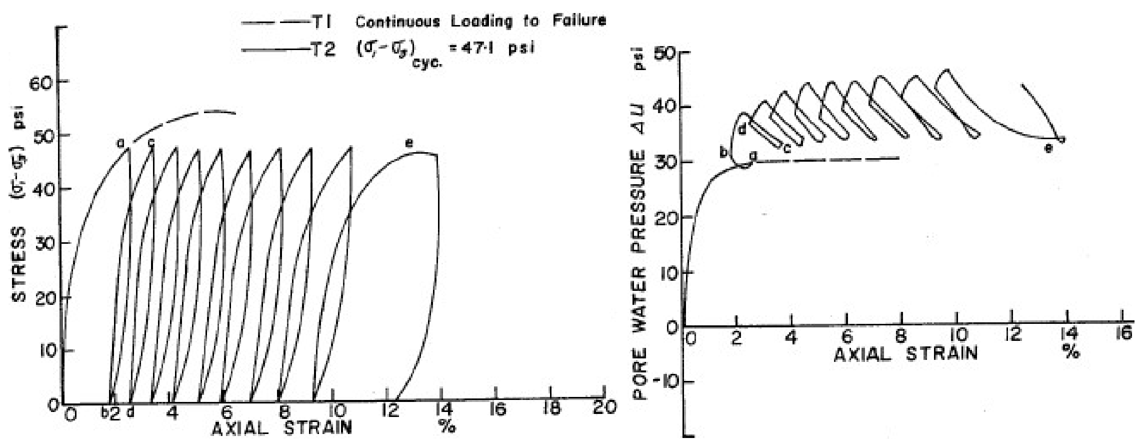


Figure 2.27: Stress-strain curve and pore pressure evolution, $\sigma_1 - \sigma_3 = 330kPa$.

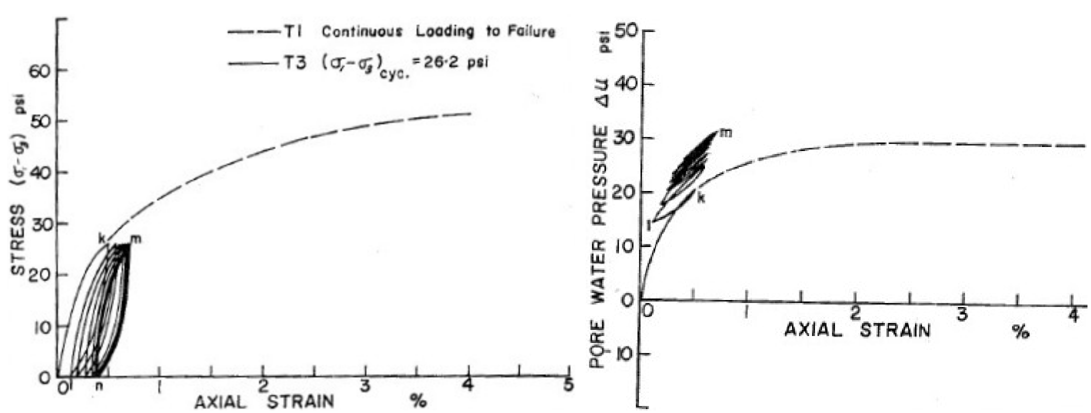


Figure 2.28: Stress-strain curve and pore pressure evolution, $\sigma_1 - \sigma_3 = 180kPa$.

Sangrey and co-workers also found that for soils that are initially at the same state of stress but at different degrees of overconsolidation, the critical cyclic stress level increases with the degree of overconsolidation. The magnitude of the pore pressure build up is dependent upon the consolidation history and the level of repeated stress. The results of samples that had been normally consolidated under an anisotropic stress and then overconsolidated were similar to those described for the samples normally consolidated under isotropic stress.

For all tests, there was a critical level of repeated stress, and this value depended on the consolidation history.

2.38 Seed and Chan (1966) [51]

This study was conducted to determine the effects of various combinations of sustained and pulsating stress on the strength and deformation characteristics of clays. Under earthquake loading conditions, an initially stressed soil element is subjected to a series of stress pulses, and this is what Seed and Chan [51] wanted to represent in this work. The following soils were tested as part of this study: San Francisco Bay mud, $PL = 43\%$ and $PI = 45$; compacted silty clay from Vicksburg, MS, $PL = 23\%$ and $PI = 14$; and compacted sandy clay from Pittsburg, CA, $PL = 19\%$ and $PI = 16$. Unidirectional and bi-directional loading conditions with a frequency of two pulses per second were employed. An experimental program including different magnitudes of the sustained and pulsating stresses, various number of stress pulses applied, and the form of the pulses wave, were carried out combining all of these factors.

First, the strength of the soil was determined by conventional monotonic undrained loading. Then, samples were loaded in the same manner to a stress level expressed as some proportion (e.g., 67%) of the normal undrained strength and allowed to come to equilibrium. At this stage, a series of 100 transient stress pulses were applied. Following the pulsations, the original sustained stress was left on the sample for a period of time until deformations ceased. The sample then was loaded to failure in a normal fashion. In other series of tests, several samples were first subjected to pulsating stresses only, and no sustained stress was applied. Some samples were able to sustain these stresses for different periods of time, but ultimately each sample failed. Thus, the number of pulses causing failure was different in most of the cases. Another series of tests, where the samples were subjected to sustained stresses of various constant values (i.e., 47, 70, and 81.5% of the monotonic strength), was also carried out. Then, different levels of pulsating stresses were superimposed. Sample pulsating stress levels used were 115%, 60%, and 40% of the monotonic strength; these induced failure after 1, 9, and 88 stress applications, respectively.

There was a significant difference between test data obtained with bi-directional loading as compared with unidirectional loading. For conditions involving the same number of stress pulses, for bi-directional stress pulses failure occurred at a stress level equal to 65%, as compared with a stress level of 82% for unidirectional pulses. In addition, Seed and Chan [51] found that the form of stress pulse influenced the behavior of the clay soil. The longer period under maximum stress for a flat peaked pulse causes larger deformations and induces failure in a smaller number of stress applications than in comparable tests using triangular pulses. Excess pore pressure measurements were not made in most of the tests because of the difficulty to achieve an equalization of the pore pressure inside the sample during the loading period. However, some tests were run slow enough to allow this equalization to occur, so the excess pore pressure measurements represented average values for the entire sample. In general, pore pressure built up progressively during the period of application of the stress pulses, increasing with the number of pulses.

In summary, Seed and Chan [51] found that clay soil behavior depends on the nature of the loading conditions, uni- or bi-directional loading, the soil type, the frequency and duration of the pulsating stresses, the number of stress pulses, and the form of the stress pulse. The form of the stress pulse can have a marked effect on the number of stress pulses required to cause failure at certain stress levels.

2.39 Sugiyama et al. (1996) [53]

This paper is written in Chinese. Undrained cyclic triaxial compression tests were performed on marine clays. Tests were carried out on isotropically and anisotropically normally and overconsolidated specimens. Tests were performed under various combinations of initial static stresses and subsequent cyclic stresses. The cyclic shear strength of anisotropically consolidated clays decreased with increasing initial static deviator stress. The response was independent of the overconsolidation ratio.

2.40 Takahashi et al. (1980) [54]

Due to the fact that reliable measurement of pore pressures in clays during cyclic loading are difficult to obtain, Takahashi et al. [54] performed tests at a low frequency ($f = 0.001\,Hz$) and used a fast response time piezometer probe to measure the excess pore pressure. Pore pressures were obtained with special care so the effective stress changes during cyclic loading in clays were properly studied. Cyclic compression-extension (two-way), cyclic compression (one-way), and monotonic extension tests were carried out on Lower Cromer Till silty clay under stress-controlled conditions. The monotonic tests were either stress or strain-controlled. Isotropically normally and overconsolidated samples were used, with $OCR = 1$, 4, and 7.

In normally consolidated samples subjected to cyclic loading, the effective stress path migrates toward the origin in stress space. This is because the pore pressures generated during loading are different in magnitude from those generated during unloading. This difference is greater in the half cycle of compression than in the half cycle of extension, so there is no symmetry in the effective stress cycles. Consequently, as shown in Figure 2.29, loops develop in the half cycle on the extension side. The extension cycle leads to the failure envelope and, correspondingly, large strains occur first in extension. On the contrary, in overconsolidated samples, although the effective stress path initially goes away from the origin in stress space, it subsequently reverses as shown Figure 2.30.

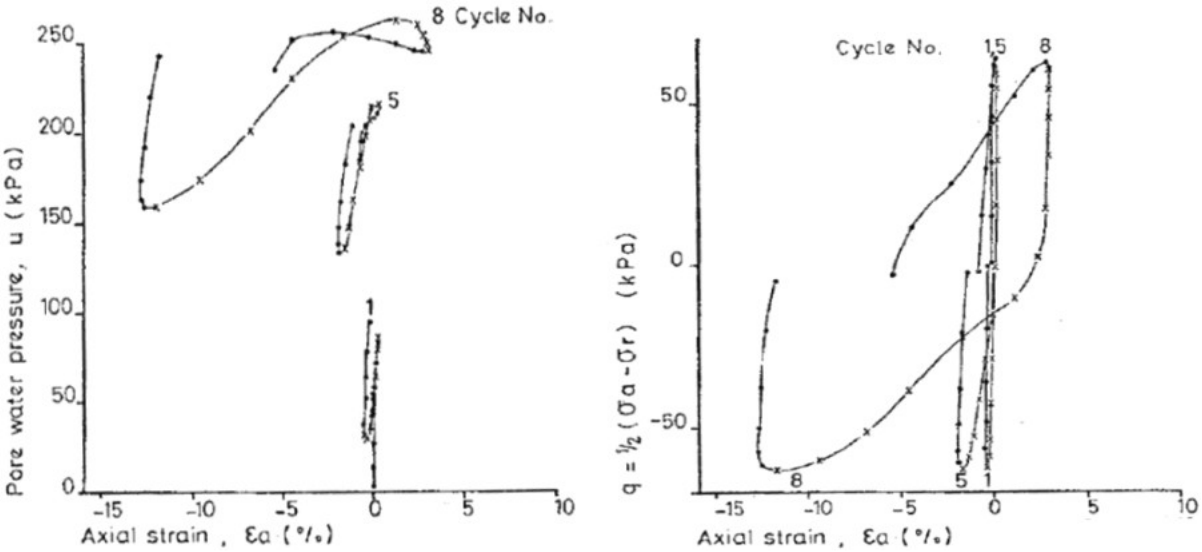


Figure 2.29: Stress-strain and pore pressure-strain curves for cycles 1, 5, and 8 (after Takahashi et al. [54]).

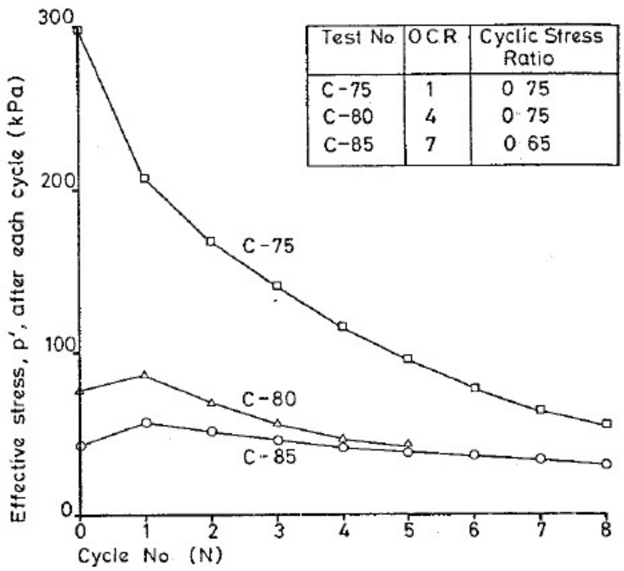


Figure 2.30: Effective stress, p', after each cycle (after Takahashi et al. [54]).

Furthermore, it was noted that while axial strain development accelerates with the number of cycles, pore pressure generation reduces, as shown in Figure 2.31.

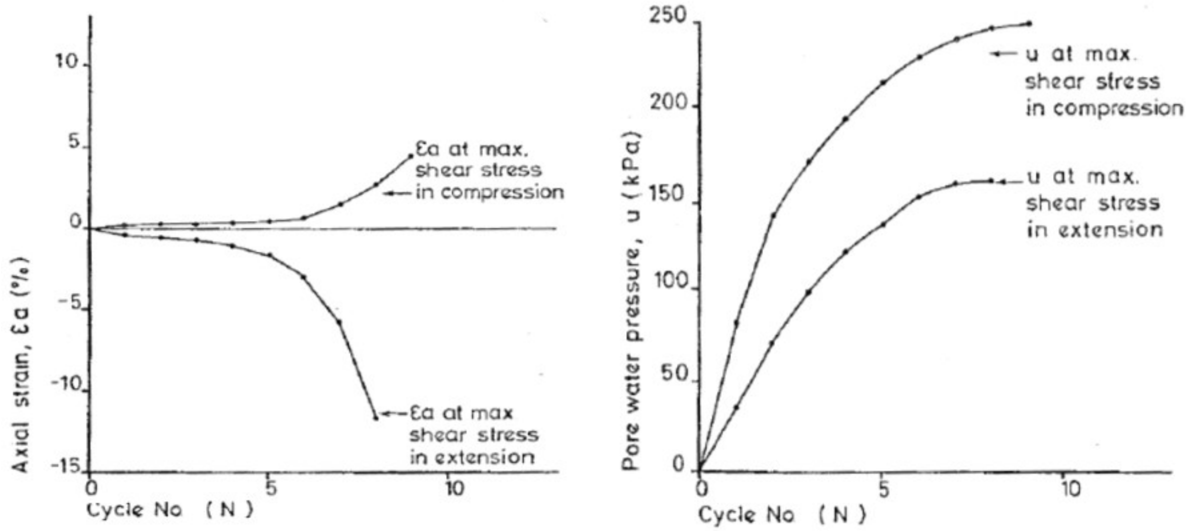


Figure 2.31: Axial strain and pore pressure at maximum shear stress in compression and ectesion in each cycle (after Takahashi et al. [54]).

The rate at which the effective stress path migrates towards the origin of the stress space, and hence the rate at which stiffness reduces and hysteresis increases, has been found to depend on the cyclic stress ratio, consolidation history, and frequency of loading. Increasing the cyclic stress ratio increases the rate of migration of the effective stress cycles. Normally consolidated samples migrate faster than overconsolidated ones. Samples loaded at low frequencies generate higher pore pressures and so migrate more rapidly than samples loaded at high frequencies.

2.41 Tang et al. (2011) [55]

Tang et al. [55] performed cyclic triaxial tests to analyze the variation of cyclic creep and pore water pressure with increasing the cyclic stress ratio (CSR) and number of load cycles. The soil tested was Mucky clay from Shanghai. The index properties of this clay are: $\omega = 50.2\%$, $PL = 23.2\%$, and $PI = 20.2$. Samples were consolidated anisotropically, with $K_0 \approx 0.7$. Axial sinusoidal cyclic loading was applied on the samples with frequencies of 0.5 and 2.5 Hz. Cyclic triaxial tests were conducted continuously on the sample under step loading, in which the initial cyclic stress amplitude was 10 kPa, the periodical amplitude increment was 5 kPa, and each stage consisted of 2,000 loading cycles.

Two threshold cyclic stress ratios were proposed, namely 1) the stability cyclic stress ratio, and 2) the failure cyclic stress ratio. These thresholds divide the cyclic creep development into three stages including, the gradual stabilization stage when the stress level is below the stability of the cyclic stress ratio, the rapid growth stage when the stress level is between the stability cyclic stress ratio and failure cyclic stress ratio, and the instantaneous failure stage when the stress level is above the failure cyclic stress ratio.

Samples subjected to the cyclic loading at low frequency were found to generate greater recoverable elastic strain, accumulated plastic strain, and residual pore water pressure when compared at the same cyclic stress level. The loading frequency was found to have a significant influence on the failure cyclic stress ratio, but not on the stability cyclic stress ratio. This means that the frequency has negligible influence when a test is carried out bellow the failure cyclic stress level.

2.42 Taylor and Bacchus (1969) [56]

Taylor and Bacchus [56] performed strain-controlled triaxial tests on reconstituted Halloysite clay ($G_s = 2.60$, $PL = 36\%$, and $PI = 26$). Isotropically normally consolidated and overconsolidated samples ($OCR = 1$, 2, 4, 8, and 16) were tested. Tests were carried out at $0.2\,Hz$, and the 100 strain cycles were maintained at constant amplitude during the tests. After the cyclic event, conventional undrained compression tests were carried out at a constant strain rate of 1% per minute. These tests were run untill failure was reached.

In normally consolidated samples, cyclic loading caused the mean effective principal stress to reduce at a decreasing rate from its initial value. On the contrary, in overconsolidated samples, the mean effective principal stress increased during the first strain cycle and then decreased. Thus, in overconsolidated samples there is not a marked change in the effective stress due to cyclic loading. As seen from Figure 2.32, the initial increase is greater with higher overconsolidation ratios.

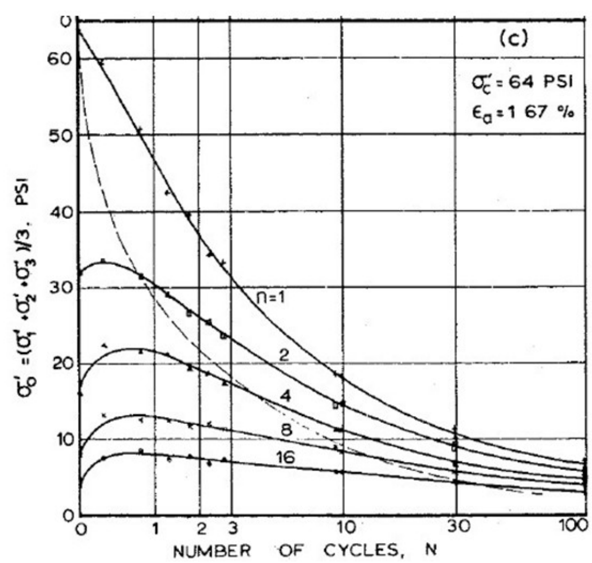


Figure 2.32: Decrease in mean effective principal stress during cyclic test (after Taylor and Bacchus [56]).

In Figure 2.33 are shown the envelopes of the compression peak shear strength for different strain amplitudes and overconsolidation ratios. It is evident that the higher the cyclic strain amplitude, the higher the rate of decrease of the peak shear strength is. For example, for $\varepsilon = 0.14\%$ the envelope is almost horizontal, thus indicating that the strength does not decrease significantly with the number of cycles; this is known as the equilibrium state. Moreover, the rate of decrease increases with overconsolidation ratio.

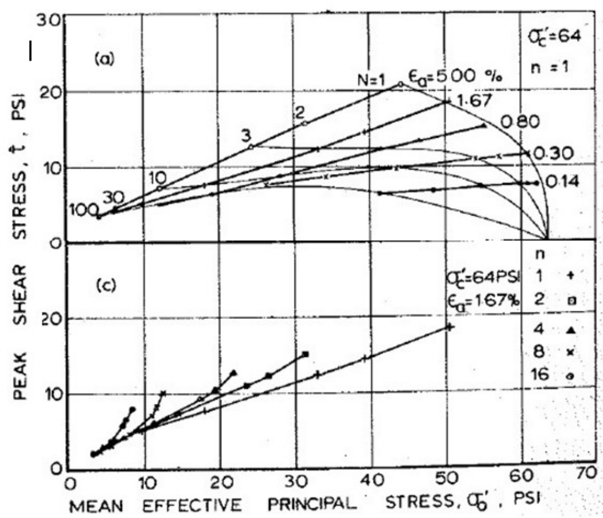


Figure 2.33: Envelopes of compression peak shear strength for different strain amplitudes and OCR values (after Taylor and Bacchus [56]).

Furthermore, cyclic loading causes an apparent overconsolidation state for samples that were initially normally consolidated. Thus, the stress paths under monotonic loading after cyclic loading are typical of overconsolidated clays as shown in Figure 2.34. In addition, results of conventional undrained compression tests showed that there is a decrease in the post-cyclic undrained strength as compared to the monotonic strength. This can be interpreted as a kind of "soil damage". In other words, when a soil is loaded cyclically for a period of time, its stress-strain behavior exhibits a reduction in shear strength. This reduction is a function of the strain amplitude.

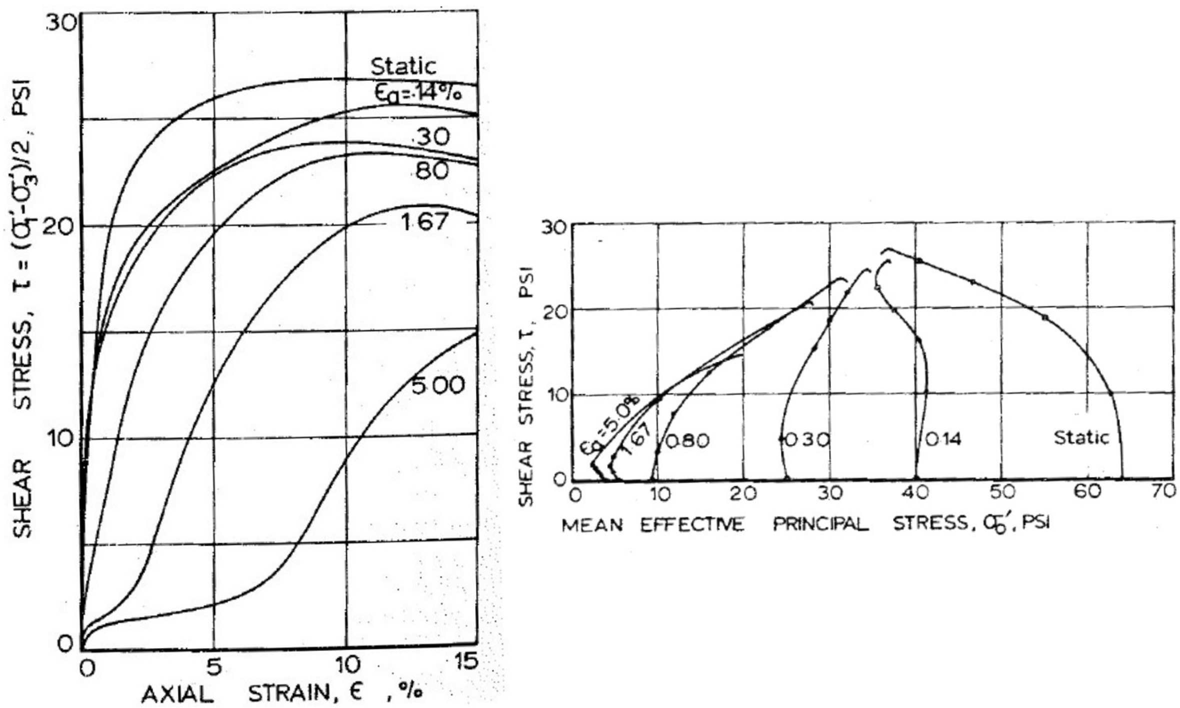


Figure 2.34: Monotonic triaxial tests after cyclic loading at different strain amplitudes (after Taylor and Bacchus [56]).

2.43 Thiers and Seed (1968) [57]

Following the previous work by Seed and Chan [51], Thiers and Seed [57], presented some additional results of cyclic stress-strain behavior of clay. The soil tested was normally consolidated San Francisco Bay Mud, a silty clay with $LL = 88\%$, $PL = 43\%$, and $PI = 45$. Thiers and Seed [57] used a simple shear device to reproduce a constant strain deformation. A uniform rate of strain produeced a complete cycle in one second (1 Hz), with reversal loading. The peak strain was varied from test to test, and each sample was subjected to 200 strain cycles.

