

**Strength Degradation of Holocene Bay Mud
from Marin County, California,
Under Seismic Loading**

By

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Abstract

Cyclic simple shear tests performed on Holocene bay mud (known locally as San Francisco Bay Mud) at the University of California at Berkeley following the 1989 Loma Prieta earthquake, suggested that the response of silty clay to cyclic loading might be more severe than earlier research had indicated. Due to the abundance of facilities constructed on Holocene bay mud, the risk to the San Francisco bay area during a moderate to large earthquake is significant, if such strength degradation leads to large deformations or failures of embankments or deep foundations on this deposit. A program of laboratory testing was therefore carried out to investigate the potential strength degradation of Holocene bay mud subjected to a range of conditions representative of those expected in a major earthquake.

The laboratory study was designed as a comprehensive dynamic testing program on Holocene bay mud from Hamilton Air Force Base located in Marin County, California. All of the tests were performed using the UC Berkeley Simple Shear Device in strain-controlled mode. Specimens were tested under monotonic, cyclic, and post-cyclic loading, and the effects of laboratory consolidation history were considered through a program designed to determine normalization parameters. Factors that were varied in the cyclic tests include strain amplitude, number of cycles, and strain rate.

Dynamic response was interpreted through determination of threshold strain parameters and changes in shear modulus, damping, and shear strength mobilization under strain-

controlled cyclic loading. The results show that the volumetric cyclic threshold shear strain is between 0.02% and 0.04%. The degradation of shear modulus with strain amplitude is consistent with data presented in an earlier study on Holocene bay mud from the same location, and it is more severe at high strain ranges than suggested by empirical correlation with plasticity index. The damping ratios for the first cycle of loading are consistent with the limits suggested for cohesive soils from earlier studies. The normalized shear stress in the first cycle of loading exhibits approximately a 13-16% increase per order of magnitude increase in strain rate, which is consistent with studies on other cohesive soils. The mobilized strength then degrades to approximately the monotonic level by the fifteenth cycle, with the greatest reduction in stress occurring between the first and fifth cycle.

The post-cyclic monotonic strengths are within $\pm 10\%$ of the monotonic strengths of specimens that had not undergone cyclic loading. There were no clear effects of varying the strain amplitude, frequency of loading, or strain rate, but dissipation of pore pressures between the end of cyclic loading and the beginning of monotonic shear increases the strength by an average of 8%.

In general, the cyclic response of Holocene bay mud in the test program was similar to that indicated by previous studies on bay mud and other similar soils. However, even though the normalized shear stresses during the first cycles of loading were high due to strain rate effects, at higher cyclic strains the strength rapidly degraded due to cyclic reversals to approximately the monotonic strength. Most of this degradation occurred

within the first five cycles of loading. One-dimensional site response analysis was performed to estimate the amplitude and number of cycles of shear strain in moderate to large earthquake, and it showed that up to five cycles or more of shear strain amplitudes exceeding 1% could be expected. Therefore, the strength degradation that was observed in the cyclic testing is within the range of interest for geotechnical earthquake engineering.

A comparison between the cyclic response of the specimens from Hamilton Air Force Base and specimens tested in the earlier study from the Marina District in San Francisco proved of particular interest. The Marina District specimens exhibited a significantly more severe degradation both during and after cyclic loading, especially for cyclic strains exceeding 1%. It appears that the higher silt content and lower plasticity index ($PI = 25$ vs. $PI = 45$ for Hamilton Air Force Base) is responsible for the difference in degradation, since the dynamic response of clays across all strain ranges has been shown by numerous studies to be a strong function of plasticity index.

Finally, available geotechnical literature and data were reviewed to provide an understanding of the variation of physical properties of Holocene bay mud across the bay area, and how results of the study may be extrapolated to shed light on the response of other, similar, clay deposits. The variation of monotonic strength of Holocene bay mud as influenced by plasticity index, anisotropic stress conditions, strain rate, and test type, falls within the expected range for clays. However, the engineering characteristics of Holocene bay mud vary considerably across the region; the plasticity index reported for bay mud ranges from 14 to 64 (the average plasticity index of specimens tested in this study was

approximately 45). Due to the strong link between cyclic resistance and plasticity index, an important conclusion of this study is that the cyclic response of Holocene bay mud should be considered in the context of the engineering index characteristics at a particular location and depth.