Cyclic loading lead to a degradation of the soil stiffness. As shown in Figure 2.35, the decrease in peak load as the number of cycles increased was reflected by the progressive flattening of the stress strain curves. In general, markable degradation occurred during the first 50 cycles; after that point the shear modulus was nearly constant. Accordingly, for a given value of peak strain, the shear modulus decreased by approximately 30% in the first 50 cycles.

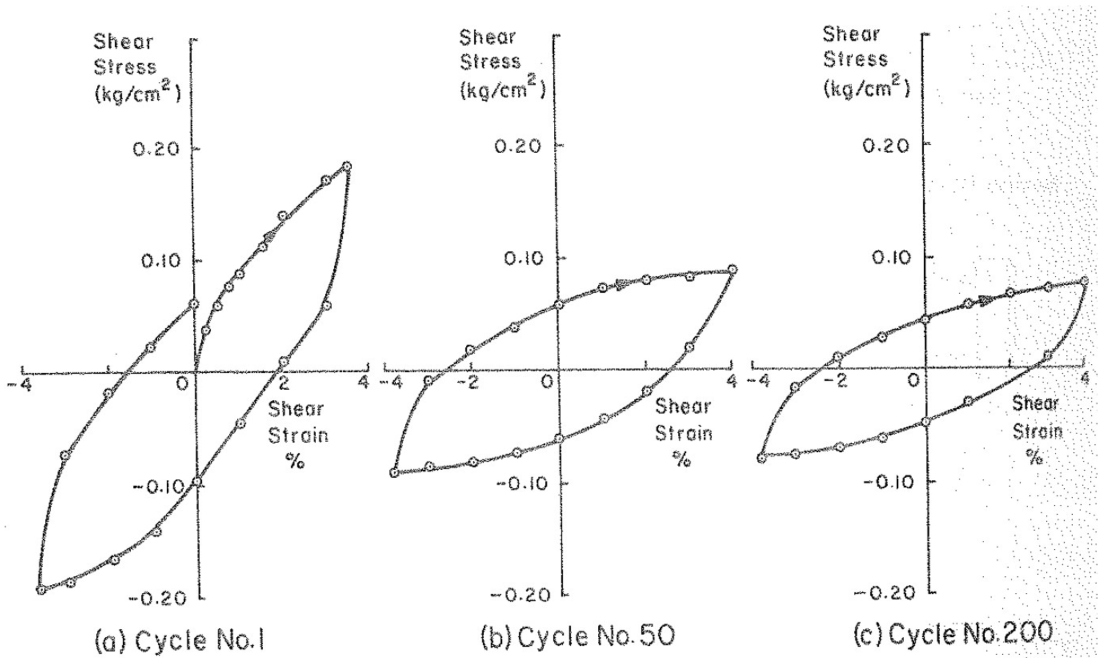


Figure 2.35: Stress strain curves corresponding to 1, 50, and 200 cycles (after Thiers and Seed [57]).

In terms of post-cyclic response, there was a reduction of the monotonic undrained shear strength after cyclic loading. In addition, it was pointed out that the elastic modulus decreased significantly more than the peak monotonic undrained strength in post-cyclic monotonic tests, including some tests in which the post-cyclic strength was essentially unchanged due to the cyclic loading event (Figure 2.36). After 200 cycles of even 2% to 3% shear strain caused a strength reduction of only about 10% as compared to the undrained shear strength before cyclic loading. A reduction in modulus of 40% to 50% was observed. Furthermore, cyclic loading caused an apparent overconsolidation state for samples that were initially normally consolidated. Thus, the stress paths under monotonic loading after cyclic loading are typical of overconsolidated clays.

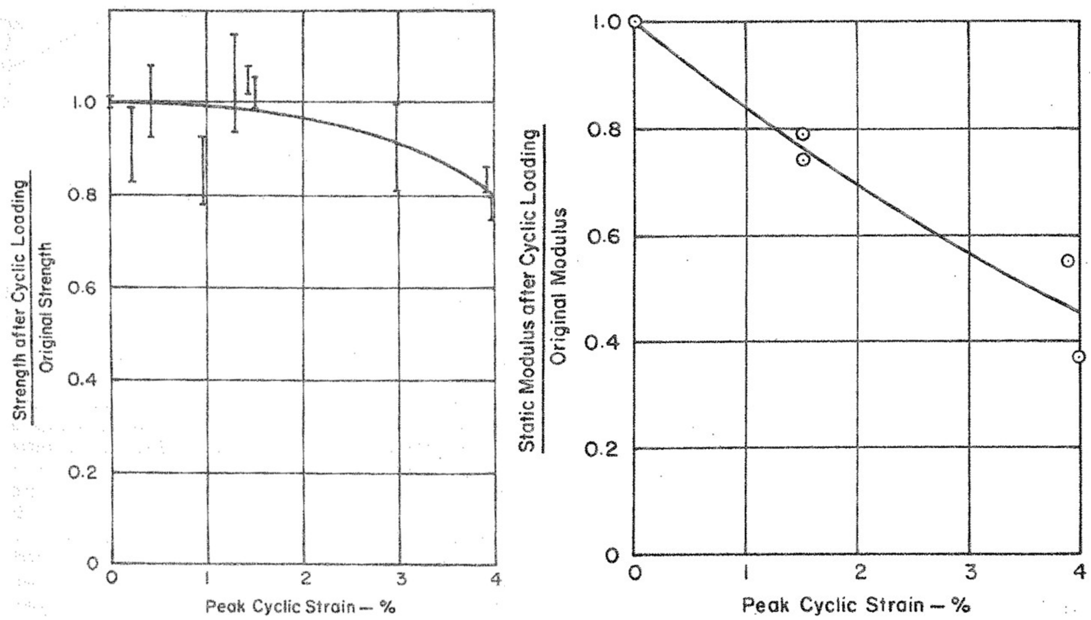


Figure 2.36: Effect of cyclic loading on undrained monotonic shear strength and shear modulus (after Thiers and Seed [57]).

2.44 Vucetic and Dobry (1988) [59] & Vucetic (1990) [58]

Vucetic and Dobry [59] and Vucetic [58] evaluated the soil stiffness degradation of anisotropically normally consolidated and overconsolidated cohesive soils using cyclic direct simple shear tests. A series of strain-controlled undrained cyclic simple shear tests was carried out on samples from the North Paria region off the coast of Venezuela. This soil is a stiff to medium plastic clay deposit (, liquid limit $LL = 71 - 93$, and plasticity index $PI = 45 \pm 6$). Water contents after consolidation ranged from $\omega = 41 - 49\%$. Samples anisotropically consolidated under K_0 conditions and overconsolidation ratio $OCR = 1$, 2, and 4 were tested. The tests were conducted at $0.2\,Hz$, with a regular sinusoidal cyclic strain wave [59], and irregular sinusoidal cyclic strain wave [58]. The cyclic strain amplitude γ_c ranged between about 0.5% and 5% and was kept constant throughout each test. The number of uniform strain cycles N applied to the different specimens ranged between 32 and 130.

Degradation of the undrained secant shear modulus during cyclic loading was measured by the degradation index $\delta = \dfrac{G_{sN}}{G_{s1}}$, where G_{sN} and G_{1N} are the secant shear modulus after N cycles and first cycle at constant shear strain amplitude, respectively. As shown in Figure 2.37, the rate of stiffness degradation decreases with increasing overconsolidation ratio. In other words, the stiffness of normally consolidated samples degrades faster than overconsolidated samples under cyclic loading. Moreover, the strain rate effect for the clay tested seems to be more pronounced for normally consolidated than for overconsolidated clays. Finally, it was observed that after significant cyclic stiffness degradation, "S-shaped" unloading-reloading curves developed, irrespective of the values of the cyclic shear strain amplitude and overconsolidation ratio.

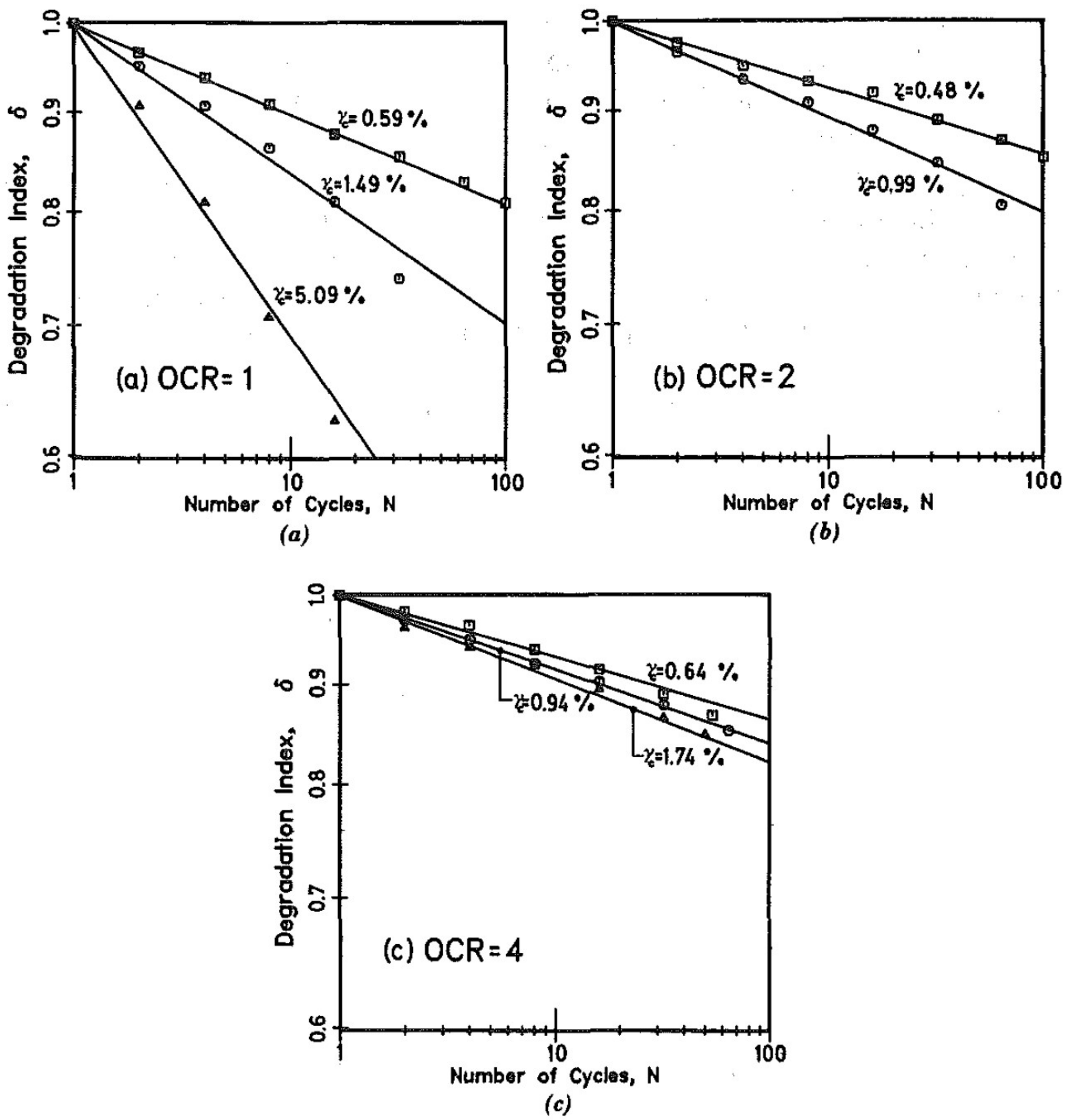


Figure 2.37: Degradation index versus number of cycles for different overconsolidation ratios.

2.45 Vucetic and Dobry (1991) [60]

Vucetic and Dobry [60] presented a study of the influence of the plasticity index (PI) on the cyclic stress-strain parameters of saturated cohesive soils needed for site response evaluations and seismic microzonation. No test were run in this work. Instead, based on the review of a number of available cyclic loading results, it was concluded that the plasticity index (PI)

is one of the main factors controlling the locations of the modulus reduction curve G/G_{max}. For a wide variety of saturated cohesive soils, as the PI increases, the ratio G/G_{max} increases if compared at the same shear strain amplitude as shown in Figure 2.38. This relation was noted for both normally consolidated as well as overconsolidated clays.

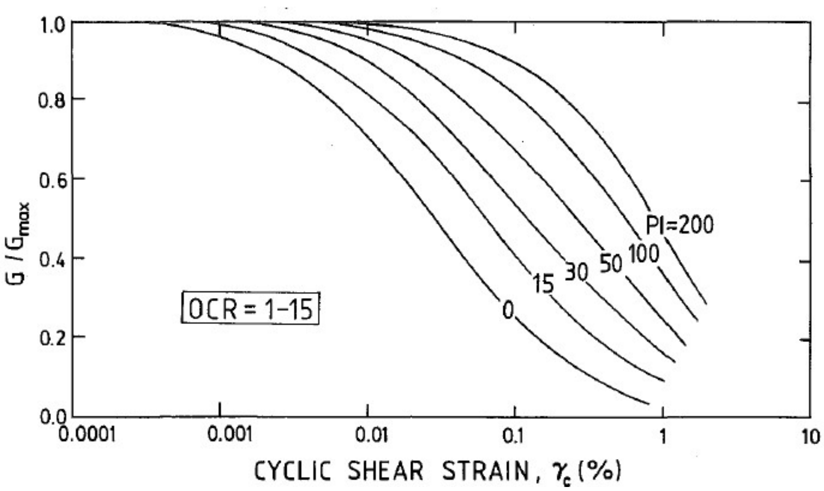


Figure 2.38: Relation between G/G_{max} versus γ_c and soil plasticity index for normally and overconsolidated cohesive soils (after Vucetic and Dobry [60]).

Figure 2.39 shows the effect of cyclic stiffness degradation on G/G_{max} for soils with PI = 15 and 50, where the G/G_{max} curves corresponding to N = 1, 10, 100, and 1000 cycles for a wide range of cyclic strain amplitudes are included. Also included are the $N = 1$ curves for PI = 0, 30, 100, and 200 (broken lines). Form this Figure is clear that the relative effect of cyclic stiffness degradation is much more significant for low plasticity clays than for medium or high plasticity soils. In general, G/G_{max} is affected more by a variation in the plasticity index than by cyclic degradation.

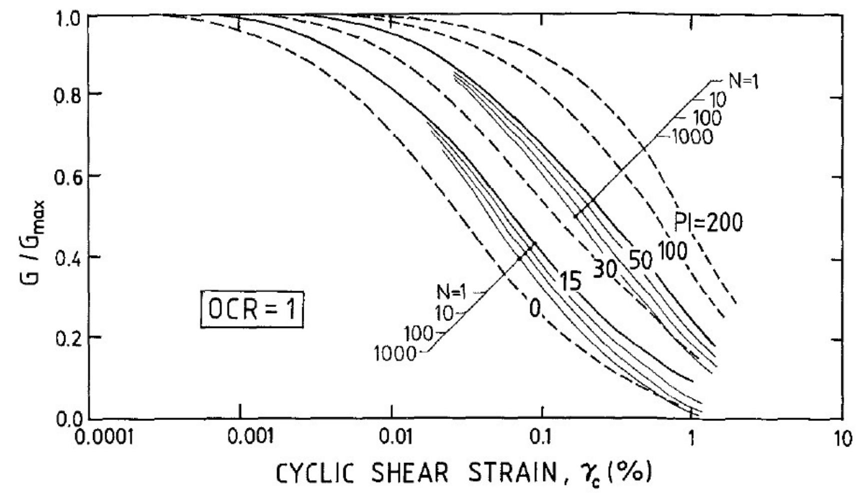


Figure 2.39: Effect of cyclic stiffness degradation on G/G_{max} for soils of different plasticity indices (after Vucetic and Dobry [60]).

2.46 Yasuhara et al. (1982) [68]

Yasuhara et al. [68] performed stress-controlled cyclic triaxial compression tests on remolded Ariake clay, a soft marine clay with $G_s = 2.65$, $LL = 115\%$, and $PI = 58$. Isotropic and anisotropic consolidated specimens were used. Multistage loading was cycled under drained or undrained conditions until the specimen attained failure, frequencies of 1 and 0.1 Hz were used. Each stage loading was applied during one hour; generally the magnitude of the cyclic deviator stress was one-fifth of the confining pressure. After the first hour, the magnitude of the cyclic deviator stress was increased to two-fifths and cyclically applied during one hour. The same loading process was repeated until failure was reached or the specimen reached 15% axial strain. Tests were performed under undrained or drained conditions.

Cyclic loading applied to isotropically consolidated samples induces pore pressure more significantly than for anisotropically conslidated ones, i.e.,. That is, the effective stress paths of anisotropically consolidated samples were almost vertical or did not change substantially. As shown in Figure 2.40, isotropically consolidated samples developed higher pore pressure than anisotropically consolidated ones. On the other hand, the results of dynamic drained triaxial tests (IRD), showed a tendency for dilatancy that was different form the results of static drained compression (CID) tests. As shown in Figure 2.41, the dilatancy effect exerted during repeated loading was found to be governed by not only the effective stress ratio, but also by the frequency of repeated loading. The higher the frequency, the more significant will be the dilative behavior.

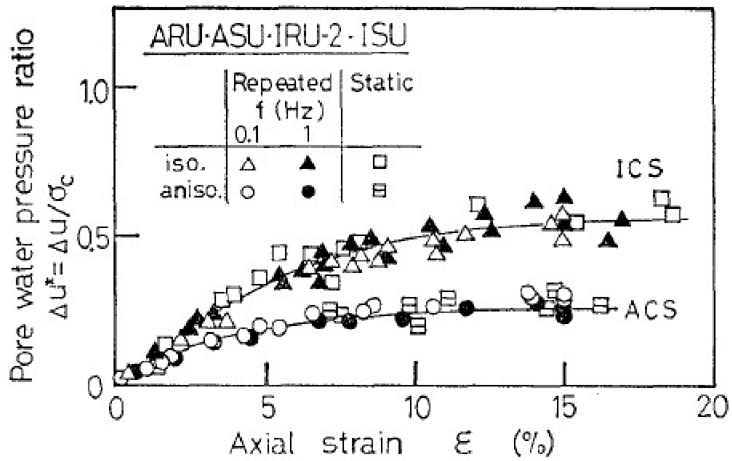


Figure 2.40: Relation between axial strain and pore water pressure on isotropically and anisotropically consolidated clay (after Yasuhara et al. [68]).

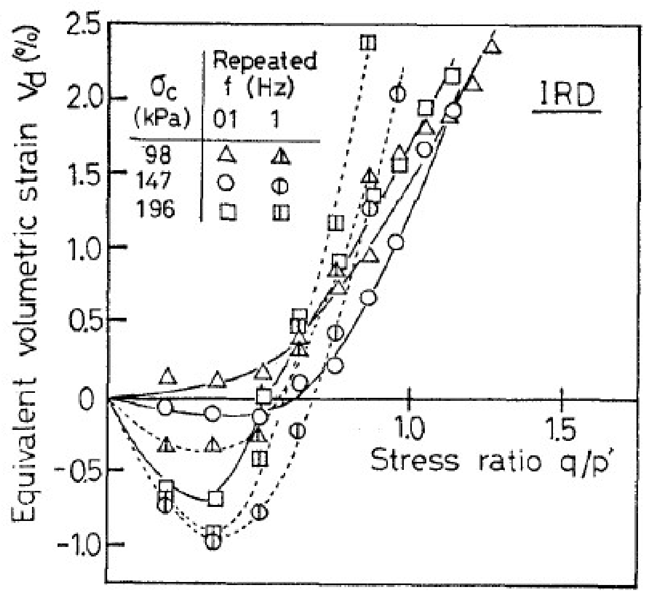


Figure 2.41: Dilatancy of isotropically consolidated samples under drained cyclic loading (after Yasuhara et al. [68]).

Finally, cyclic strength of anisotropically consolidated samples was found to be larger than that of isotropically consolidated samples below 10% shear strain. But when the strain amplitude is 15%, no difference in strength between anisotropically and isotropically consolidated specimens was noted. The cyclic strength is not appreciably affected by the frequency of cyclic loading, but the time to reach a specific shear strain will be much less for low frequencies than high frequencies because of creep effects.

2.47 Yasuhara and Andersen (1989, 1991) [65; 66]

Yasuhara and Andersen [65; 66] used a cyclic direct simple shear device to simulate a series of storm loading type events. In these simulations the cyclic loading was separated by periods of drainage. The soil used in these tests was plastic Drammen clay ($G_s = 2.76$, $\omega = 52\%$, $LL = 55\%$, and $PI = 27$). Specimens with $OCR = 4$, 10 and 40 were tested. This work focused on the settlement of overconsolidated clay due to the dissipation of cyclically induced pore pressure. On average, the number of load cycles was equal to 100. For each cyclic event the frequency was $0.1\,Hz$.

Results from tests on normally consolidated specimens indicated that the cyclically induced pore pressure decreases for each series of cyclic loading, and the reduction in void ratio during subsequent pore pressure dissipation becomes smaller after each series. An opposite tendency was observed for overconsolidated samples.

The cyclically induced pore pressure increased for each series of cyclic loading, and the change in void ratio during the subsequent pore pressure dissipation increased. Cyclic loading, followed by drainage, may have a deteriorating effect on overconsolidated clays and may lead to lower resistance to subsequent undrained cyclic loading. As shown in Figures 2.42 and 2.43, pore pressure generation increased with successive cyclic loading periods. Also Figures 2.44 and 2.45 show that, during the subsequent pore pressure dissipation, the void ratio increases.

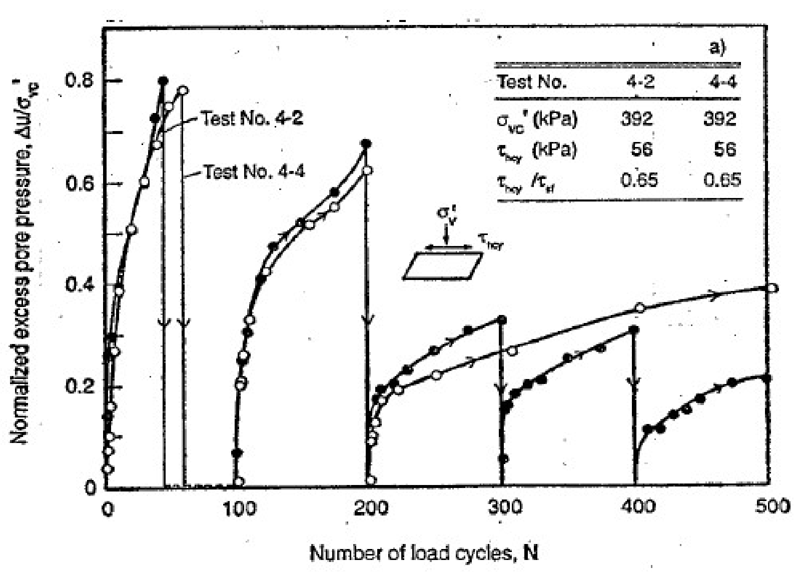


Figure 2.42: Pore pressure evolution during 5 cyclic loading events each followed by drainage, normally consolidated soil.

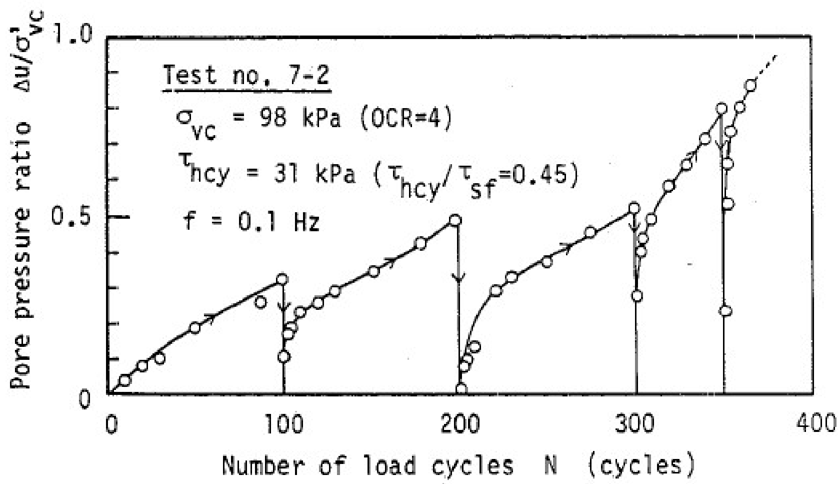


Figure 2.43: Pore pressure evolution during 5 cyclic loading events each followed by drainage, overconsolidated soil.

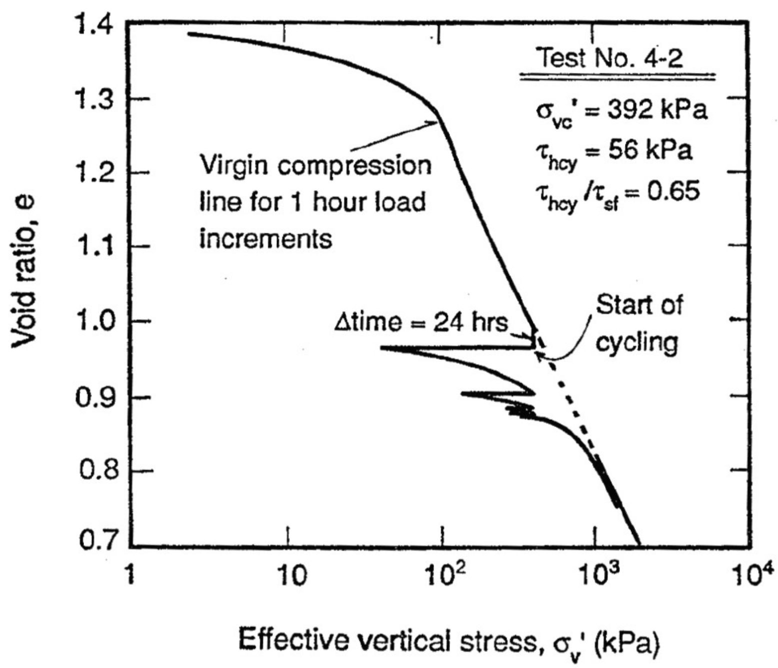


Figure 2.44: Void ratio evolution during cyclic loading events each followed by drainge, normally consolidated soil.

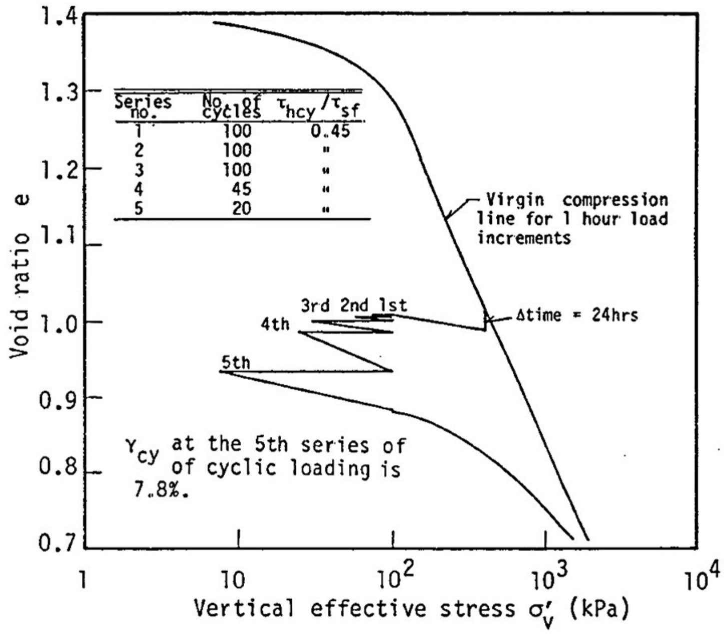


Figure 2.45: Void ratio evolution during cyclic loading events each followed by drainge, overconsolidated soil.

Recompression settlements will occur when cyclically induced pore pressures dissipate. The post-cyclic recompression followed the same reloading consolidation curve as an oedometer test subjected to the same number of vertical unloading-reloading stress histories as in the cyclic direct simple shear test. However, the settlements accumulated after a number of cyclic load series may be significantly greater than after the same number of static unloading-reloading histories in an oedometer test. It was concluded that the post-cyclic recompression settlements of overconsolidated clay could approximately be estimated by using the recompression index in oedometer tests.

2.48 Yasuhara et al. (1992) [69]

Yasuhara et al. [69] performed cyclic triaxial tests in order to investigate the behavior of clay during and following undrained cyclic loading. The soil tested was Ariake clay, a highly plastic marine clay ($G_s = 2.58 - 2.65$, $LL = 115 - 123\%$, and $PI = 72 - 69$). First, samples of this clay were isotropically consolidated. Then, cyclic loading was applied to specimens under stress-controlled conditions at frequencies of 0.1, 1.0 and 3.0 Hz. Patterns of one-way and two-way loading were used. After the cyclic loading event, some samples were allowed to drain. Finally, conventional monotonic undrained triaxial tests were performed until the samples reached failure.

Excess pore pressure and residual shear strain both developed more rapidly under two-way cyclic loading than under one-way loading. Thus, cyclic strength of clay samples under two-way loading was found to be slightly smaller that under one-way loading (Figure 2.46).

Undrained strength decreased with amplitude of cyclic induced excess pore pressure, while post-cyclic undrained strength followed by drainage increased in proportion to the amplitude of cyclic induced pore pressure. Post-cyclic recompression volumetric strains were governed by the magnitude of cyclic induced pore pressures.

Results from post-cyclic monotonic loading showed that stress paths of clay with apparent overconsolidation induced by cyclic loading are close to those of clay overconsolidated by an actual unloading. In general, for Ariake clay, the post-cyclic undrained strength of normally consolidated specimens without drainage decreased in accordance with the magnitude of pore pressure developed during cyclic loading. As shown in Figure 2.47, the undrained strength of normally consolidated samples was reduced by an undrained cyclic loading history and was increased by drainage after undrained cyclic loading.

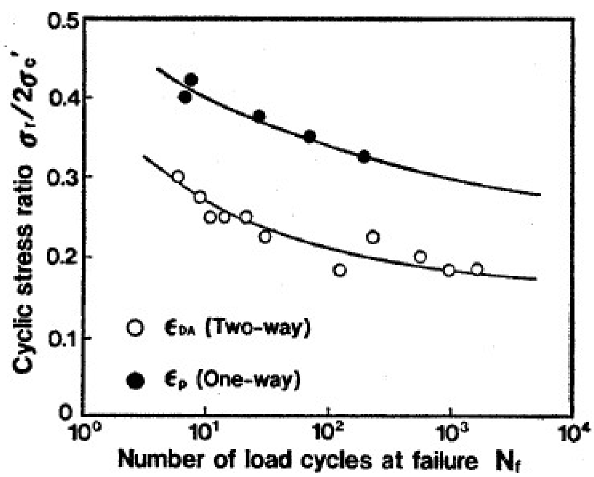


Figure 2.46: Cyclic stress ratio with the number of cycles during one way and two way cyclic loading (after Yasuhara et al. [69]).

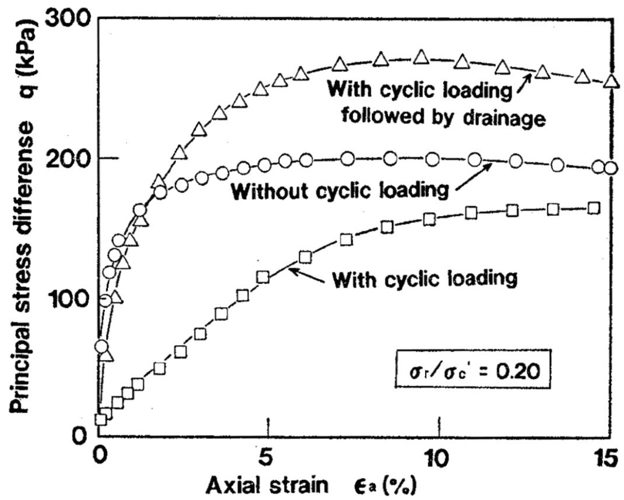


Figure 2.47: Stress-strain curves in undrained monotonic triaxial tests after two-way undrained cyclic loading (after Yasuhara et al. [69]).

2.49 Yasuhara and Toyota (1997) [64]

Yasuhara and Toyota [64] investigated the effect of applying an initial deviator stress q_s, associated with anisotropic stress states, on post-cyclic degradation of strength and stiffness of a reconstituted silty clay. Low plasticity Keuper Marl silty clay with index properties $LL = 40.2\%$, $PI = 23$, was employed in the study. Different values of initial sustained deviator stress were used: $q_s/p'_c = 0$, 0.3, 0.6 and 0.75. Here $p'_c = 198\,kPa$ is the mean effective principal stress. Cyclic loading at $0.1\,Hz$ was applied to the specimen until a prescribed excess pore pressure was observed. Then monotonic undrained shear was applied.

In Figures 2.48 and 2.49 are shown the results for the cases of $q_s/p'_c = 0$, which corresponds to an isotropic stress state and $q_s/p'_c = 0.30$, which corresponds to an anisotropic stress state (i.e., initial stress $q_s = 58.8\,kPa$). In both conditions, with and without initial sustained shear stress, a common observed characteristic was that as the cyclic induced excess pore pressure increased, the effective stress path became very similar to that for an overconsolidated clay during undrained shear. Normally consolidated samples exhibited an apparent overconsolidation due to cyclic loading. In addition, there was no large difference between the pre-cyclic and post-cyclic undrained shear strengths (Figure 2.49). In other words, cyclic loading on a normally consolidated soil induces positive excess pore pressure. As a result, if the sample is subsequently sheared under monotonic loading, the response will be quite similar to that for an overconsolidated sample.

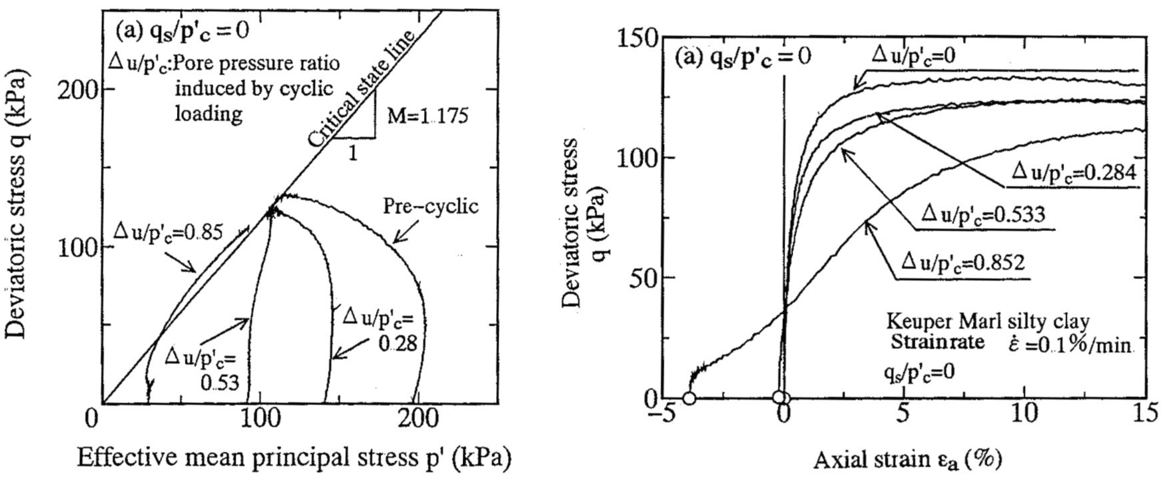


Figure 2.48: Pre-cyclic and post-cyclic stress paths for isotropic stress state ($p'_c = 198\,kPa$) (after Yasuhara and Toyota [64]).

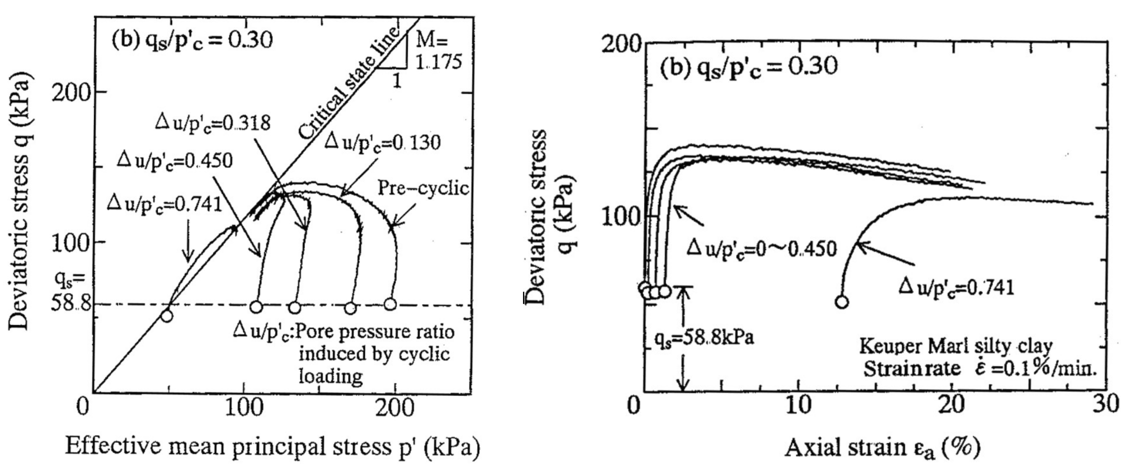


Figure 2.49: Pre-cyclic and post-cyclic stress-strain curves for anisotropic stress state ($p'_c = 198\,kPa$) (after Yasuhara and Toyota [64]).

Degradation characteristics in relation to strength and stiffness in relation to u/p'_c were not influenced by the magnitude of initial sustained shear stress. In addition, the stiffness degradation was more markable than the strength degradation due to cyclic loading (Figure 2.50). The post-cyclic strength only decreased by approximately 20% of the initial or pre-cyclic undrained strength.

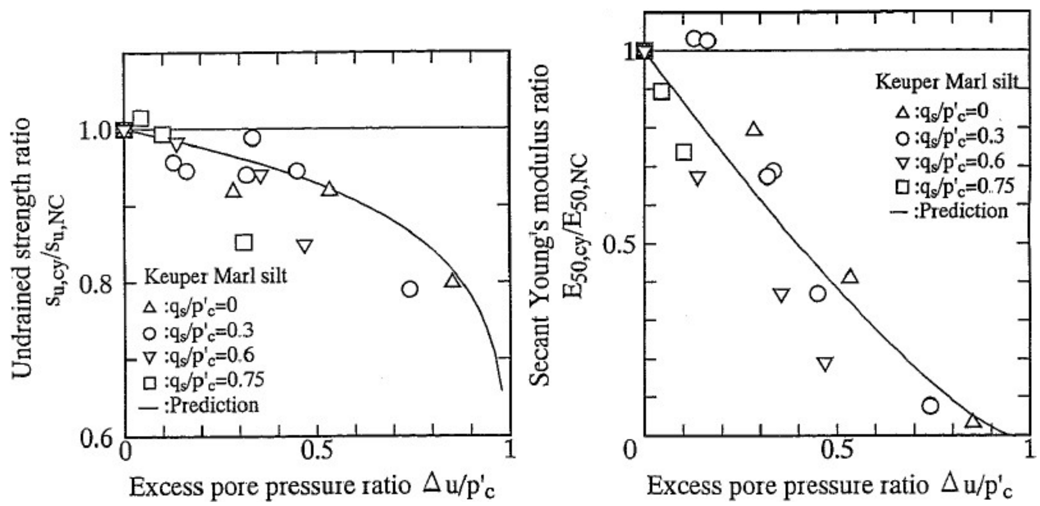


Figure 2.50: Undrainded shear strenght and Young moduli degradation (after Yasuhara and Toyota [64]).

2.50 Yasuhara et al. (2003) [70]

Stress-controlled undrained cyclic triaxial tests followed by strain controlled monotonic triaxial test were carried out on normally and overconsolidated Keuper Marl silty clay to investigate strength and stiffness degradation characteristics. Reconstituted soil was used, index properties $G_s = 2.787$, $LL = 38.6\%$, and $PI = 19.7\%$. First, samples were isotropically consolidated in the triaxial cell with different overconsolidation ratios, $OCR = 1$, 2, 4, and 10. Then, an initial sustained deviator stress (q_s) was applied under drained conditions (i.e., anisotropic consolidation). Then, two way loading cyclic loading was applied at $0.1\,Hz$ in stress controlled conditions.

In figure 2.51 are presented the stress paths before and after cyclic loading for normally and overconsolidated samples. It was evident that normally consolidated samples subjected to cyclic loadig become in an apparent overconsolidation due to cyclic loading, and this apparent state increase with the increase of the generated pore water pressure by cyclic loading as reported before. From Figure 2.52 it is noticeable that the gradient and hence the stiffness at the beginning of each curve decreased along with the peak deviator stress, with increasing cyclic-induced excess pore pressures (i.e., increasing the number of cycles). Cyclic degradation of stiffness is more marked than that of undrained strength and this tendency increases with increasing overconsolidation ratio. In addition, the post-cyclic undrained shear strength did not reduce more than 20% of the precyclic undrained shear strength in most of the cases (i.e., $OCR = 1$, 2, 2, 10). For this reason, the authors concluded that the cyclic softening which sometimes triggers lateral deformation and instability in fine-grained plastic silt during earthquakes must be caused by a decrease in stiffness rather than undrained strength.

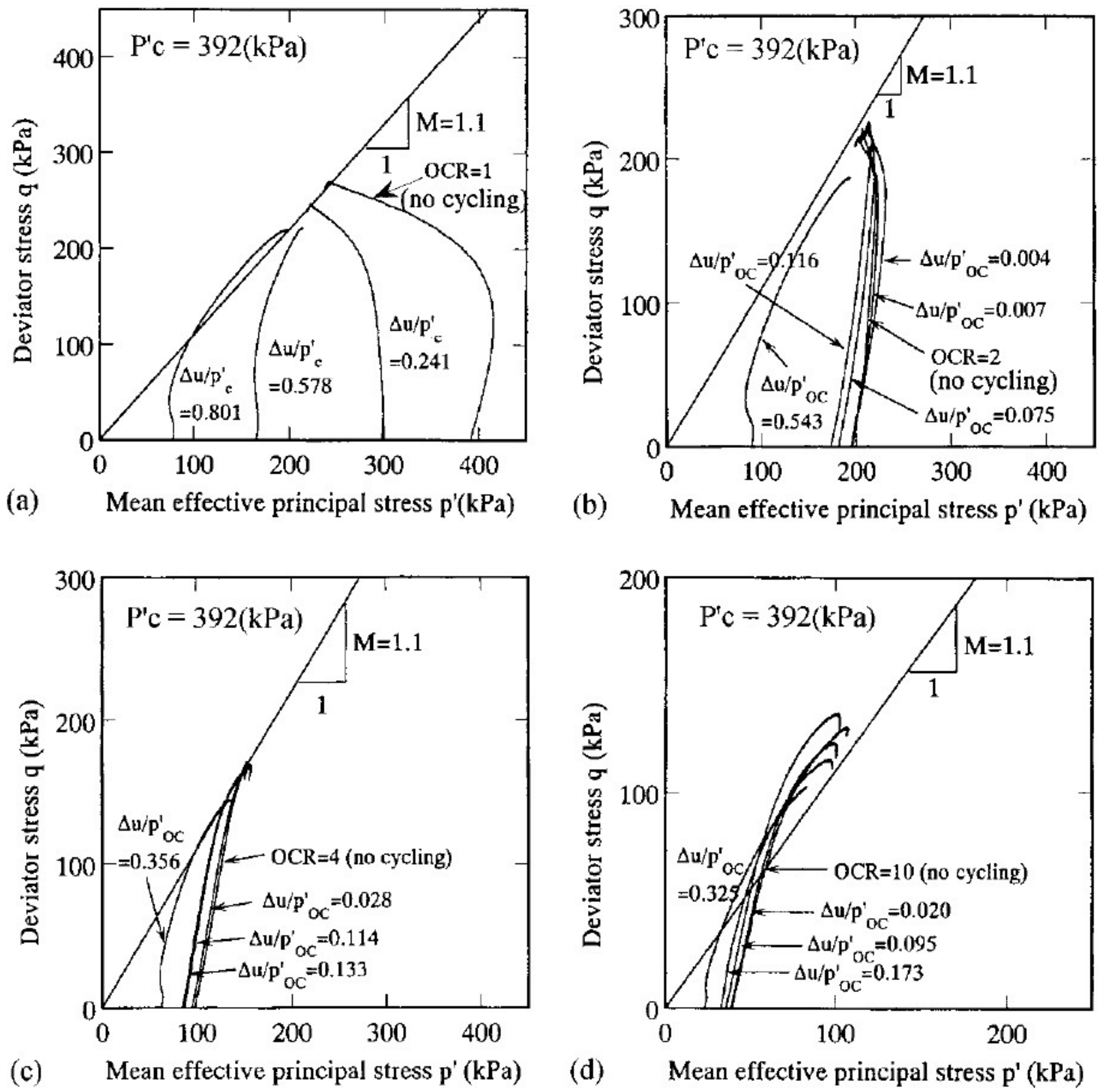


Figure 2.51: Post-cyclic stress paths for $OCR = 1$, 2, 4, and 10 (after Yasuhara et al. [70]).

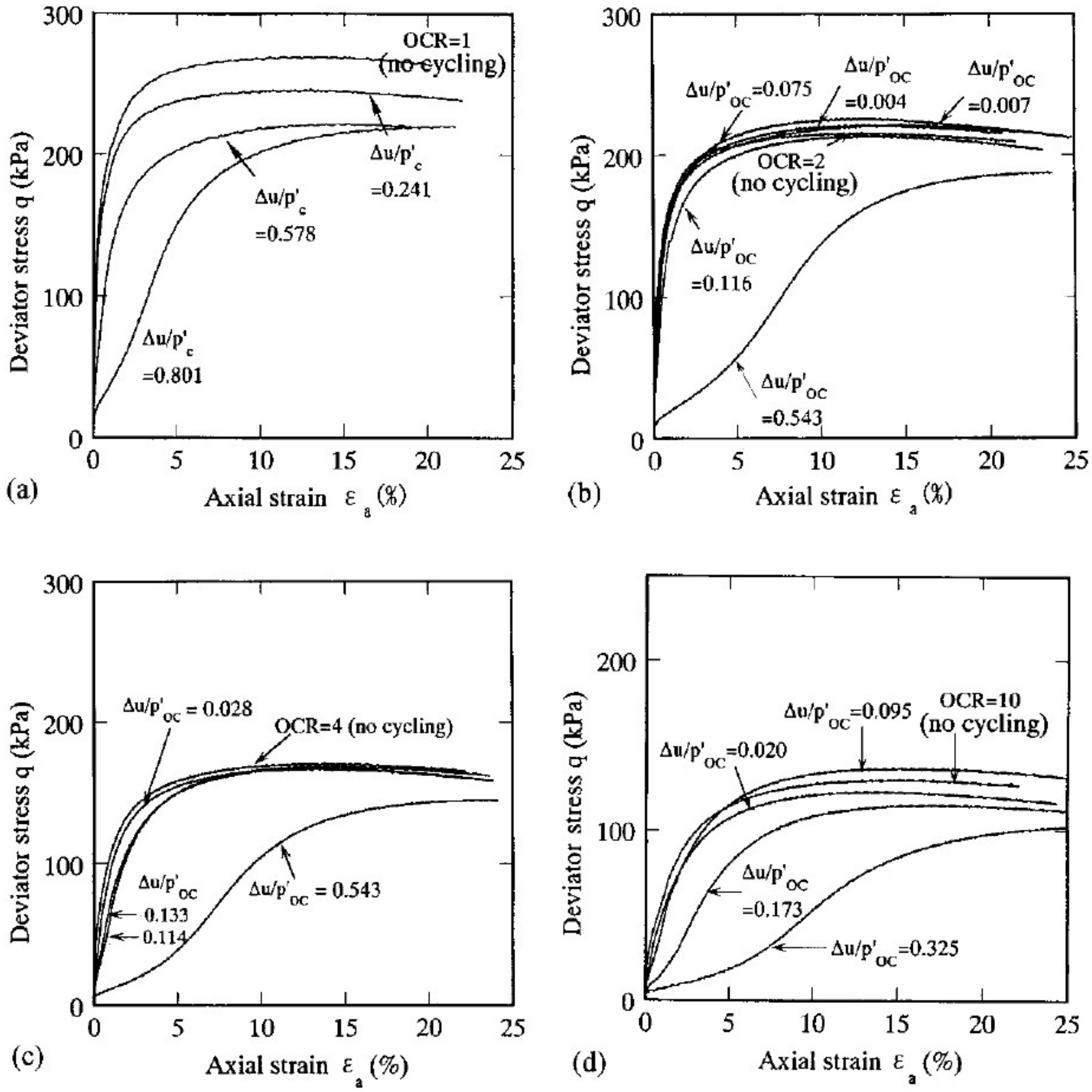


Figure 2.52: Post-cyclic stress-strain curves for OCR = 1, 2, 4, and 10 (after Yasuhara et al. [70]).