TABLE OF CONTENTS

Abstract.....	i
Table of Contents.....	v
List of Figures.....	viii
List of Tables.....	xi
List of Symbols.....	xii
Acknowledgments.....	xv
CHAPTER 1 INTRODUCTION.....	1
CHAPTER 2 REVIEW OF PREVIOUS STUDIES.....	7
2.1 Case Histories.....	7
2.2 Response of Clay Under Monotonic, Cyclic, and Post-Cyclic Loading.....	8
2.2.1 Monotonic Response and Normalization Parameters.....	8
2.2.2 Response Under Cyclic Loading.....	21
2.2.2.1 Threshold Levels.....	21
2.2.2.2 Shear Modulus.....	25
2.2.2.3 Damping.....	34
2.2.2.4 Cyclic Strength.....	38
2.2.3 Post-Cyclic Response.....	46
2.2.3.1 Apparent Overconsolidation Hypothesis.....	49
CHAPTER 3 NUMERICAL STUDIES OF SHEAR STRAIN TIME HISTORIES.....	54
3.1 Analysis of Strain Time Histories.....	55

CHAPTER 4 DEPOSITION HISTORY AND CHARACTERIZATION OF HOLOCENE BAY MUD FROM SAN FRANCISCO BAY.....	61
4.1 Depositional History and Distribution.....	61
4.2 Distribution and Variability of Engineering Characteristics.....	65
4.3 Holocene Bay Mud at Hamilton Air Force Base.....	71
CHAPTER 5 SAMPLING, SPECIMEN PREPARATION, AND TEST PROCEDURES.....	73
5.1 Sampling Procedures and Specimen Preparation.....	73
5.2 Consolidation Tests.....	73
5.3 Monotonic and Cyclic Simple Shear Tests.....	75
5.3.1 UC Berkeley Simple Shear Device.....	75
5.3.2 Stress States in Simple Shear Tests.....	77
5.3.3 Test Procedures.....	78
CHAPTER 6 TEST RESULTS.....	83
6.1 Consolidation Tests.....	83
6.2 Monotonic Simple Shear Tests.....	84
6.2.1 Normalization Parameters – Normally Consolidated Specimens..	86
6.2.2 Normalization Parameters – Overconsolidated Specimens.....	90
6.3 Cyclic Simple Shear Tests.....	95
6.3.1 Threshold Levels.....	95
6.3.2 Modulus Degradation.....	100
6.3.3 Damping Ratio.....	105
6.3.4 Effect of Strain Rate on Mobilized Shear Stress.....	105
6.3.5 Effect of Cycle Number on Mobilized Shear Stress.....	105

6.4 Post-Cyclic Simple Shear Testing.....	112
6.4.1 Comparison of Pre- and Post-Cyclic Monotonic Tests.....	112
6.4.2 Effect of Pore Pressures Post-Cyclic Strength.....	120
 CHAPTER 7 SUMMARY AND CONCLUSIONS.....	 127
7.1 Scope of Research.....	127
7.2 Research Findings.....	129
7.3 Recommendations for Future Research.....	133
REFERENCES.....	135
APPENDIX A: Soil Profiles Used in Numerical Response Studies.....	A-1
APPENDIX B: Individual Undrained Monotonic Simple Shear Test Results.....	B-1
APPENDIX C: Individual Undrained Cyclic Simple Shear Test Results.....	C-1

LIST OF FIGURES

<u>FIGURE</u>	<u>PAGE</u>
2.1 Vane shear strength vs. plasticity index.	10
2.2 Triaxial test results for bay mud, Hamilton Air Force Base.....	15
2.3 Influence of anisotropy on undrained triaxial strength of clays.....	17
2.4 Influence of strain rate on undrained strength of clay.....	19
2.5 Influence of test type on undrained strength ratio of clays.....	20
2.6 Influence of plasticity index on threshold shear strain levels.....	23
2.7 Shear modulus relationship in a cyclic test.....	27
2.8 Influence of plasticity index on modulus reduction	29
2.9 Modulus reduction relationships for bay mud.....	31
2.10 Calculation of damping ratio.....	36
2.11 Influence of plasticity index on damping curves for clay.....	37
2.12 Definition of strains in a cyclic test.....	40
2.13 Cyclic strength curves.....	41
2.14 (a) Cyclic strength curves assuming failure at unlimited strain..... development; (b) Cyclic strength curves for different strain levels.	45
2.15 Post-cyclic normalized strength of three soils.....	48
2.16 The Apparent Overconsolidation Hypothesis.....	51
3.1 Response spectra for bedrock motions used as input for response analyses.....	58
4.1 Structure contours on base of bay mud formation.....	63