Furthermore, stiffness degradation is more marked than that of undrained strength. In Figure 2.53 is shown the post-cyclic undrained strength ratio versus normalized excess pore pressure relations for different overconsolidation ratios. It is noted that degradation (i.e., undrained shear strength and stiffness) increases with the increase of the generated pore water pressure by cyclic loading. Besides, degradation is more notable in overconsolidated samples than in normally consolidated ones.

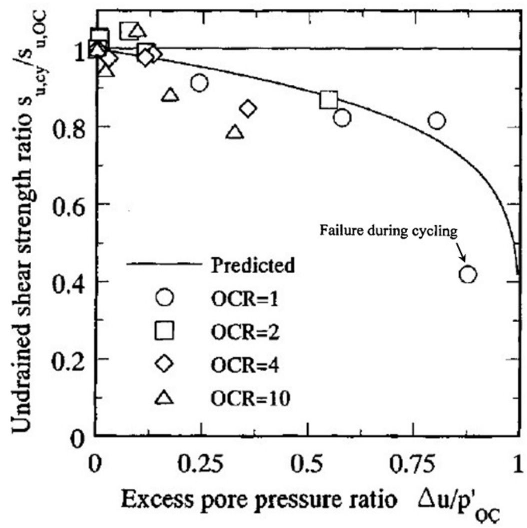


Figure 2.53: Post-cyclic undrained strength ratio for different overconsolidation ratios (after Yasuhara et al. [70]).

2.51 Yildirim and Ersan (2007) [71]

Using a simple shear device, Yildirim and Ersan [71] investigated the consolidation settlements of soft clay caused by cyclic loading and associated factors such as the number of cycles and stress level. Normally consolidated samples were subjected to five consecutive series of cyclic loading; each series consisted of 50 cycles. Between each cyclic loading series drainage was allowed for $60\,min$. Cyclic tests were performed under stress-controlled conditions at $0.1\,Hz$. Two-way sinusoidal wave loading with different stress levels and number of cycles was employed.

Figure 2.54 indicates that relatively high pore water pressures at the first loading stage were observed during undrained cyclic loading. However, due to drainage after each cyclic loading stage, which leads to lower void ratios, excess pore pressure generation gradually decreased for the consecutive loading stages. Moreover, increasing the cyclic stress amplitude (i.e., increasing the stress ratio) caused an increase in pore water pressures and consequently post-cyclic settlements.

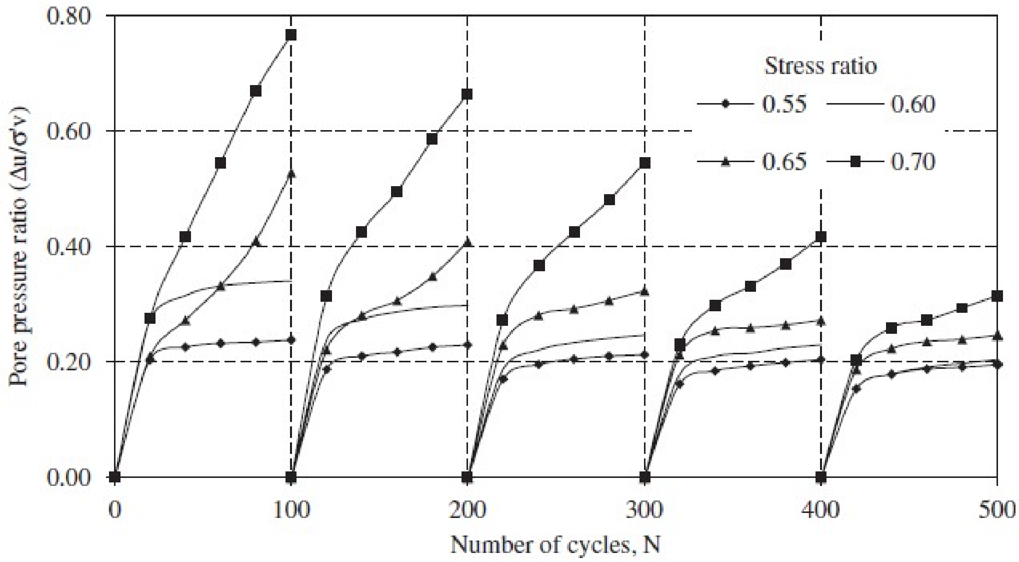


Figure 2.54: Change of pore pressure for different stress ratios (after Yildirim and Ersan [71]).

Furthermore, the cyclic stress ratio and the number of cycles affects the development of shear strains. Strains increased with increasing stress ratio and number of cycles. However, for stress ratio levels of 0.55 and 0.60, the increase of the shear strain with number of cycles was essentially negligible. as shown in Figure 2.55, larger strains developed at the next two stress levels and shear strains begin to accelerate with increasing number of cycles.

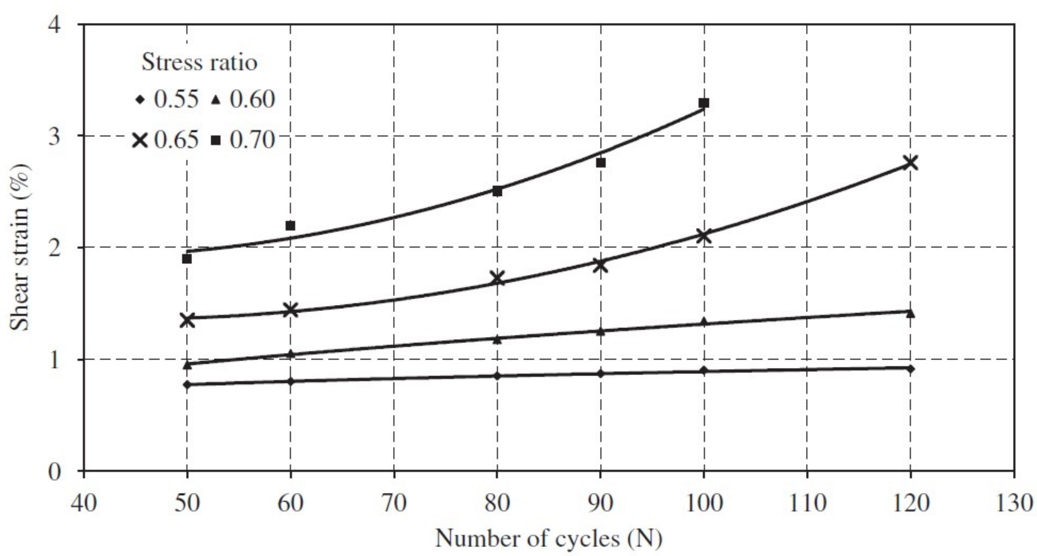


Figure 2.55: Shear strain versus number of cycles for different cyclic stress ratios.

In terms of volumetric strain after each cyclic stage, Figure 2.56 shows the change of volumetric strains versus stress ratio and number of cyclic loading stages for $N = 100$. Volumetric strains at the first cyclic loading stage are higher than the last cyclic stages. Accordingly, voulmetric strain decreased and became smaller for the final stages.

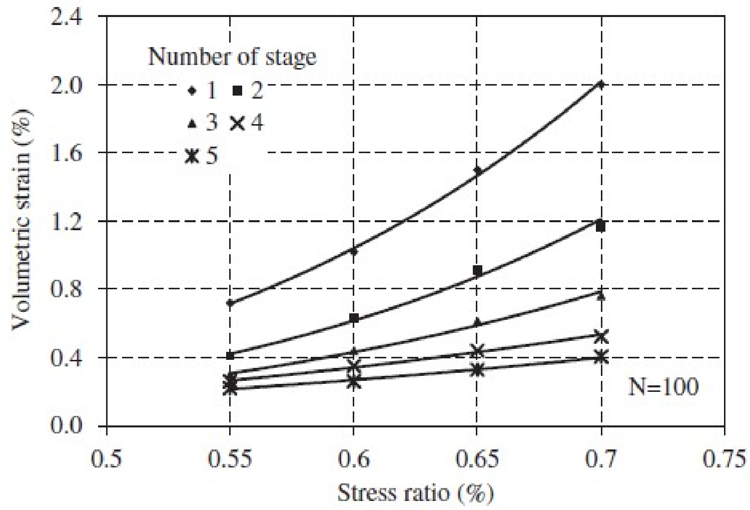


Figure 2.56: Volumetric strain changes.

2.52 Yilmaz et al. (2004) [72]

Yilmaz et al. [72] investigated the undrained shear and deformation behavior of silty clay mixtures through a series of standard and rapid monotonic triaxial tests as well as stress-controlled cyclic triaxial tests. The natural soil samples (PI between 7 and 25) were anisotropically consolidated. Cyclic tests were carried out under one-way (non-reversal) and two-way (reversal loading) cyclic loading conditions. Sinusoidal cyclic axial loads were applied with frequencies of $1\,Hz$ and $0.5\,Hz$, so the viscous behavior of the soil was properly reflected. At the end of each cyclic test, samples were brought back to the anisotropic stress state of pre-cyclic loading and kept for about $5\,min$ under undrained conditions to observe whether an imminent state of failure existed or if any deformations occurred due to possible substantial loss of strength during cyclic loading. Finally, samples were sheared under rapid monotonic loading with a range of stress rates between 10 and $160\,kPa/s$.

Cyclic strain accumulation characteristics were found to critically depend on the relative level of peak cyclic stress with relation to the monotonic shear strength. While plastic strains can rapidly accumulate at practically constant rates per cycle in the case of monotonic strength is exceeded, they tend to remain insignificantly small. Substantial increases of apparent strength were observed during rapid loading tests after cyclic loading, reflecting the pronounced viscous nature of these soils under loads with increased rates. These results underline the dependency of undrained stress-strain response of cohesive soils on loading history and rate effects due to the pronounced viscous response induced by the increased speed of loading. In addition, these results imply that substantial permanent displacements are not likely to occur, unless the foundation capacity defined by monotonic strength is exceeded during seismic shaking.

2.53 Zergoun (1991) [73] & Zergoun and Vaid (1994) [74]:

Zergoun [73] and Zergoun and Vaid [74] presented the results of slow undrained two-way cyclic loading of Cloverdale clay, a soft natural marine clay with $\omega = 50\%$, $PL = 26\%$, $PI = 24$. The effect of cyclic stress level on the development of strain and pore pressure with number of cycles was studied. Consequently, various cyclic deviator stress magnitudes, as a percentage of the initial monotonic undrained strength, were used.

Two patterns were distinguished depending on the cyclic stress level. For cyclic stress levels below 55% of the monotonic undrained shear strength, both residual strain and pore pressure tended toward an equilibrium state with increasing the number of cycles.

However, true equilibrium did not appear to be reached, and the residual strain and pore pressure continued to increase, although at a decreasing rate (Figure 2.57). Second, for high cyclic stress levels the residual strain started to accelerate towards failure after a given number of cycles. Three states were observed in this pattern: first, strain developed at a decreasing rate per cycle, then strain developed at a constant rate per cycle, finally strain developed at an accelerating rate per cycle towards failure. Even though a complete equilibrium state was not attained, there was a clear difference according to the cyclic stress level. A stable condition with negligible residual strain and excess pore pressure evolution was observed provided that the cyclic stress level was lower than the threshold cyclic stress. Moreover, the stress-strain hysteresis loops maintained about the same size, implying low stiffness degradation due to cyclic loading below the threshold cyclic stress. On the other hand, for high cyclic stress levels the stress-strain hysteresis loops increased in size and showed an accelerated migration toward the extension side, with markable stiffness degradation (Figures 2.58 and 2.59).

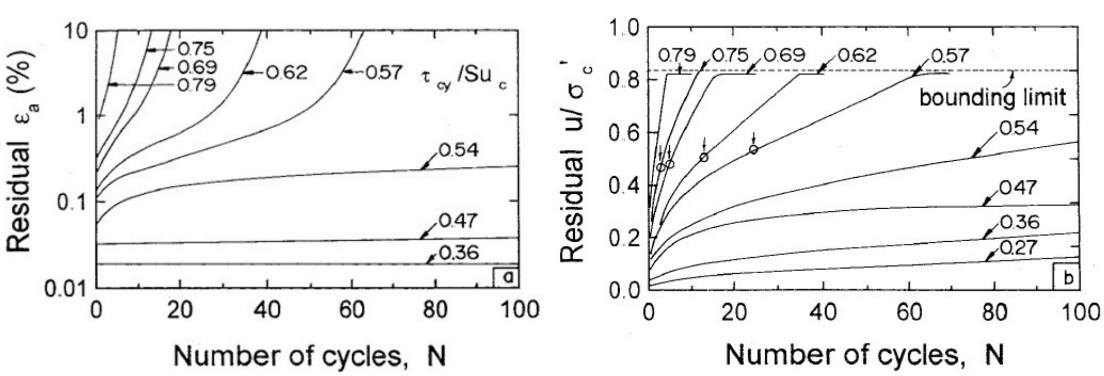


Figure 2.57: Development of residual strain and excess pore pressure.

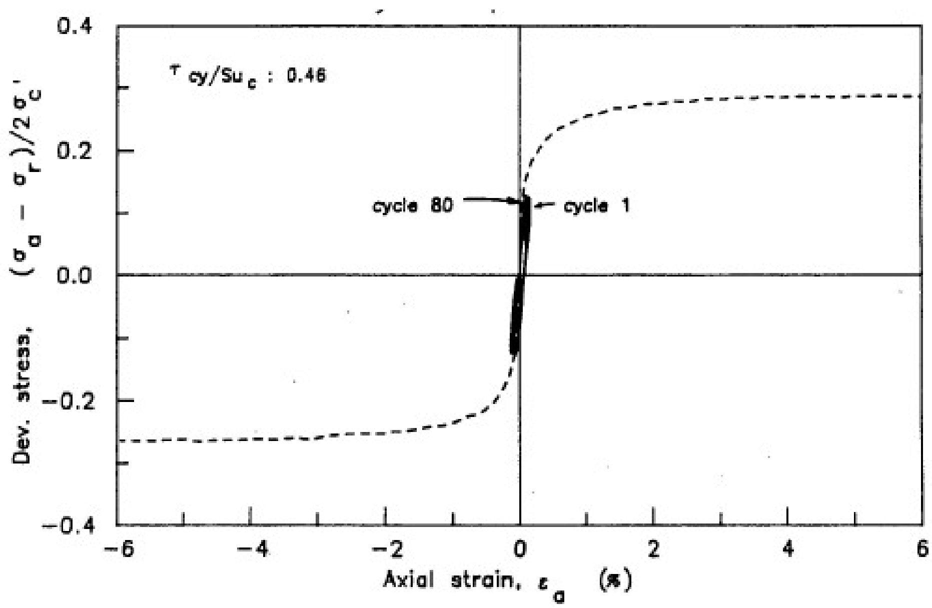


Figure 2.58: Stress-strain hysteresis loops for cyclic stress level below the threshold.

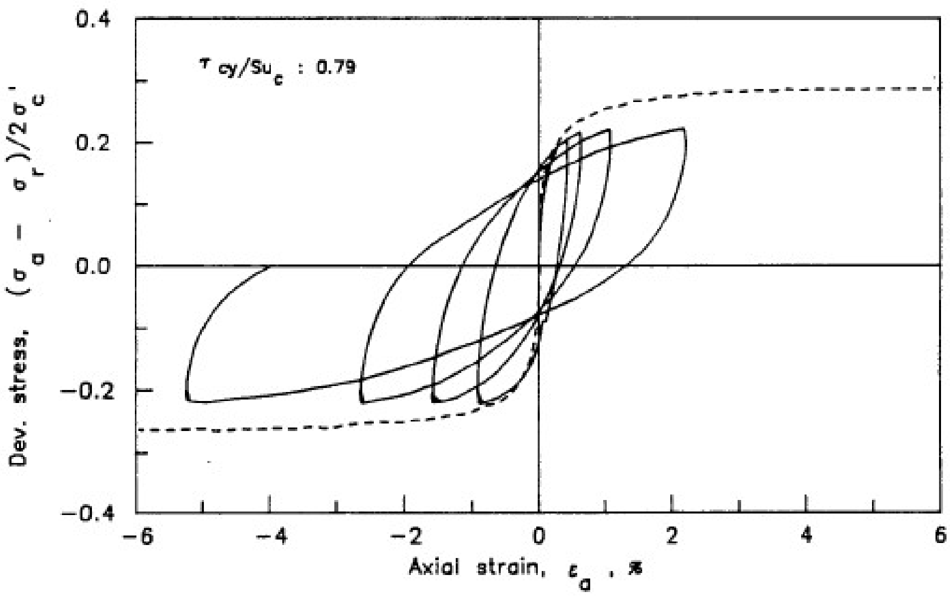


Figure 2.59: Stress-strain hysteresis loops for cyclic stress level over the threshold.

In addition, a general degradation of both strength and stiffness was noted as a result of cyclic loading that seems to increase with the magnitude of the maximum strain during cyclic loading. In Figure 2.60 is shown the relation between maximum strain due to two-way cyclic loading and post-cyclic undrained strength for different test conditions such as: 1) compression-extension and extension-compression cyclic loading, 2) cyclic loading under constant stress ratio amplitude, and 3) post-cyclic compression and extension tests. It is evident that this relation is applicable regardless of the manner in which this strain was accumulated.

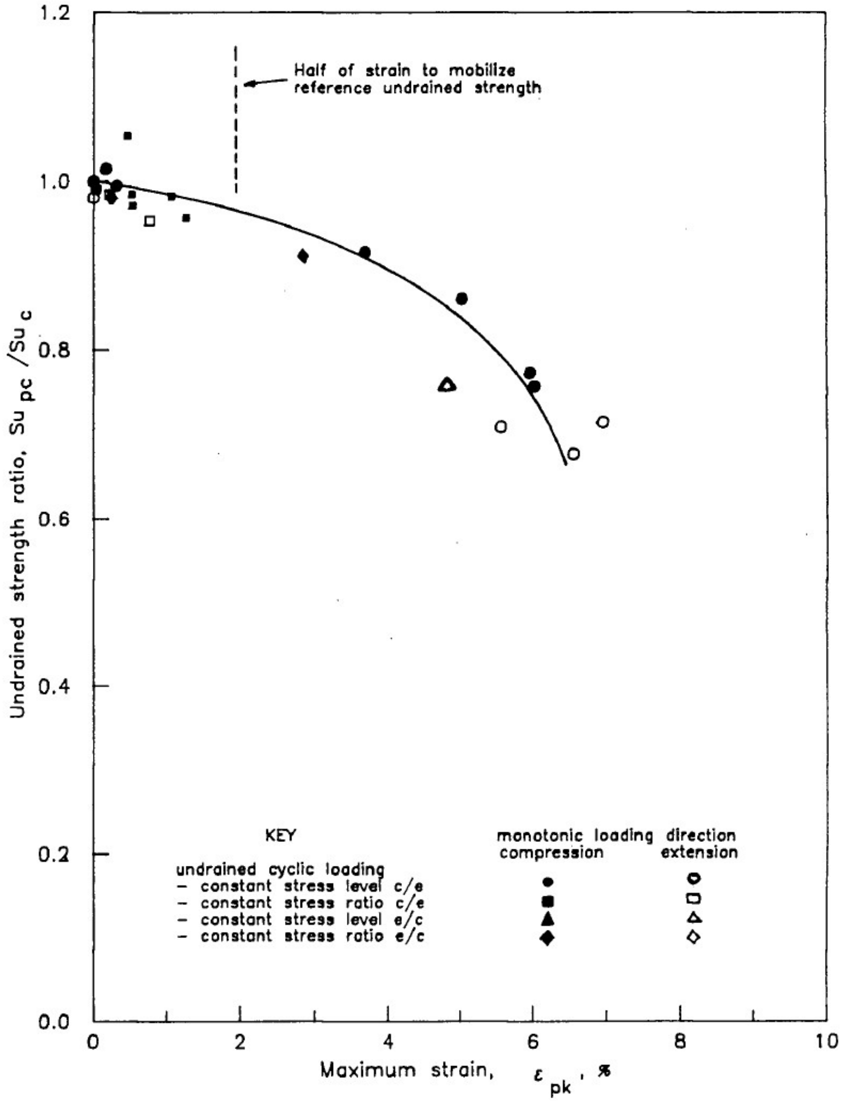


Figure 2.60: Relationship between maximum strain due to cyclic loading and post-cyclic undrained strength.

2.54 Zhou and Gong (2001) [75]

Zhou and Gong [75] performed cyclic triaxial tests in order to study the degradation characteristics of Hangzhou clay from China.

All samples were undisturbed and normally consolidated, with $\omega = 32.1\%$, $G_s = 2.71$, $PL = 19.33\%$, and $PI = 14.43$. Stress-controlled cyclic loading was imposed in the vertical direction, with a semi-sine wave at a frequencies of 0.01, 0.05, 0.1, 0.5 Hz. Different confining stresses, numbers of cycles, cyclic stress ratios, and overconsolidation ratios (OCR = 1.2, 1.4, 2, 4, 8) were used.

According to the results, for low cyclic stress ratios, strain and pore pressure increased slowly with increasing the number of cycles, and failure was reached after a large number of cycles. However, when the cyclic stress ratio was high, large amplitudes of strain and accumulation of pore pressure occurred at the beginning of the test. There was a critical cyclic stress ratio in the clay subjected to cyclic loading. When the cyclic stress ratio was higher than the critical stress ratio, the variations in pore pressure and axial strain were different from those when the cyclic stress ratio was lower than the critical stress ratio.

To quantify the degradation due to cyclic loading, the degradation index was defined as the ratio of the cyclic axial strain corresponding to cycles 1 and N. A large degradation index value corresponds to a low degree of soil degradation. As shown in Figure 2.61a, soil degradation was found to be directly related to the cyclic stress ratio. Comparing the degradation index at the same number of cycles, it becomes smaller with increasing cyclic stress ratio, which implies more soil degradation. Additionally, the degradation index was lower for the normally consolidated sample, which means that stiffness in normally consolidated samples degrades to a greater degree and faster than for overconsolidated ones. This work suggested that the overconsolidation ratio is an important factor in the study of soil degradation.

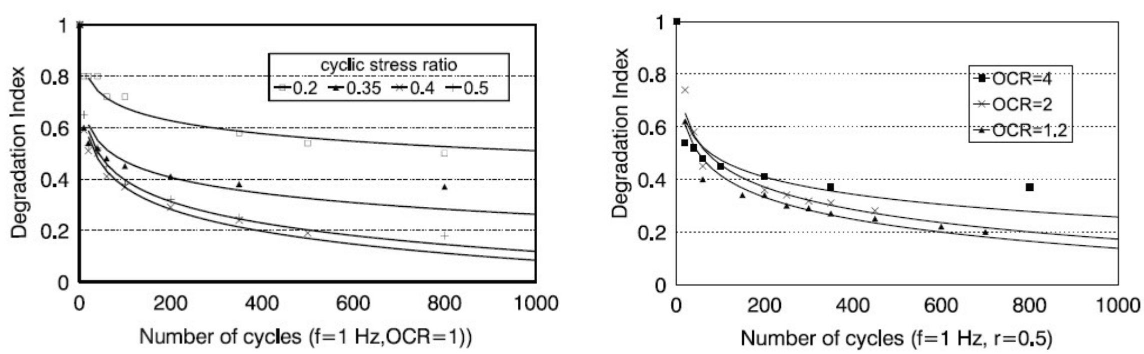


Figure 2.61: Effect of cyclic stress ratio and overconsolidation ratio on degradation index (after Zhou and Gong [75]).

Frequency affects in normally consolidated clay diminished with an increasing number of cycles and with a decreasing shear stress amplitude. In addition, the degree of degradation was high for low loading frequencies; if the loading frequency increases the degree of degradation will be lower. When the frequency of loading was less than 0.1 Hz, the sample degraded quickly even for a small number of cycles (Fig. 2.62).

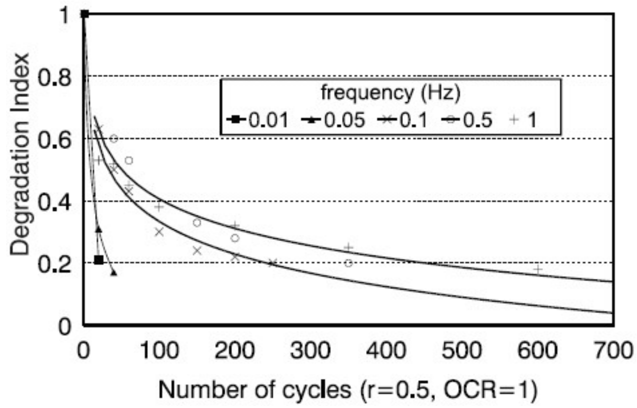


Figure 2.62: Effect of frequency on the degradation index (after Zhou and Gong [75]).

Chapter 3

General Behavioral Trends for Cyclically Loaded Cohesive Soils

This chapter summarizes the most important findings of the experimental work on cyclically loaded cohesive soils that was presented in Chapter 2. It rather significantly expands the previous review of this subject by Lee and Focht [35], and summarizes the general behavioral trends observed for cyclically loaded cohesive soils.

3.1 Cyclic Thresholds

In one of the earliest experimental investigations of the cyclic response of cohesive soils, Larew and Leonards [31] observed the existence of limiting cyclic stress values for compacted soil samples. For stress levels below such a limit, a cyclically loaded sample does not accumulate inelastic strain, nor does it suffer any degradation in stiffness. In addition, under undrained conditions, no excess pore pressure is accumulated. In short, the sample exhibits essentially elastic response. If the cyclic stress amplitude exceeds the aforementioned limiting value, the sample exhibits inelastic response, possibly leading to failure.

Sangrey [48] performed cyclic triaxial tests at very low frequencies so as to ensure reliable excess pore pressure measurements. Similar to Larew and Leonards [31], Sangrey [48] observed the existence of a critical stress level beneath which the soil behaved elastically when subjected to repeated loading. For isotropically normally consolidated clay, the critical stress level was about *two-thirds* of the maximum undrained strength as determined in a monotonic load test. The response of anisotropically consolidated and overconsolidated samples was quite similar to that for normally consolidated ones. It was thus concluded that a critical stress level exists for all initial stress conditions. Figures 3.1 and 3.2 show results for deviatoric cyclic stress levels above and below the critical stress level, respectively. When the cyclic deviator stress is approximately 50% of the initial undrained shear strength, it is evident that the sample reaches an equilibrium state that is characterized by a closed hysteresis loop. There is no more plastic or residual strain and the excess pore pressure increases during loading but decreases during unloading. The sample did not fail, though as many as one-hundred cycles were applied.

Sangrey [48] also found that for soils that are initially at the same state of stress but at different degrees of overconsolidation, the critical cyclic stress level will be higher with increased OCR. The magnitude of the pore pressure build-up was dependent on the consolidation history and on the level of repeated stress.

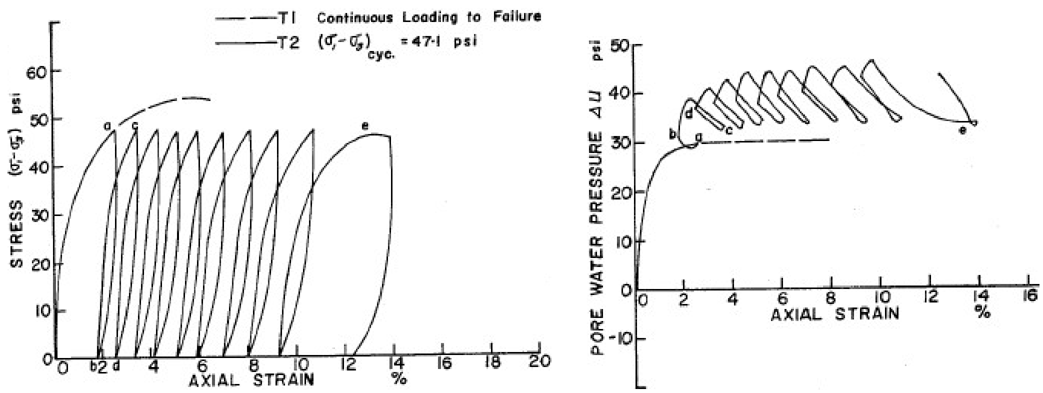


Figure 3.1: Stress-Strain curve and pore pressure evolution, $\sigma_1 - \sigma_3 = 330kPa$ (after [48]).

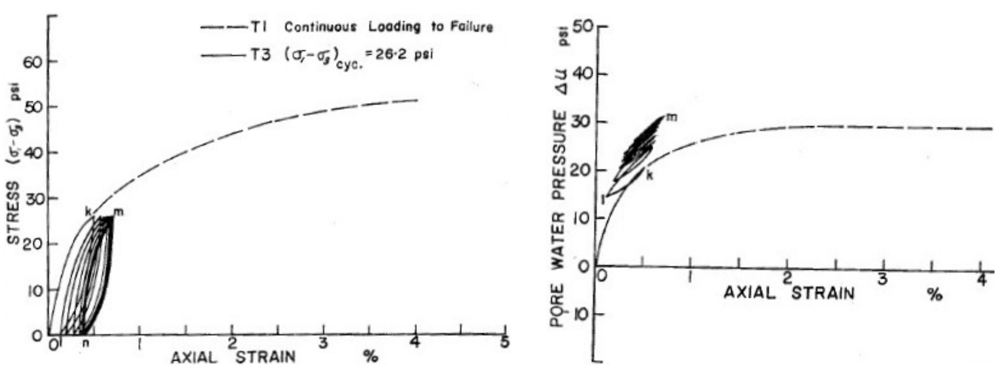


Figure 3.2: Stress-Strain curve and pore pressure evolution, $\sigma_1 - \sigma_3 = 180kPa$ (after [48]).

Several investigators ([63; 1; 40; 2; 9; 3; 75; 72; 16]) subsequently confirmed the existence of limiting cyclic stresses, which were typically referred to as "threshold cyclic stresses." It was also determined that, for the same soil, the threshold cyclic stress level increases with OCR [1]. Generally, stabilization in normally consolidated samples requires fewer numbers of cycles than in overconsolidated ones [40].

For normally consolidated, as well as overconsolidated samples, the threshold stress level was found to be about two-thirds of the maximum undrained shear strength as determined in a monotonic load test [48]. This finding holds for both isotropically and anisotropically consolidated samples.

At high cyclic stress levels, after a certain number of cycles, the residual strain accelerates towards failure. This trend has been observed in both undisturbed and remolded cohesive soils [2; 11]. Comparing test results with the same cyclic deviator stress amplitude, overconsolidated samples tend to failure after a smaller number of cycles as compared to normally consolidated ones. Also, the number of cycles to failure decreases with increasing the cyclic deviator stress for both normally and overconsolidated samples [1]. For normally consolidated samples, failure occurs by reduction of the effective stresses. However, for overconsolidated samples, there is practically no evolution of excess pore pressure during cyclic loading, even at failure [40].

In cemented cohesive soils, the threshold condition is governed by the cementation strength [42; 43]. Consequently, cementation bonds significantly influence the response. At sufficiently high cyclic stress levels, such bonds are broken, leading to the development of inelastic strains and excess pore pressure.

The limiting cyclic response can also be defined by the amplitude of the cyclic shear strain. Thus, if the cyclic strain is less than some threshold value, only negligible inelastic strains will accumulate, and excess pore pressure will increase, even with a large number of loading cycles [44; 37; 75; 8]. In general, the threshold cyclic strain tends to increase with plasticity index, albeit sometimes only marginally.

More recently, different limits or thresholds have been proposed based on specific soil response [3; 45; 10; 55]. For example, the "elastic threshold" corresponds to small cyclic strain amplitudes [need ref!!]. Below this threshold, the degradation of the dynamic and post-cyclic engineering properties would be expected to be negligible. Once the elastic threshold is exceeded, however, the induced cyclic shear strain increases and the material tends to exhibit strain softening, stiffness degradation, inelastic strain accumulation, and increases in excess pore pressure. This response continues up to the so-called "plastic threshold". If this limit is exceeded, the soil will reach large strain amplitudes due to the accumulation of inelastic strains and to stiffness degradation. The final critical strain level is referred to as the "flow threshold"; at this level the soil has reached a steady state condition and behaves as a viscoplastic material up to failure.

In a relatively few cases the presence of a cyclic threshold stress was either not present, or not fully realized. In particular, Brown et al. [6] did *not* observe the existence of a threshold stress. In general, strain continued to build up even after 10^6 stress cycles. An apparent failure condition was only obtained for the highest cyclic stress levels on overconsolidated ($OCR = 4$, 10 and 20) samples. The highest cyclic stress levels were 93%, 94%, 82% of the monotonic undrained shear strength, respectively. However, in order to put these test results into perspective, it is important to note that the aforementioned tests consisted of one-way cyclic loading at a frequency of $10\,Hz$. Consequently, there was not enough time for the soil microfabric to re-orient itself, thus precluding any stiffness degradation for high cyclic deviator stress levels. From Figure 3.3 it is evident that, for low cyclic deviator stress levels, the strain accumulation between 10^2 and 10^3 cycles was not overly large. This, in turn, may imply the existence of an equilibrium state. It is evident, however, that if cyclic loading continues past 10^3 cycles (even up to 10^6 cycles), the accumulation of inelastic strain will continue. As such, the existence of an equilibrium state is somewhat questionable. It is timely to note that the high frequency and large number of loading cycles are more applicable to pavement engineering rather than geotechnical engineering.

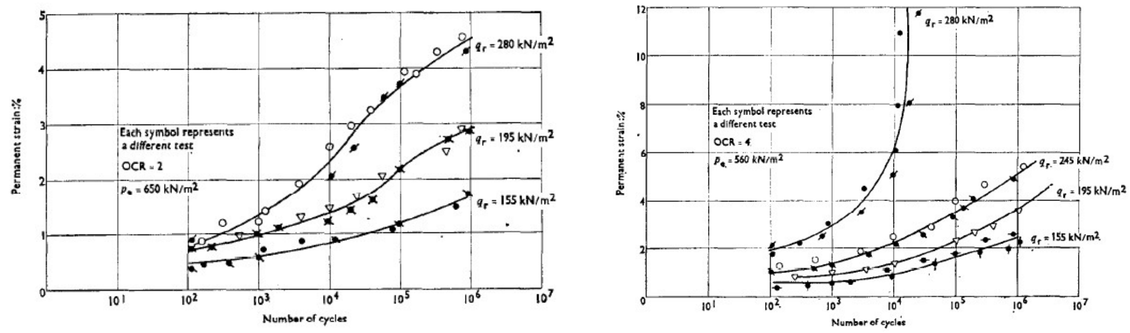


Figure 3.3: Permanent strain during cyclic loading, [6]

The results presented by Zergoun [73] and Zergoun and Vaid [74] appear to indicate the lack of a true equilibrium state. In particular, for stress levels below 55% of the monotonic undrained shear strength, both residual inelastic strains and excess pore pressures tended toward an equilibrium state with an increasing number of cycles. True equilibrium did not, however, appear to have been reached, as residual strains and excess pore pressures continued to increase, albeit at a decreasing rate (Figure 3.4). For higher cyclic stress levels, the residual strain rate started to accelerate after a given number of cycles; this eventually led to failure of the sample. In such cases, the accumulated strain versus number of cycles response exhibits three rather distinct phases: first, strain develops at a decreasing rate per cycle; next it develops at a fairly constant rate per cycle; finally, the rate of accumulation accelerates, ending in failure[1]. Finally, the stress-strain hysteresis loops associated with the the work of Zergoun [73] and Zergoun and Vaid [74] are noteworthy. At lower cyclic stress levels, the stress-strain loops change very little, resulting in low hysteresis (Figure 3.5a). On the other hand, for high cyclic stress levels the stress-strain loops increase in size, indicating more significant hysteresis and drift towards extensional stress states and stiffness degeneration (Figure 3.5b).

[1]This response is similar in form to the strain versus time response associated with undrained creep of cohesive soils. In the latter case the three phases of response are referred to as the *primary*, *secondary* and *tertiary* phases of creep.

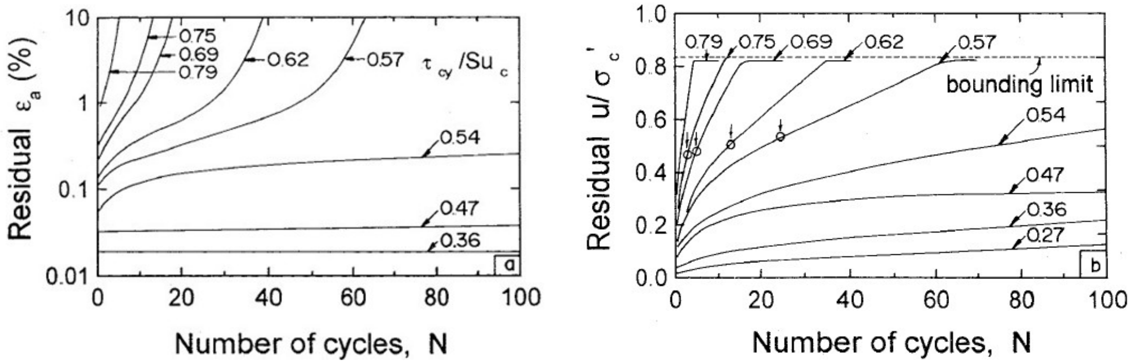


Figure 3.4: Development of residual strain and excess pore pressure in response to cyclic loading (after [74]).

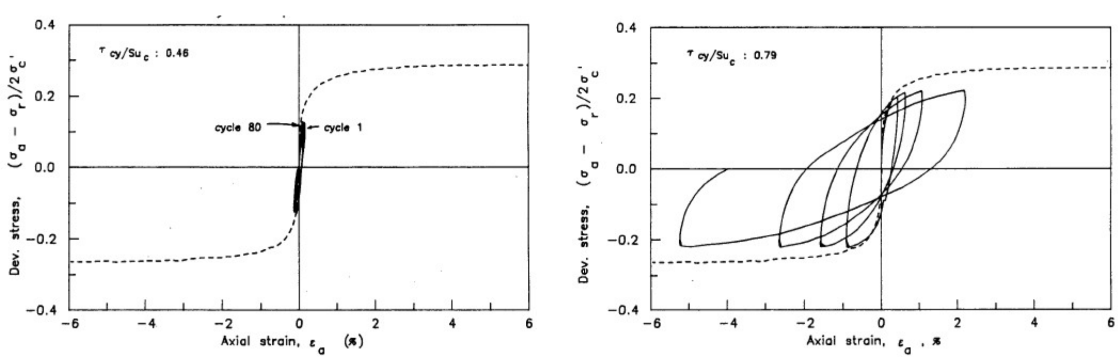


Figure 3.5: Stress-strain hysteresis loops for two different cyclic stress levels (after [73]).

3.2 Effect of Loading Conditions

In terms of loading conditions, both excess pore pressure and inelastic strain develop more rapidly under two-way (reversal) cyclic loading than under one-way (non-reversal) loading. As shown in Figure 3.6, the cyclic strength under two-way loading is slightly smaller than for two-way loading. In addition, the amount of stiffness degradation is higher for the case of two-way loading.

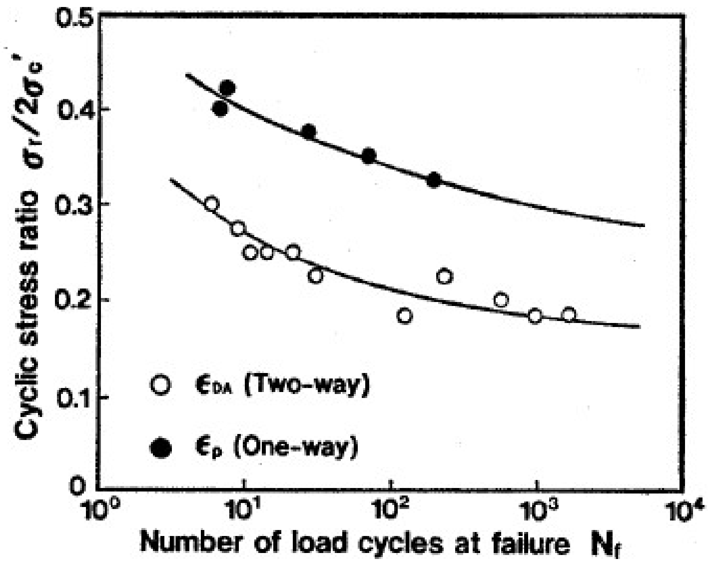


Figure 3.6: Cyclic response under different loading conditions and consolidation states: Effect of loading type (after [69]).

The effect of the magnitude of the initial deviator stress on the cyclic response has been rather well-studied [51; 27; 46; 22; 28; 23; 33; 24; 19]. The higher the initial deviator stress (i.e., there higher the initial degree of anisotropy), the fewer the number of cycles that are required to reach failure, and the lower the cyclic strength (Figure 3.7). Due to the fact that in some cases the reversal condition disappears for high levels of initial deviator stress, the rate of stiffness degradation in anisotropically consolidated soils is lower than for isotropically consolidated ones. In the latter case any cyclic deviator stress magnitude will involve reversed loading, thus accelerating the rate of stiffness degradation.

To evaluate the effect of an initial deviator stress, Hyodo et al. [22, 23] carried out cyclic triaxial tests on anisotropically consolidated soil under different initial deviator stress values (q_s). It was found that the cyclic shear strength decreases with increasing the initial static deviator stress as shown in Figure 3.8. Hyodo et al. [22, 23] also compared the behavior between Itsukaichi clay and Toyoura sand; they determined that, when subjected to initial static shear stress, the clay is more unstable than sand during cyclic loading.

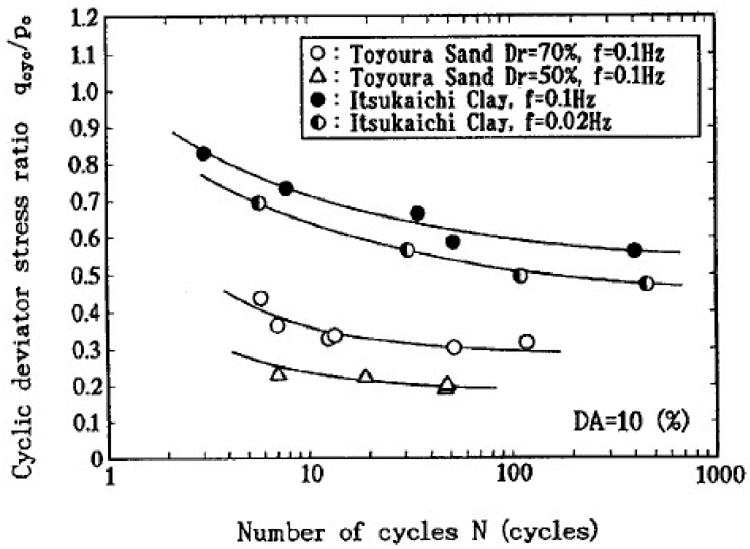


Figure 3.7: Effect of plasticity index on cyclic strength for normally consolidate clays (after [22]).

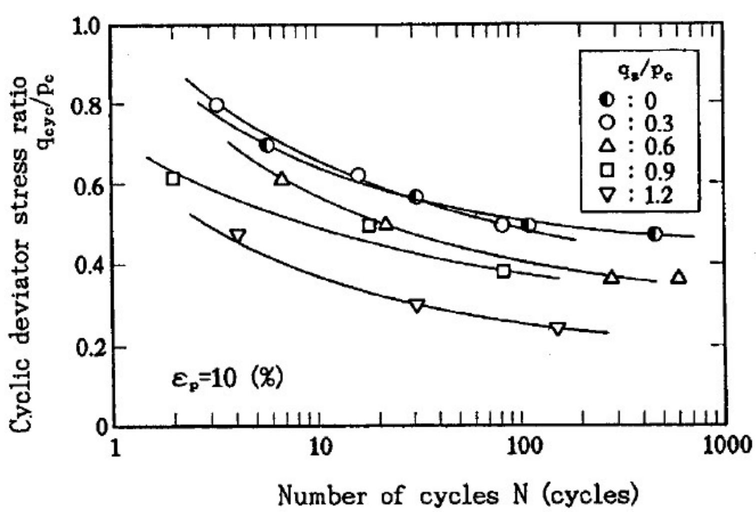


Figure 3.8: Cyclic response under different loading conditions and consolidation states: Effect of magnitude of initial deviator stress (after [22]).

Finally, as shown in Figure 3.9, isotropically consolidated samples tend to develop higher excess pore pressures as compared to ones consolidated anisotropically [68].

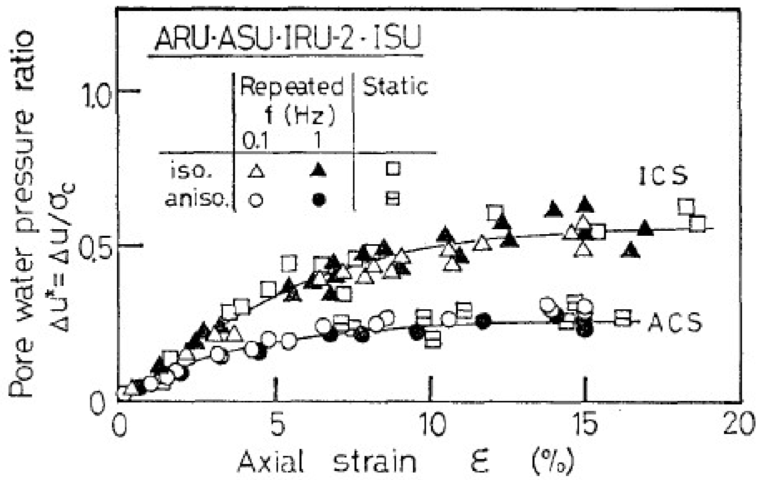


Figure 3.9: Cyclic response under different loading conditions and consolidation states: Effect of initial consolidation state (after [68]).

3.3 Effect of Loading Frequency

In general, loading frequency effects tend to diminish with increasing number of cycles and with decreasing cyclic deviator stress amplitude. The influence of frequency appears to be significant if relatively small numbers of cycles are considered. When loaded at high frequencies, cohesive soils exhibit higher cyclic shear strength then when loaded at lower frequencies. In general, for a given number of cycles, larger shear strains and excess pore pressures are generated at lower frequencies [27; 38; 47; 32; 2; 28; 33; 75; 36; 41; 62].

[38] evaluated the effect of loading frequency on cyclic response of normally consolidated and overconsolidated clays. For a given number of cycles, higher excess pore pressures and axial strains were generated at lower frequencies (See Figure 3.10). In addition, the effect of frequency on both excess pore pressure and axial strain is quite similar. In terms of the overconsolidation ratio, [38] determined that the greater the OCR, the greater the negative residual pore pressure after a small number of load repetitions, and the smaller the amount of positive residual pore pressure after a large number of cycles. For $OCR \geq 2$, the positive residual pore pressure is negligible within 10^4 cycles (See Figure 3.11). Thus, clay with high OCR will have more resistance against subsequent excess pore pressure generation.

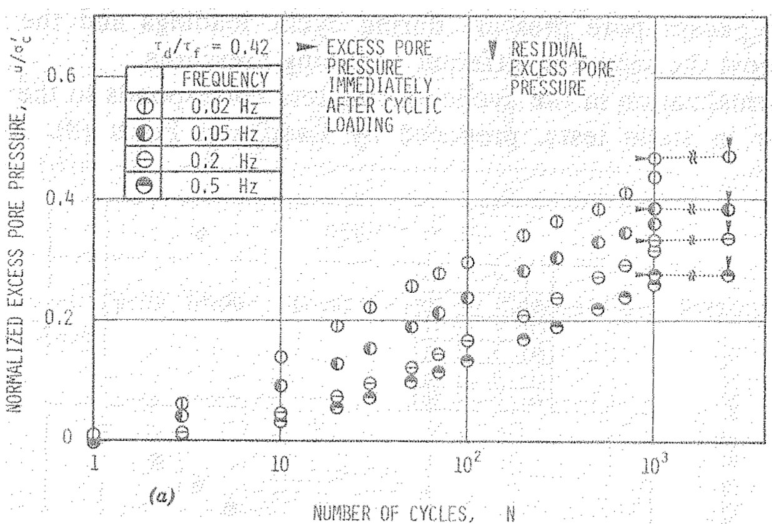


Figure 3.10: Effect of loading frequency and overconsolidation ratio on excess pore pressure (after [38]).

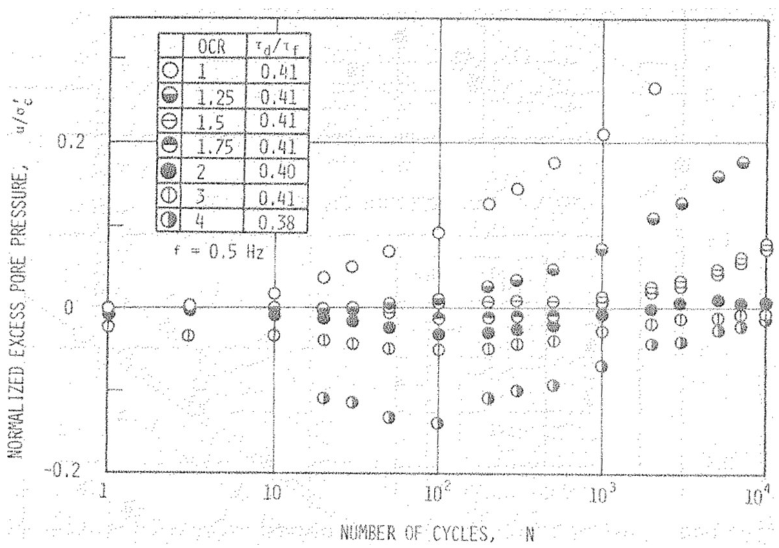


Figure 3.11: Effect of loading frequency and overconsolidation ratio on excess pore pressure (after [38]).

Ansal and Erken [2] observed that the frequency effect in normally consolidated clay diminishes with an increasing number of cycles and with decreasing shear stress amplitude. Similarly, Zhou and Gong [75] evaluated the effect of loading frequency on normally consolidated and overconsolidated samples. Similar to Matsui et al. [38], they found that the degree of stiffness degradation is high for low loading frequencies; for higher frequencies the degree of degradation will be lower. When the frequency of loading is less than 0.1 Hz, the sample will degrade quickly even for a small number of cycles (Figure 3.12).

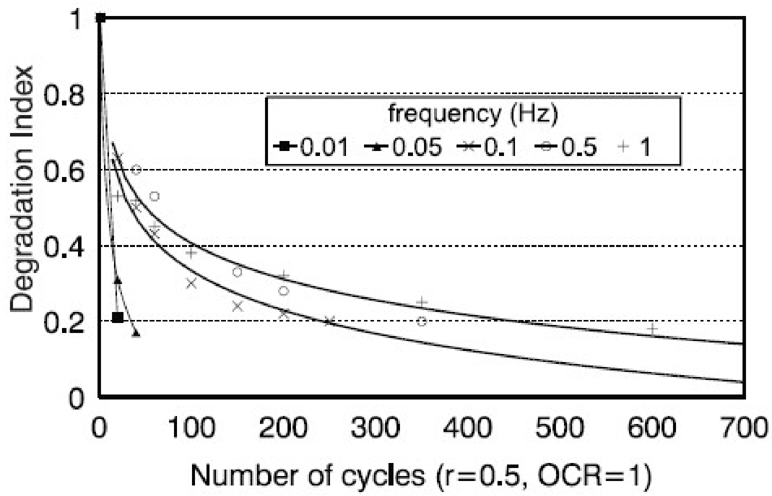
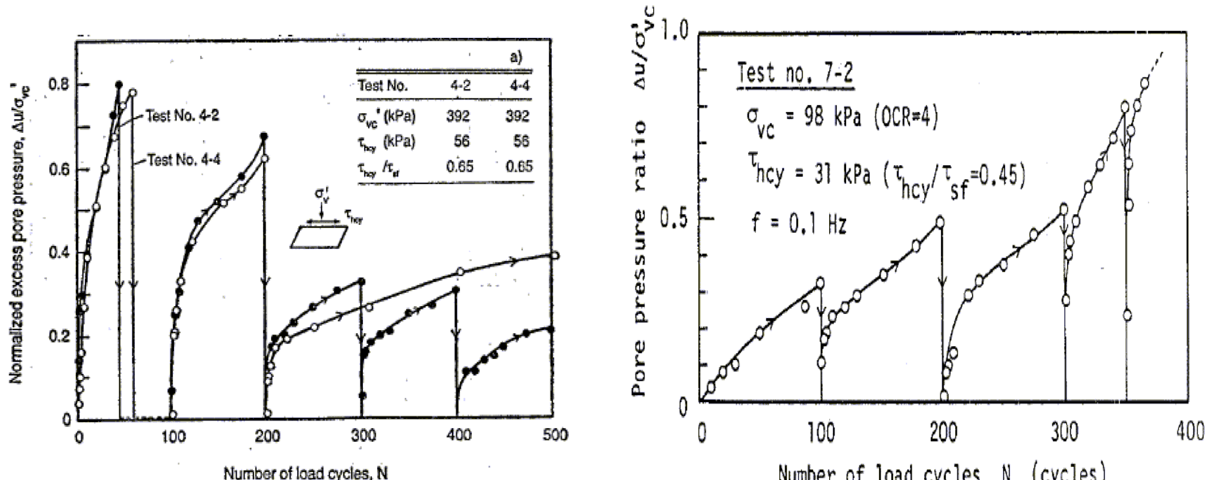


Figure 3.12: Effect of frequency on the degradation index (after [75]).

3.4 Effect of Drainage Period

Cyclic tests have been performed on normally consolidated [65; 71] and overconsolidated samples [66] so as to investigate the recompression or settlement of cohesive soils following consecutive cyclic loading events. In such tests, undrained cyclic events were separated by periods of time to allow for the dissipation of excess pore pressure under drained conditions.

Results from such "partially drained" tests on normally consolidated samples indicate that the excess pore pressure decreases for each cyclic loading event (Figure 3.13a). During the subsequent periods of pore pressure dissipation, the void ratio continuously decreases. Tests performed on overconsolidated samples showed opposite tendencies. In particular, the excess pore pressure increases during each loading event (Figure 3.13b); during the subsequent dissipation phases, the void ratio increases. Thus, cyclic loading events followed by periods of drainage can have a detrimental effect on the response of over consolidated soils since they will have lower resistance to subsequent undrained cyclic loading events.



a) b)

Figure 3.13: Pore water pressure during cyclic loading with drainage periods, [65; 66].

A rather limited number of "partially drained" tests has been performed [12; 20; 21; 16] to investigate the volumetric strains generated during cyclic loading. In normally consolidated samples, the dissipation of excess positive pore pressure leads to a decrease in water content and void ratio after each cycle. This, in turn, results in an increase in the shear strength. By contrast, the partially drained loading of overconsolidated samples is characterized by negative residual excess pore pressures that remain after the first undrained load-unload cycle. As such, under drained conditions water is drawn into such samples. The process continues for subsequent loading cycles, though a smaller negative residual pore pressure is developed with each subsequent cycle. Nonetheless, if cyclic loading continues, an overconsolidated sample will draw in sufficient water as to reduce its shear strength. Finally, as compared to samples tested under fully undrained conditions, partially drained ones require a greater number of cycles and higher deviator stress levels to reach failure.

3.5 Undrained Stress Paths

In cyclically loaded normally consolidated samples tested under undrained conditions, the effective stress paths migrate towards the origin of the stress space. The net associated pore pressure will thus be increasingly positive. Since the material continues to harden, the soil undergoes an apparent overconsolidation similar to that exhibited by normally consolidated cohesive soils during undrained creep.

In cyclically loaded overconsolidated samples, the effective stress path tends to initially proceed away from the stress origin; subsequently, however, it reverses its direction towards the origin. Figure 3.14 shows how the effective mean stress stress first increases and then decreases for overconsolidated samples (in this figure the symbol n denotes the OCR.). Although the development of axial strain accelerates for overconsolidated samples, the rate of excess pore pressure development progressively diminishes.

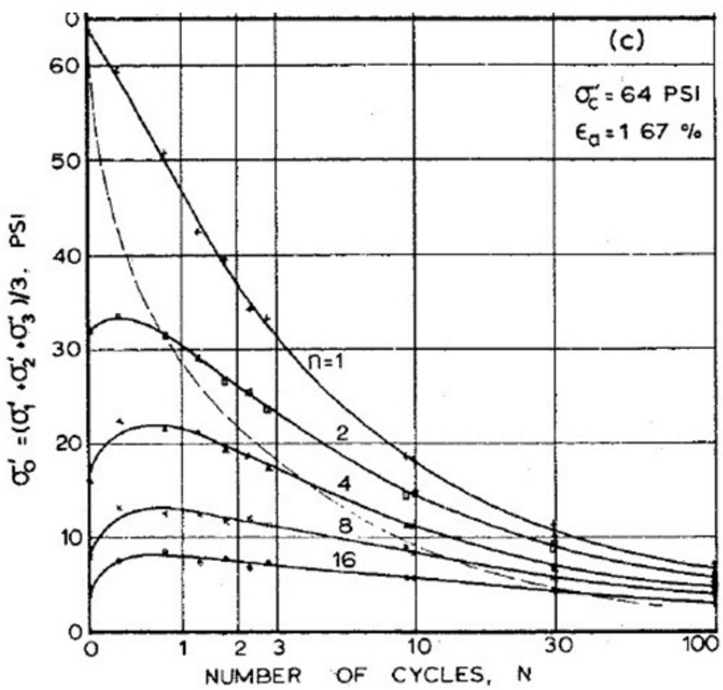


Figure 3.14: Decrease in mean effective principal stress during cyclic test (after [56]).

Figure 3.15 shows the envelopes of peak compressive shear strength for different strain amplitudes and overconsolidation ratios. It is evident that the higher the cyclic strain amplitude, the higher is the rate of decrease of the peak shear strength. For example, for $\varepsilon = 0.14\%$ the envelope is almost horizontal, thus indicating that the strength does not decrease significantly with the number of cycles; this is known as the equilibrium state. The rate of decrease in shear strength increases with OCR.

The rate of migration of the effective stress path towards the stress origin depends on the cyclic stress/strain magnitude, the consolidation history, and on the frequency of load-

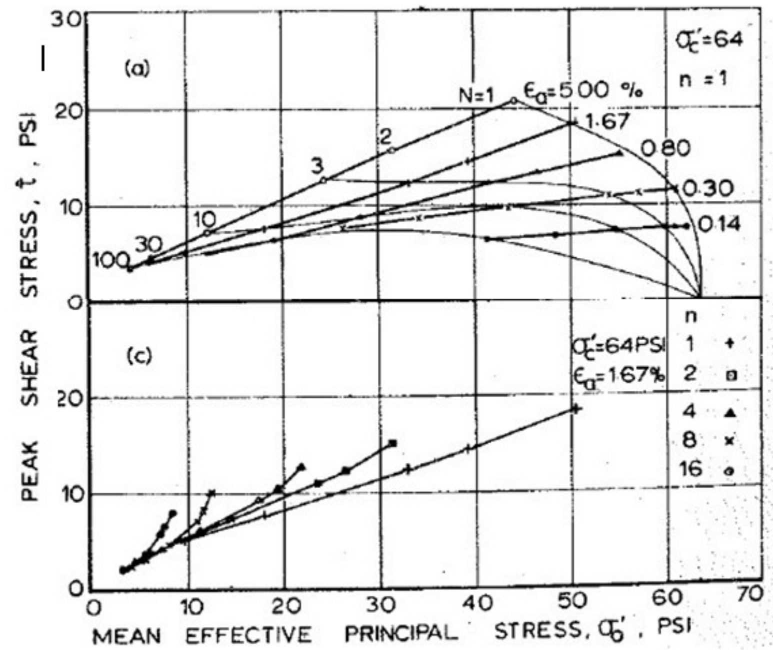


Figure 3.15: Envelopes of compression peak shear strength for different strain amplitudes and OCR values (after [56]).

ing. In particular, normally consolidated samples migrate faster than overconsolidated ones. Samples loaded at low frequencies generate higher pore pressures; their migration towards the stress origin is thus more rapid than for ones loaded at higher frequencies [56; 54; 44].

3.6 Pore Pressure Response

Closely related to the issue of undrained stress paths is the generation of excess pore pressure during undrained cyclic loading. As noted in Section 3.5, loading of normally consolidated samples generates positive excess pore pressure. upon unloading, the excess pore pressure would be expected to continuously reduce in magnitude. Certain experimental results do not, however, show this to always be the case. For example, Sangrey [48] performed one-way (non-reversal) cyclic tests on undisturbed samples of Newfield clay ($I_p = 10$). To ensure equalization of the excess pore pressure, an axial strain rate of approximately 0.0002% per minute was used (each loading-unloading cycle was thus approximately 10 hours in duration). Sangrey [48] found that during the unloading phase the excess pore pressure initially decreased. This trend reversed itself rather quickly, however, as the excess pore pressure again became positive (Figure 3.16a). This behavior was likewise observed for overconsolidated samples (Figure 3.16b).

The above pore pressure response will occur provided the magnitude of the cyclic deviator stress (q_{cyc}) exceeds the threshold stress. If q_{cyc} is less than the threshold, the excess pore pressure increases during loading (i.e., during increases in q_{cyc}) and decreases upon unloading. An equilibrium state is thus reached. Figure 3.17 shows the response of two samples

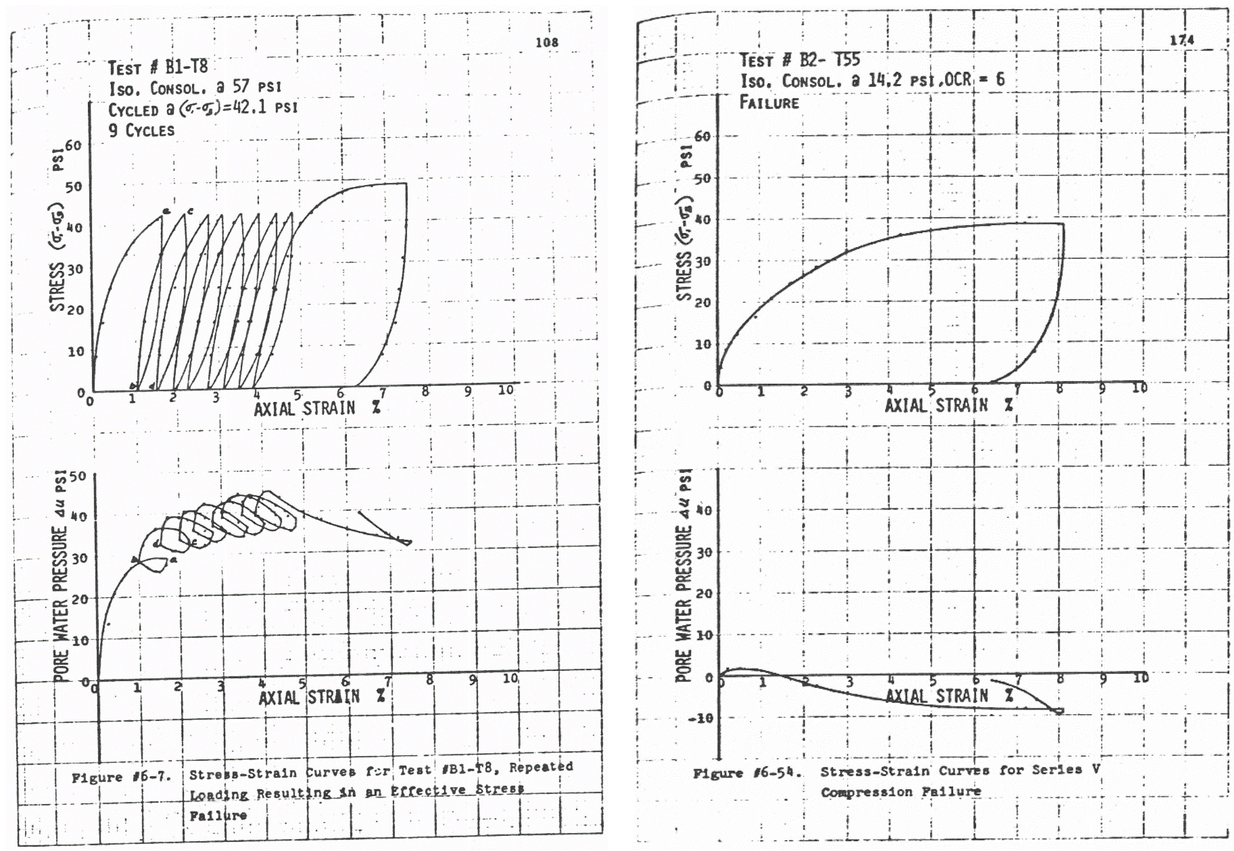


Figure 3.16: Stress-strain-pore pressure with large q_{cyc} for a) $OCR = 1$ and b) $OCR = 6$ (after [48]).

that were both loaded to q_{cyc} levels below the threshold value. With an increasing number of lading cycles, the excess pore pressure is seen to increase upon loading and to then decrease upon unloading. The resulting deviator stress-strain loops are essentially closed, indicating very little hysteresis and thus largely elastic response.

In a related experimental program, Sheu [52] performed one-way (non-reversal) cyclic tests on reconstituted samples of Georgia kaolin clay ($I_p = 20$). To ensure that the desired equalization in excess pore pressure, a loading frequency of $0.001\,Hz$ was used in these tests. Depending on the cyclic deviator stress level, three different types of response were noted. For relatively low values of q_{cyc}, the excess pore pressure generated during loading differed in magnitude and sign from that generated during unloading (Figure 3.18a). For higher levels of q_{cyc} this response changed. Now, the excess pore pressure first decreased but then increased during unloading (Figure 3.18b). Finally, for even higher levels of q_{cyc}, the excess pore pressure increased even more significantly during unloading. As pointed out by Sheu [52], such pore pressure response significantly complicates the mathematical modeling and numerical simulation of cohesive soils subjected to cyclic loading.

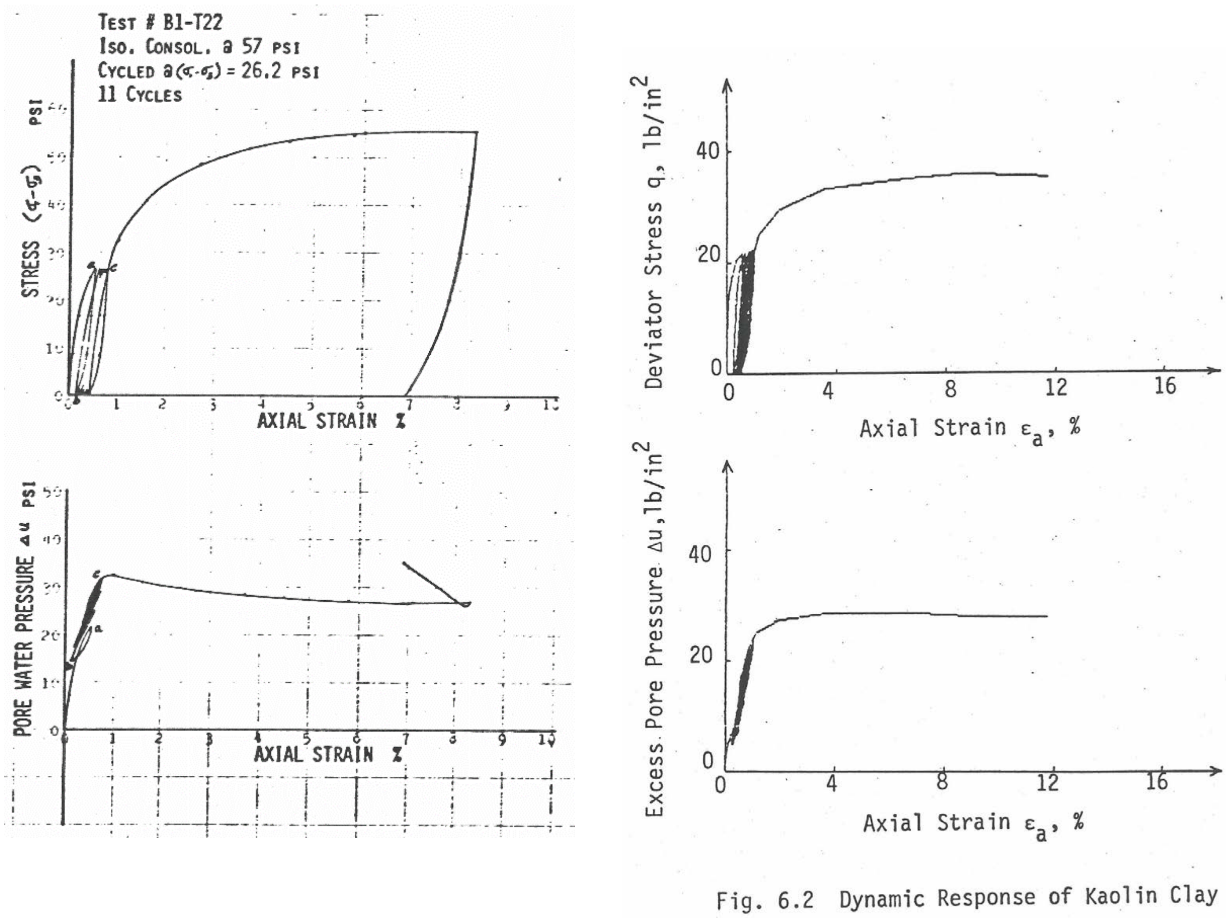


Figure 3.17: Stress-strain-pore pressure response with small q_{cyc} (after [48] and [52]).

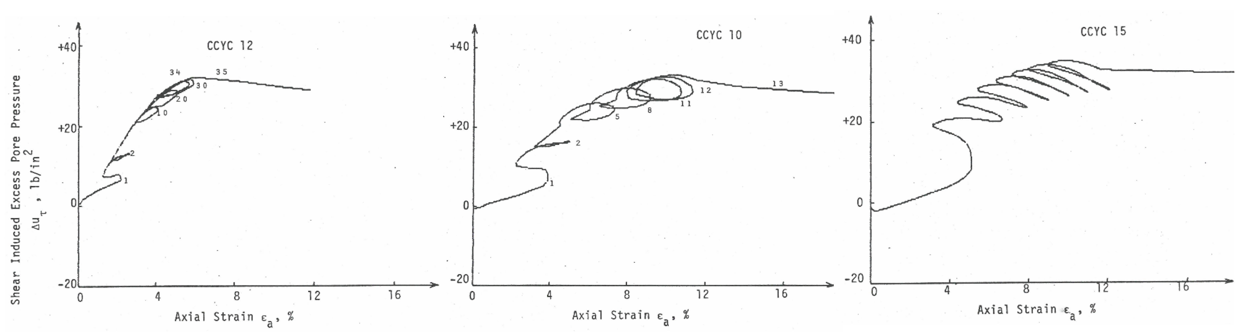


Figure 3.18: Stress-strain-pore pressure with increasing q_{cyc} (after [52]).

In closing this section it is timely to note that the aforementioned excess pore pressure response was observed for samples that were loaded very slowly. It is thus quite likely that time effects were active[2] , particularly during the loading phases. Consequently, such effects should not be ignored in the modeling and numerical simulation of cohesive soils subjected to cyclic loading.

3.7 Stiffness Degradation

Cyclically loaded cohesive soils may exhibit stiffness degradation [57; 25; 26; 27; 59; 58; 75; 34]. This characteristic is rather strongly dependent on the level of cyclic stress/strain applied to the sample. Thus, cyclic loading that induces large strains tends to produce appreciable stiffness degradation. In general, substantial degradation typically occurs during the first few loading cycles.

Thiers and Seed [57] carried out one of the first experimental studies of soil stiffness degradation. They performed cyclic direct simple shear tests under strain-controlled conditions. The elastic shear modulus G was found to decrease by approximately 50% to 80% as the peak strain level increased from 0.5% to 2%. However, the change in G was negligible for strains above 2%. In general, the largest changes in G took place during the first 50 loading cycles.

To study the degradation in soil stiffness, Idriss et al. [25] performed both stress and strain-controlled cyclic triaxial tests on normally consolidated clay samples. The degradation was found to be strongly dependent on the cyclic strain level applied to the sample. Thus, cyclic loading with high strain levels substantially degraded the soil stiffness. In addition, increases in the cyclic strain amplitude tended to increase the area in the hysteresis loops for the first cycle of loading, and to flatten out these loops. For small values ($\leq 0.75\%$) of applied strain the loops were essentially symmetric; for larger cyclic strain amplitudes these loops were no longer symmetric but instead approached an “S-shape”.

Based on the results of their cyclic tests, Andersen et al. [1] also concluded that G decreased with increasing number of cycles. In addition, the higher the cyclic stress level, the more pronounced was the decrease in G. Finally, at low cyclic stress levels the stiffness degradation was negligible; G was thus independent of the number of cycles.

Comparing test results at the same cyclic deviator stress amplitude, Andersen et al. [1] found that overconsolidated samples reached failure after a smaller number of cycles as compared to normally consolidated ones. Also, for both normally and overconsolidated samples the number of cycles to failure decreased with increasing cyclic deviator stress. Meimon and Hicher [40] found that for normally consolidated samples, failure occurred by reduction

[2]During both the loading and unloading phases the total stress level is changing; true creep conditions are thus never realized. In addition, during both loading and unloading phases the sample deforms; true relaxation conditions are thus never realized. Consequently, the time-related response is likely general in nature.

in the effective stresses. However, for overconsolidated samples, there was practically no evolution of the pore pressure during cyclic loading, even at failure.

Vucetic and Dobry [59] also evaluated soil stiffness degradation of anisotropically (K_0) normally consolidated and overconsolidated clay using cyclic direct simple shear tests. Degradation of the undrained secant shear modulus during cyclic loading was quantified by the so-called "degradation index" δ, which is defined as follows:

$$\delta = \frac{G_{sN}}{G_{s1}} \tag{3.1}$$

where G_{sN} and G_{1N} are the secant shear modulus after N cycles and first cycle at constant shear strain amplitude, respectively. Small δ values correspond to high degree of stiffness degradation.

As seen from Figure 3.19, δ is lower for normally consolidated samples, meaning that G degrades more and faster than in the case of overconsolidated samples. In addition, the controlled cyclic shear strain amplitude (γ_c) increases with OCR (Figure 3.19). Yasuhara et al. [70] also observed such response.

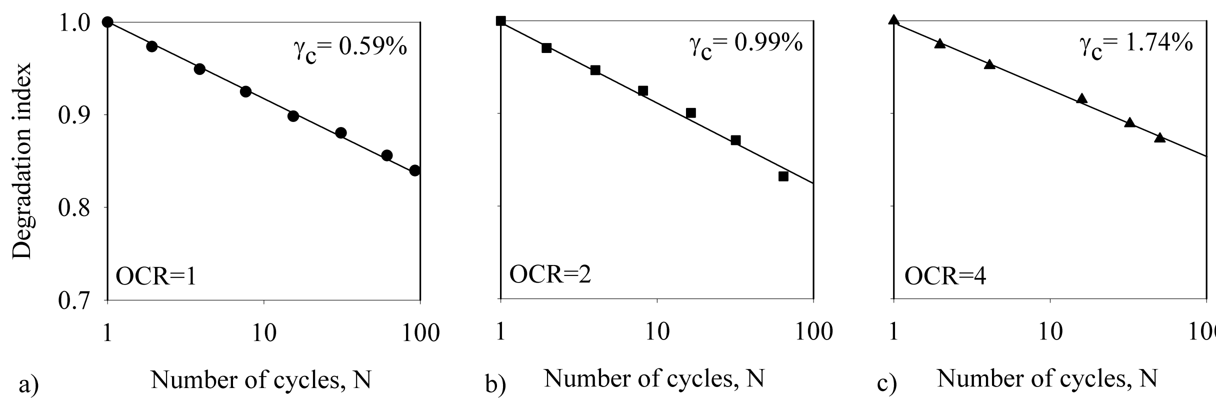


Figure 3.19: Degradation index versus number of cycles for different OCR values (after [59]).

Zhou and Gong [75] also quantified stiffness degradation through δ. They showed that soil degradation was directly related to the cyclic stress ratio(Figure 3.20a). Comparing the δ at the same number of cycles, it was found to be smaller with increasing the cyclic stress ratio, which implies more soil degradation. In addition, δ was lower for normally consolidated samples, which confirmed that stiffness in such samples degrades more and faster than in overconsolidated ones. This work suggested that the OCR was an important factor in the study of soil degradation.

Okur and Ansal [45] also investigated the issue of stiffness degradation due to cyclic loading. Figure 3.21 shows the variation of the dynamic shear modulus ratio and damping ratio against cyclic shear strain amplitude for different cycles for one particular test. The

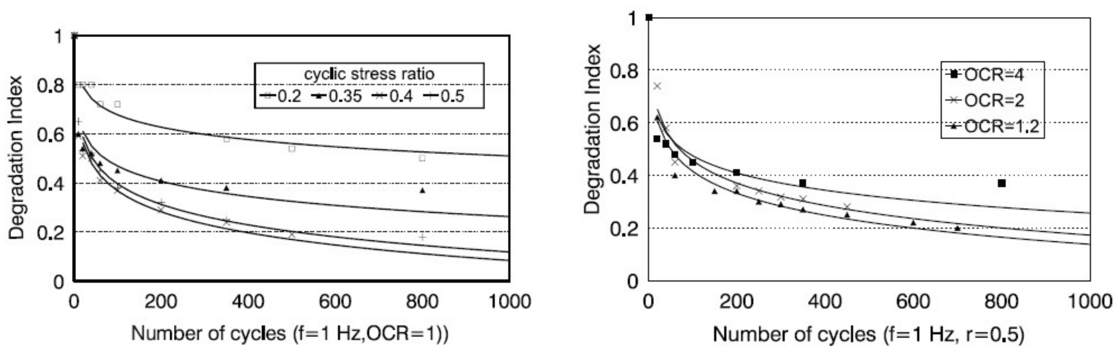


Figure 3.20: Effect of cyclic stress ratio and overconsolidation ratio on degradation index (after [75]).

shape of the hysteresis loops corresponding to cycles 10 to 15 have essentially the same area, which indicates nonlinear elastic soil response. The reduction of stiffness becomes noticeable once a critical cyclic shear strain level, defined as the “elastic threshold”, was exceeded. A second critical strain level, the so-called “flow threshold”, was also defined. Once the soil sample exceeds this stress level, it starts to behave as a viscoplastic material. The elastic and flow strain thresholds were found to be around $\gamma_E = 0.0046$ and $\gamma_P = 0.38$, respectively (See Figure 3.21). This curve is, of course, also a function of the plasticity index.

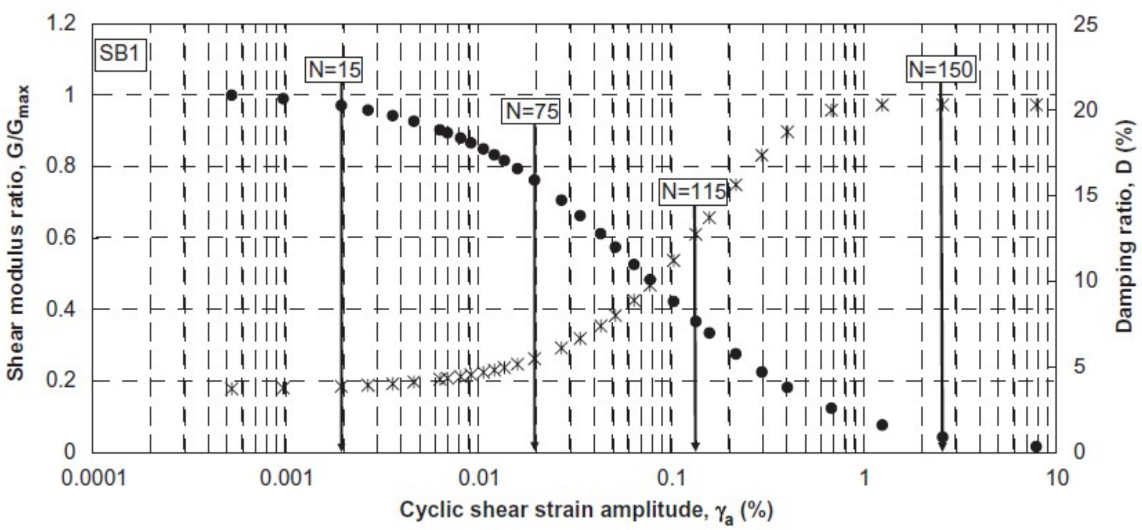


Fig. 10. Variation of shear modulus and damping ratio for different cycles.

Figure 3.21: Variation of shear modulus and damping ratio for different cycles, $G_{max} = 34\,MPa$ (after [45]).

For both normally and overconsolidated samples, G decreases with increasing numbers of cycles. In addition, the higher the deviator cyclic stress level, the more pronounced is the decrease in G.

3.8 Role of Plasticity Index

The plasticity index appears to be a key indicator of the response of cohesive soils subjected to cyclic loading. In particular, as the plasticity index increases from non-plastic (NP) to highly plastic soils, the cyclic shear strength likewise increases (Figure 3.22). Micromechanically this is explained by the fact that clays with higher values of plasticity index have larger values of specific surface (i.e., the ratio of the surface area per mass of dry soil particles). Thus, such clays have a greater potential for attracting neighboring particles (to often form particle clusters). This, in turn, increases the resistance of the clay to cyclic loading.

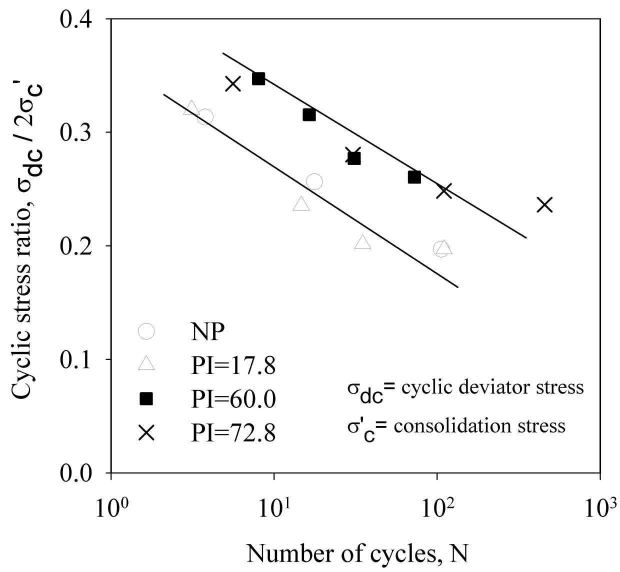


Figure 3.22: Effect of plasticity index on cyclic strength in normally consolidated clays (after [24]).

Hyodo et al. [24] investigated the effect of the initial (drained) shear stress on the cyclic shear strength of various types of soils. In general, samples with initial shear stress failed as a result of increases in the peak axial strain. In the case of low plasticity clay, the increase in initial shear stress caused a slight increase in the cyclic shear strength. By contrast, for high plasticity clays the strength decreased markedly with increases in the initial shear stress. Figure 3.23 shows the cyclic deviator stress ratio required to cause failure (defined in terms of the peak axial strain) after 20 cycles against the initial (drained) stress ratio. It is evident that for clays the cyclic strength decreases at higher initial shear stress ratios. For sands, on the other hand, liquefaction is more critical for low initial shear stress ratios. In addition, the response of non-plastic clay was found to be similar to that of the sand.

In order to evaluate the the effect of the overconsolidation ratio on such response, Hyodo et al. [24] performed triaxial tests on normally consolidated and overconsolidated samples. Based on the results shown in Figure 3.24, the cyclic strength seems to increase with the OCR.

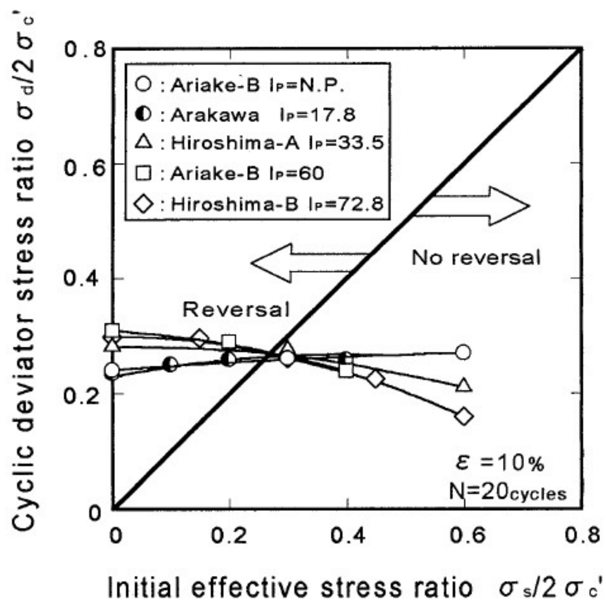


Figure 3.23: Variation with plasticity of the cyclic strength with initial drained shear stress (after [24]).

The threshold or critical strain level tends to increase with PI [refs??]. In addition, experimental results [60; 24; 3; 11; 45; 5; 62] confirm the fact that the PI is one of the key indicators of the degradation of dynamic shear modulus during cyclic loading (recall the discussion of Section 3.7). Finally, the relationship between plasticity and cyclic response has been found to be independent of the pore fluid chemistry in fine-grained soils [15].

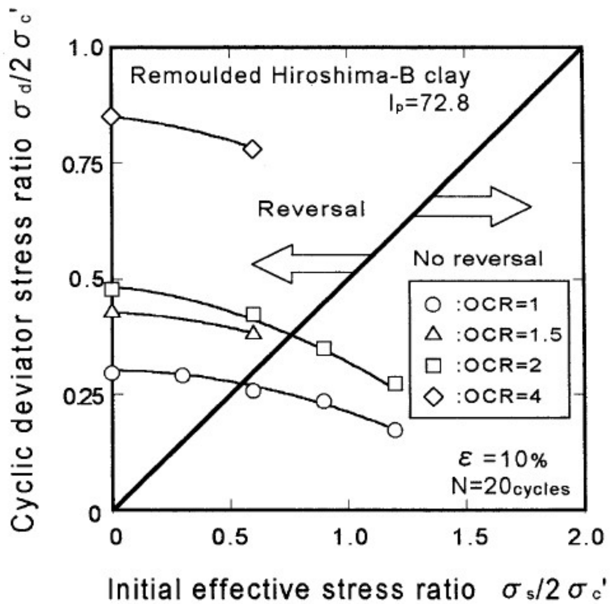


Figure 3.24: Variation with overconsolidation ratio of the cyclic strength with initial drained shear stress (after [24]).

Chapter 4

Post-Cyclic Response

As evident from the previous chapter, a rather substantial body of experimental work has been performed on cyclically loaded cohesive soils. Perhaps of equal importance, though not nearly as extensively studied, is the post-cyclic response of such soils. Post-cyclic tests generally consist of standard monotonic loading that is applied to samples after completion of the cyclic loading.

As noted in Section 3.5, normally consolidated samples subjected to undrained cyclic loading exhibit an apparent overconsolidation. The level of such overconsolidation increases with continued cyclic loading. Since this is accomplished by increases in the excess pore pressure, the effective stress state progressively reduces. It follows that if monotonic loading is applied following a cyclic loading event, the sample will respond similar to an overconsolidated one [57; 56; 1; 40; 18; 4; 39; 69; 64; 70]. Figure 4.1 illustrates the fact that the stress paths under monotonic loading applied after cyclic loading are typical of those associated with overconsolidated samples.

Yasuhara et al. [70] confirmed the observation that normally consolidated samples subjected to cyclic loading exhibit an apparent overconsolidation. The overconsolidation increased with the increase of excess pore water pressure generated by cyclic loading. From Figures 4.2 and 4.3 it is evident that the slope, and hence the stiffness, at the beginning of each curve decreases with increasing cyclic-induced excess pore pressures (i.e., increasing the number of cycles). The same trend likewise applies to the peak deviator stress. Cyclic degradation of stiffness is more marked than that of undrained strength, and this tendency increases with increasing OCR. Yasuhara et al. [70] found that in most of the cases (i.e., $OCR = 1$, 2, 4, 10), the post-cyclic undrained shear strength did not reduce by more than 20% of the value associated with monotonic loading. For this reason, it was concluded that the cyclic softening that sometimes triggers lateral deformation and instability in fine-grained plastic silt during earthquakes must be caused by a decrease in stiffness rather than undrained strength. Indeed, the effective shear strength parameters c' and ϕ' appear to be unaffected by cyclic loading [1; 30; 36].

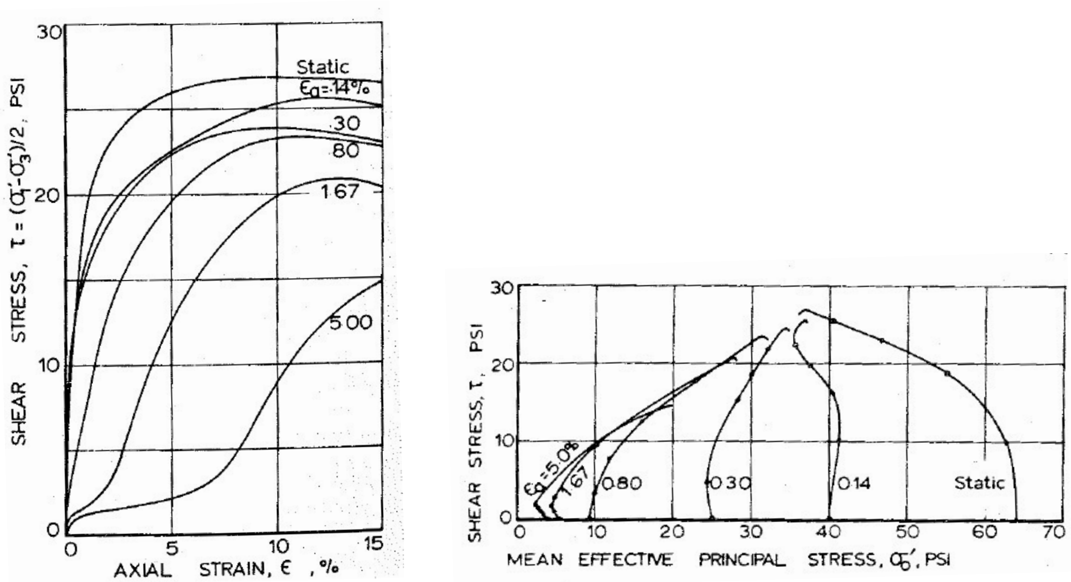


Figure 4.1: Monotonic triaxial tests after cyclic loading (after [56]).

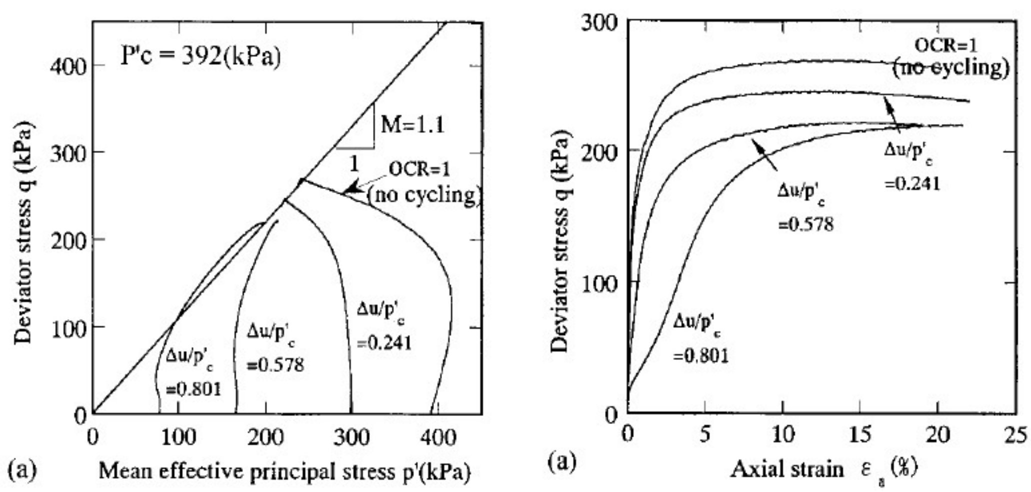


Figure 4.2: Post-cyclic stress paths for $OCR = 1$ and $OCR = 10$ (after [70]).

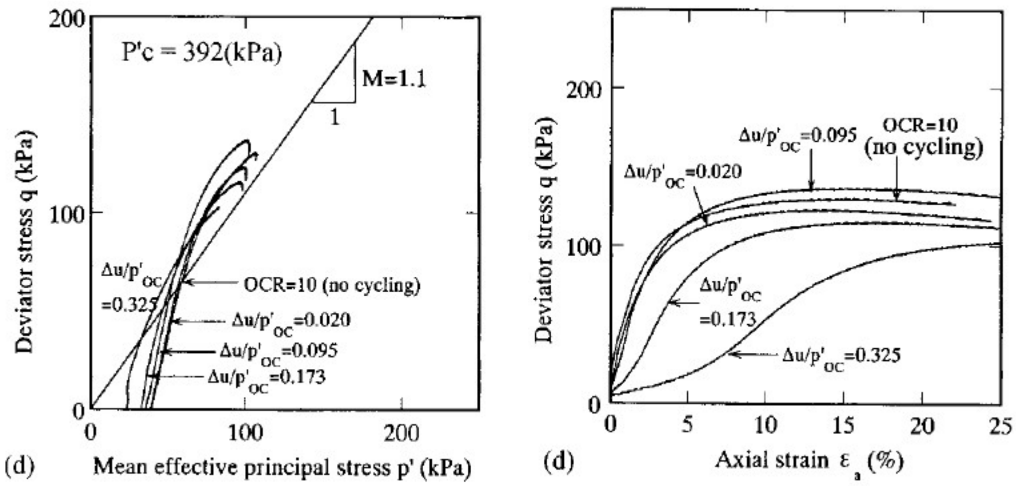


Figure 4.3: Post-cyclic stress-strain curves for $OCR = 1$ and $OCR = 10$ (after [70]).

Andersen et al. [1] compared the undrained shear strength before and after cyclic loading. They found that the latter decreased by approximately 25 %, provided the cyclic shear strain was less than $\pm 3\%$ after 1000 cycles. The aforementioned reduction in shear strength increased with both the level of the cyclic shear strain and with the number of loading cycles. Andersen et al. [1] also confirmed the observation that cyclic loading produces an apparent overconsolidation. Figure 4.4 illustrates how the post-cyclic stress paths tend towards the stress origin. In particular, following cyclic loading, the stress path of a normally consolidated sample reconsolidated to 400 kPa attains a mean normal effective stress of 95 kPa; this value is quite similar to a monotonically loaded overconsolidated sample with $OCR = 4$.

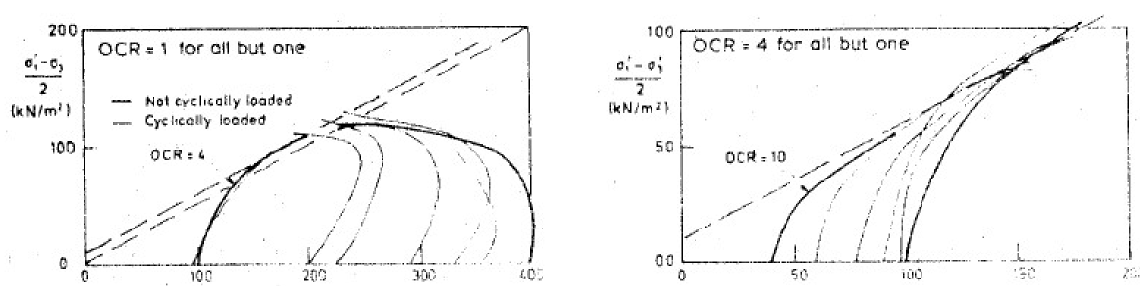


Figure 4.4: Effective stress paths for monotonic loading before and after cyclic loading (after [1]).

Hyde and Ward [18] found that the reduction in post-cyclic shear strength is greater for normally consolidated and lightly overconsolidated samples as compared to ones that were heavily overconsolidated. Heavily overconsolidated samples did not generate large excess pore pressures under cyclic loading; consequently, they did not, therefore, exhibit large changes in post-cyclic shear strength.

The undrained shear strength of cohesive soils tends to decrease for larger induced cyclic strain amplitudes. Since stiffness degradation is rather strongly dependent on the level of cyclic stress/strain applied to the sample (recall the discussion of Section 3.7), it follows that the more stiffness degradation that a sample exhibits, the greater will be the reduction in post-cyclic shear strength[1]. Finally, since cyclically loaded overconsolidated samples do not generate large positive excess pore pressures, it follows that the reduction in their shear strength will be smaller than for normally consolidated samples.

Meimon and Hicher [40] explained that the reduction of the post-cyclic undrained shear strength depends on the disturbance of the clay structure created by the cyclic loading. They related the decrease in the normalized post-cyclic undrained shear strength with the permanent axial strain (ε_{max}) at the end of the cyclic test (Figure 4.5). It is evident that, provided that ε_{max} does not exceed 5 %, the reduction in the undrained shear strength will be less than about 8 %. For higher values of ε_{max}, the aforementioned reduction can, however, reach 40%.

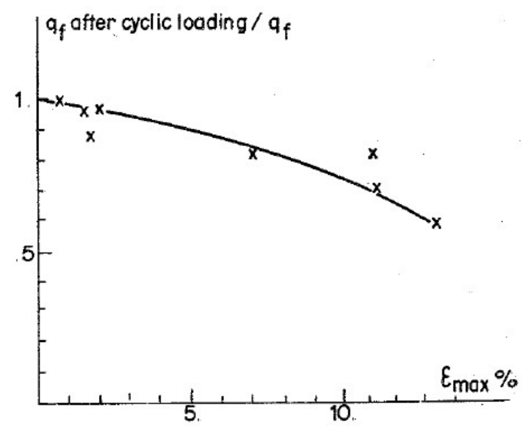


Figure 4.5: Reduction of undrained shear strength for $OCR = 1$ and 4 (after [40]).

Zergoun [73] and Zergoun and Vaid [74] indicated that a general degradation of both strength and stiffness is noted as a result of cyclic loading; this degradation seemed to increase with the magnitude of the maximum strain during cyclic loading. Figure 4.6 shows the relation between maximum strain due to two-way cyclic loading and post-cyclic undrained

[1]It is timely to note that both stiffness degradation during cyclic loading and the post-cyclic response are directly affected by the magnitude of the induced cyclic shear strain, rather than by the cyclic stress level. If the strain is below the threshold value, the reduction in post-cyclic shear strength will be small or altogether nil. If, on the other hand, the induced cyclic strain exceeds the threshold value, the reduction in post-cyclic shear strength will be quite significant.

strength for different test conditions; these include compression-extension and extension-compression cyclic loading, cyclic loading under constant stress ratio amplitude, and post-cyclic compression and extension tests. There is evident that this relation is applicable regardless of the manner in which the maximum strain was accumulated.

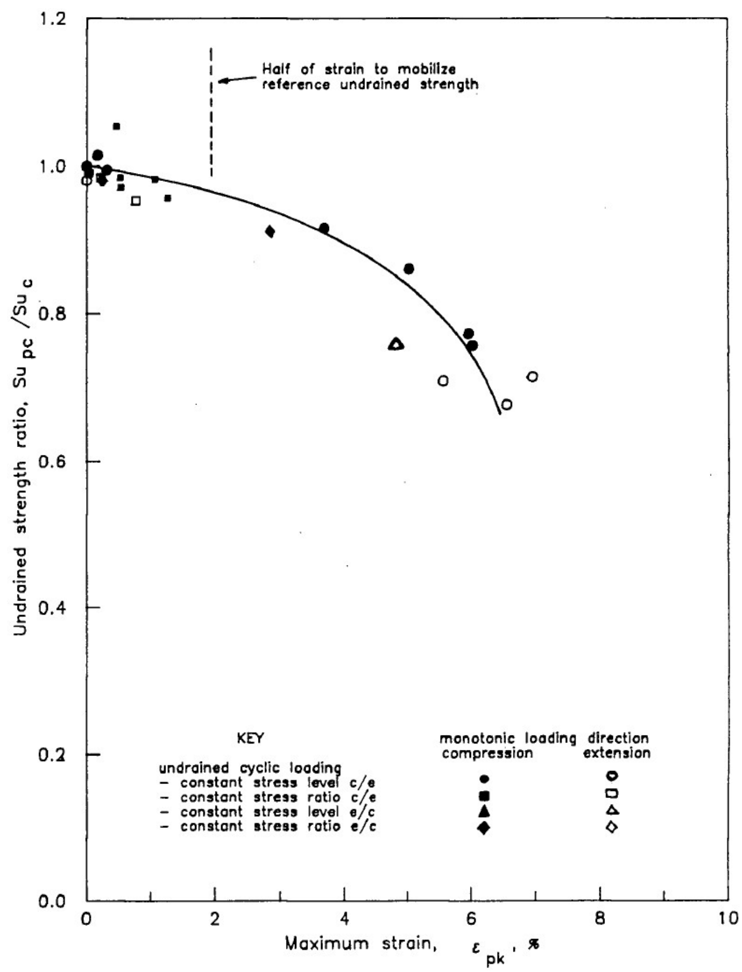


Figure 4.6: Relationship between maximum strain due to cyclic loading and post-cyclic undrained strength (after [73]).

Yasuhara [64] evaluated the effect of applying an initial deviator stress q_s (and thus creating an anisotropic initial stress state) on post-cyclic soil response. Cyclic deviator stress was applied until some specific value of excess pore pressure was reached. Then, monotonic undrained shearing of the sample took place. In Figures 4.7 and 4.8 are shown the results for the cases of $q_s/p'_c = 0$ (which corresponds to the isotropic initial stress state) and $q_s/p'_c = 0.30$ (which corresponds to an anisotropic initial stress state; i.e., initial stress $q_s = 58.8\,kPa$).

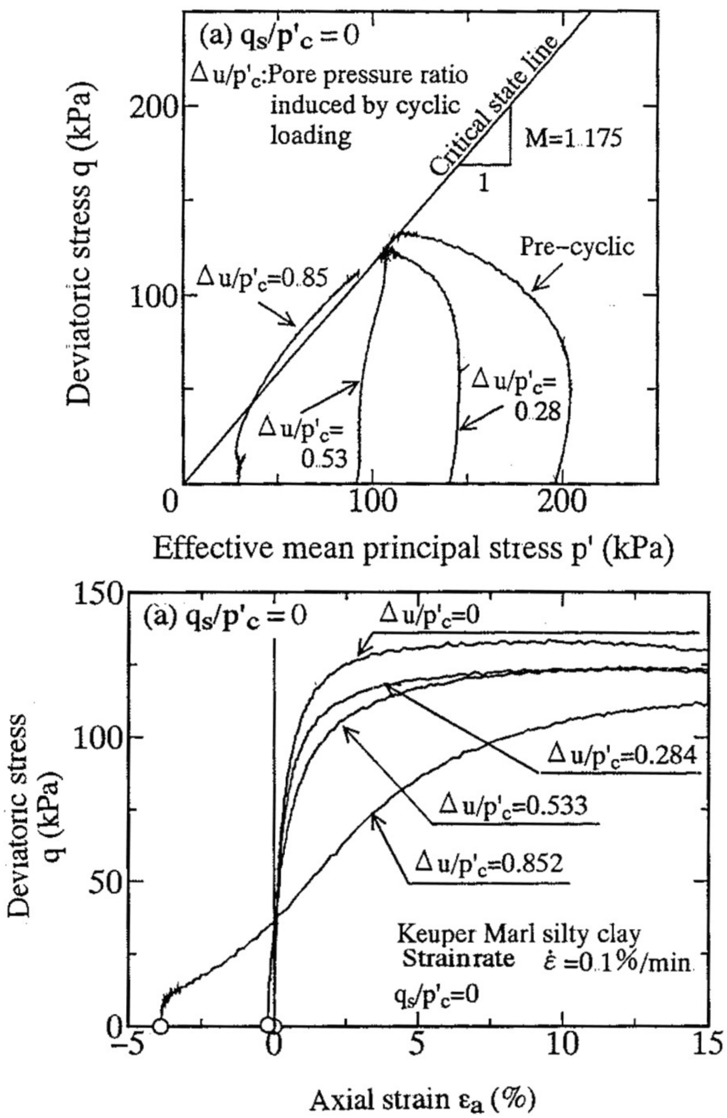


Figure 4.7: Pre-cyclic and post-cyclic stress paths for isotropic stress state ($p'_c = 198\,kPa$) (after [64]).

Figures 4.9 and 4.10 illustrate the effect that drainage prior to post-cyclic loading has on the undrained shear strength. Comparing first the post-cyclic response without drainage to the monotonic case, it is evident that the elastic modulus decreases significantly more that the shear strength. Many researchers [57; 56; 25; 29; 1; 26; 18; 9; 73; 39; 69; 74; 67; 3; 11; 61] have confirmed such behavior.

In the case of post-cyclic response with drainage allowed, from Figure 4.10 it is evident that the reduction in void ratio that accompanies the drainage manifests itself in volumetric hardening and a rather substantial increase in shear strength [19; 40; 69].

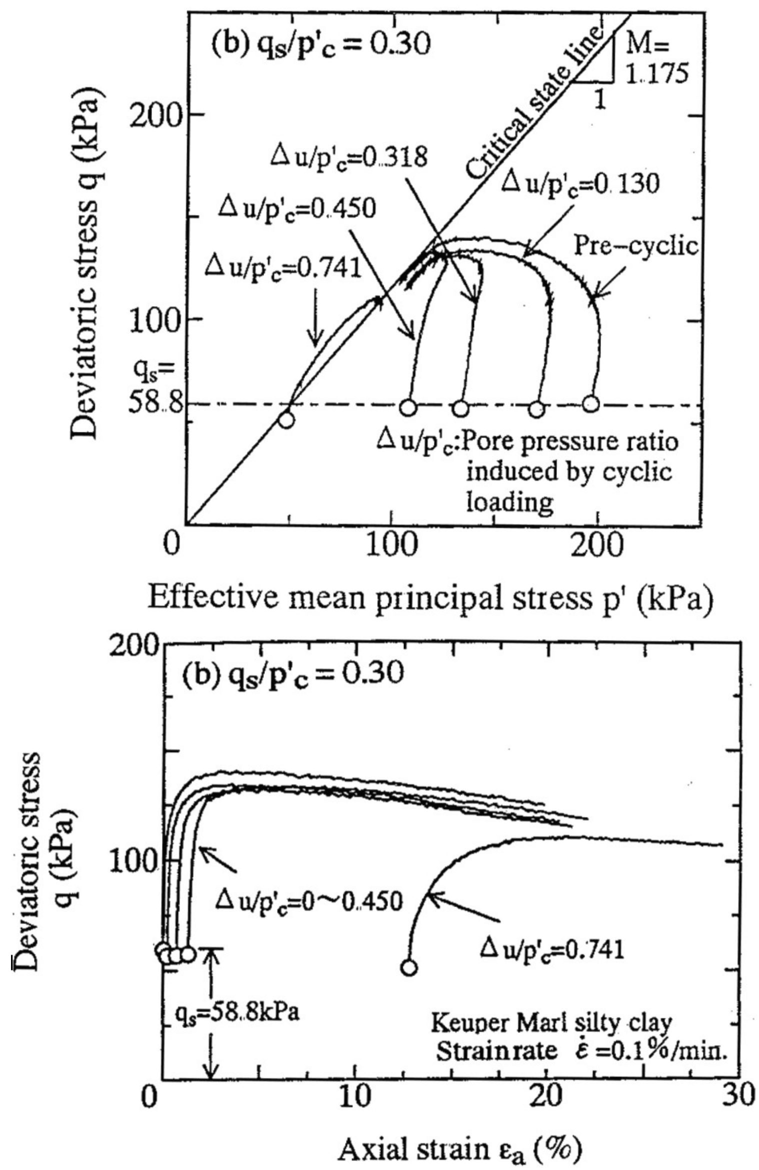


Figure 4.8: Pre-cyclic and post-cyclic stress-strain curves for anisotropic stress state ($p_c' = 198\,kPa$) (after [64]).

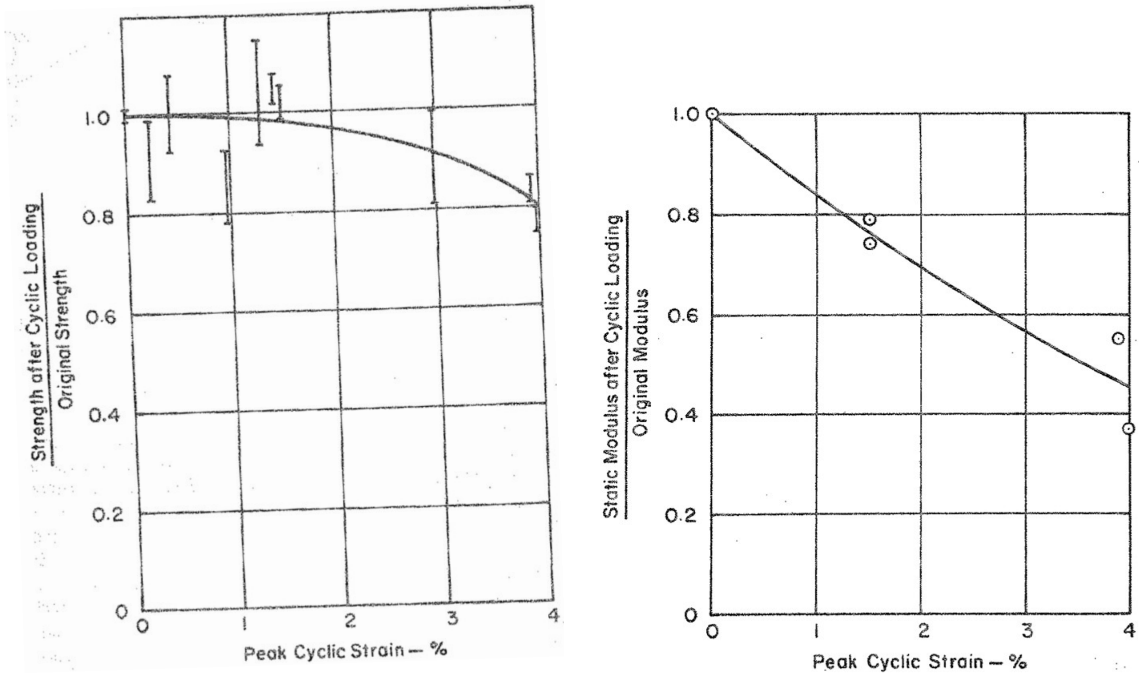


Figure 4.9: Effect of cyclic loading on undrained monotonic shear strength and shear modulus (after [57]).

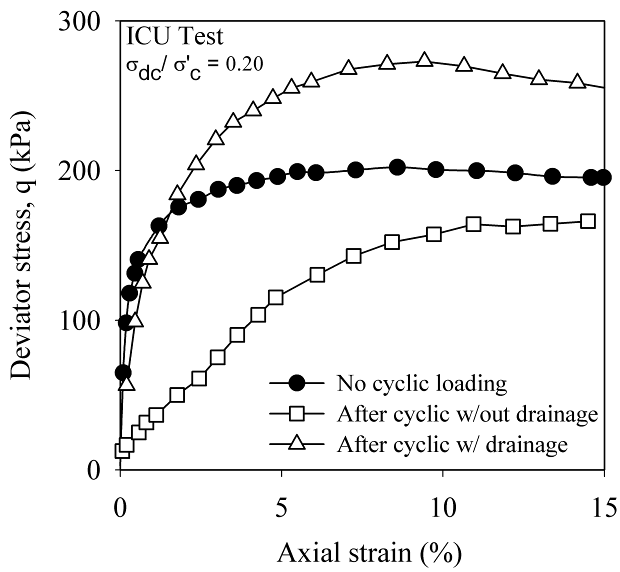


Figure 4.10: Postcyclic response of cohesive soils and secant Young modulus [ref??].

Chapter 5

Concluding Remarks

The main features of the response of saturated cohesive soils subjected to cyclic loading are summarized below.

Concerning Response during Cyclic Loading

- There is a critical cyclic stress or strain that is sometimes referred to as the "cyclic threshold". This threshold clearly delineates two district types of response. For cyclic stresses/strains amplitudes below the threshold, a sample reaches an equilibrium state. If, on the other hand, the cyclic stress/strain amplitude exceeds the threshold, a sample will accumulate inelastic strains, will generate excess pore pressure and may fail.
- The cyclic stress threshold is about 60% to two-thirds of the undrained shear strength as measured in a single cycle or standard monotonic triaxial test. This condition is for tests where the sample is not subjected to an initial sustained load.
- When a sample reaches the equilibrium condition, the evolution of plastic strain and excess pore pressure ceases. Only elastic strains are generated in loading-unloading-reloading cycles. The elastic strains are non-linear but are associated with essentially closed hysteresis loops.
- An equilibrium state is generally reached for a *lesser* number of cycles for normally consolidated samples as compared to overconsolidated ones.
- Cyclic loading causes an apparent overconsolidation of normally consolidated samples due to the reduction in the effective stress during cyclic loading. As a result, the effective stress path in post-cyclic monotonic loading for an initially normally consolidated sample will be similar to an effective stress path associated with an overconsolidated sample.
- For cohesive soils that are initially at the same state of effective stress but at different overconsolidation ratios, the threshold stress increases with overconsolidation ratio. In addition, the cyclic strength will increase with overconsolidation ratio.

- Clays with high overconsolidation ratios have more resistance to subsequent positive excess pore pressure generation with increasing number of cycles.
- Undrained cyclic loading causes a build-up of positive excess pore pressures in normally consolidated samples. By contrast, in overconsolidated samples a negative pore pressure may develop in the early cycles. With continued cyclic shearing, the generation of negative excess pore pressure slowly decreases; the excess pore pressure may thus start to increase and may eventually become positive. The higher the overconsolidation ratio, the more resistance there is against subsequent positive excess pore pressure generation with cyclic loading.
- The peak shear strength for each cycle decreases according to the cyclic strain amplitude. The higher the cyclic strain amplitude, the higher is the rate of strength decrease.
- Since cohesive forces increase in higher plasticity clays, such soils develop lower amounts of cyclic shear strain as compared to lower plasticity silts and clays. Cyclic strength thus increases with plasticity index.
- Soft clays are weaker under cyclic loading than stiff clays.
- In anisotropically normally consolidated samples, excess pore pressures increase with cyclic loading. The effective stress state thus moves toward a failure state. The higher the cyclic deviator stress, the fewer the number of cycles required to reach failure.
- Cyclic strength decreases at higher levels of initial drained shear stress.

Concerning Stiffness Degradation

- Soil degradation is strongly dependent on the cyclic strain level applied to a sample.
- Low frequencies lead to the lowest cyclic strength and more degradation effects than higher frequencies.
- Cyclic stress reversal or two-way cyclic loading has a more damaging effect on clay behavior than non-reversal (one-way) cyclic loading.
- The stiffness of normally consolidated samples degrades faster than that of overconsolidated samples.
- The magnitude of soil stiffness degradation (i.e., lower degradation index) is larger for normally consolidated samples than for overconsolidated ones.
- The rate of soil stiffness degradation during cyclic loading decreases with increasing overconsolidation ratio.

- The rate at which the effective stress path migrates towards the origin of the stress space, and hence the rate at which stiffness reduces and hysteresis increases, has been found to depend on cyclic stress ratio, consolidation history, and frequency of loading. Increasing the cyclic stress ratio thus increases the rate of migration of the effective stress cycles. Normally consolidated samples migrate faster than overconsolidated ones. Samples loaded at low frequencies generate higher excess pore pressures and so migrate more rapidly than samples loaded at higher frequencies.

Concerning Post-Cyclic Response

- When the induced cyclic shear strain amplitude is less than the threshold cyclic strain, no significant stiffness degradation will occur; the reduction in post-cyclic strength will therefore be negligible.
- Cyclic loading leads to softening of a sample. The decrease of the undrained modulus (E_u) in monotonic loading after cyclic loading is thus larger than the decrease of strength.
- In general, post-cyclic undrained shear strength is not less than 25% of the initial or pre-cyclic undrained shear strength. The strength decrease becomes larger with increasing cyclic shear strain level (ε_{max}) and with an increase in the number of cycles. For example, if $\varepsilon_{max} < 5\%$, this reduction can be as low as 8%. For higher values of ε_{max} this reduction can be on the order of 40%.
- Even though some authors report that post-cyclic undrained shear strength does not depend on the overconsolidation ratio, others have found that the reduction in strength after cyclic loading is greater for normally consolidated and lightly overconsolidated samples than for heavily overconsolidated ones.

Concerning Periods of Drainage Prior to Post-Cyclic Shearing

- The inclusion of drainage periods between or after cyclic events produces a strengthening effect on normally consolidated samples.
- In overconsolidated samples, the inclusion of drainage periods between or after cyclic events produces a deteriorating effect that leads to lower resistance to subsequent undrained cyclic loading.

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