4.2	Thickness contours of bay mud.....	64
4.3	Field vane and UU-triaxial shear strength profiles at MUNI Metro..... Turnaround Facility, San Francisco.	66
4.4	Plasticity chart for bay mud, various sites and depths.....	70
4.5	Geotechnical test site at Hamilton Air Force Base.....	72
5.1	UC Berkeley simple shear device.....	76
6.1	Preconsolidation stress profile at Hamilton Air Force Base.....	85
6.2	Stress-strain curves for monotonic simple shear tests MONO-1 through..... MONO-4	88
6.3	Stress-strain curves for monotonic simple shear tests MONO-5 through..... MONO-7	93
6.4	Development of pore pressures as a function of cycle number during..... cyclic loading	98
6.5	Development of pore pressures as a function of strain during cyclic..... loading	99
6.6	Modulus reduction curve for specimens from Hamilton Air Force Base.....	101
6.7	Modulus reduction curve for specimens from Hamilton Air Force Base,..... first cycle of loading	102
6.8	Modulus reduction curve for specimens from Hamilton Air Force Base,..... effect of cycle number	104
6.9	Damping ratio for specimens from Hamilton Air Force Base, first cycle..... of loading	106
6.10	Cyclic stress-strain (backbone) curves for Holocene bay mud from..... Hamilton Air Force Base	107
6.11	Cyclic stress-strain (backbone) curves for Holocene bay mud from..... the Marina District, normally consolidated specimens	109
6.12	Cyclic stress-strain (backbone) curves for Holocene bay mud from..... the Marina District, overconsolidated specimens	111

6.13	Typical test results of strain-controlled cyclic and post-cyclic simple shear.....	114
	testing of Holocene bay mud	
6.14	Normalized stress-strain curves for post-cyclic tests on Holocene bay mud.....	115
	from Hamilton Air Force Base	
6.15	Cyclic shear strain effects on post-cyclic strength.....	117
6.16	Cyclic frequency effects on post-cyclic strength.....	119
6.17	Pore pressure development in post-cyclic monotonic tests.....	121
6.18	Pore pressures generated at the end of cyclic loading.....	123
6.19	Comparison of post-cyclic strength data with apparent overconsolidation.....	125
	model.	

LIST OF TABLES

<u>TABLE</u>		<u>PAGE</u>
2.1	Undrained shear strength of bay mud, San Francisco Bay Area, California (normally consolidated specimens)	12
3.1	Rock motions used in SHAKE analyses	57
3.2	Summary of shear strain time history analyses	60
4.1	Index properties of bay mud, San Francisco Bay Area, California	68
6.1	Consolidation test results	84
6.2	Test parameters for monotonic tests	86
6.3	Tabulated results for monotonic tests	87
6.4	Test parameters for cyclic tests	96
6.5	Test results for post-cyclic tests	113

NOMENCLATURE

A_c	area of stress-strain cycle
A_T	area of triangle
AOCR	apparent overconsolidation ratio
C_c	compression index
CLRL	critical level of repeated loading
CSR	cyclic stress ratio
e_0	initial void ratio
E	energy
G_{max}	small strain shear modulus
G_s	secant shear modulus
ICU	isotropically consolidated, undrained
K_c	Lateral stress ratio after consolidation ($K_c = \sigma_{hc}'/\sigma_{ve}'$)
K_0	Lateral stress ratio for the at-rest condition
m	exponent
n	exponent
N	cycle number
NC	normally consolidated
OC	overconsolidated
OCR	overconsolidation ratio
PI	plasticity index

S_u	Undrained strength
t	degradation parameter
t_c	triaxial compression
u_{max}	excess pore pressure at failure (monotonic tests)
	average pore pressure in last cyclic of loading (cyclic tests)
UU	undrained, unconsolidated
V_s	shear wave velocity
VST	vane shear test
w_c	Water content after consolidation
w_i	Initial water content
z	depth
δ	degradation index
ϵ_f	axial strain at failure
ϕ	angle of internal friction
ϕ'	effective stress angle of internal friction
γ	shear strain
γ_{cyc}	single amplitude cyclic shear strain
γ_{p-p}	peak strain amplitude
γ_{tl}	linear cyclic threshold shear strain
γ_{tv}	volumetric cyclic threshold shear strain
λ	damping ratio
ρ	density

σ_{hc}'	horizontal effective consolidation stress
σ_p'	maximum past vertical effective consolidation stress
σ_{vc}'	vertical effective consolidation stress
σ_{vo}'	in situ vertical effective consolidation stress
τ	shear stress
τ_f	shear stress on horizontal plane at failure
τ_s	static shear stress after consolidation

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CHAPTER 1 INTRODUCTION

The 1964 Niigata and Great Alaska earthquakes served as an impetus for much of the modern research into the seismic response of soils. While the phenomenon of liquefaction often receives most of the recognition, the behavior of clays and clay slopes has also received notable attention (Kovacs et al., 1971; Arango and Seed, 1974; Idriss, 1985; de Alba and Pyke, 1987). Two decades after the Niigata and Great Alaska earthquakes, the 1985 Mexico City Earthquake vividly demonstrated that amplification of ground motions could be significant in soft clay deposits, due to period matching of these deposits to the predominant periods of an earthquake (Borja-Navarrete et al., 1986; Seed et al., 1987). The phenomenon was also very much evident in the 1989 Loma Prieta Earthquake, in which ground motion amplification occurred in areas underlain by Holocene bay mud along the margins of San Francisco Bay. Records from a number of instrumented sites provided reliable quantitative evidence that amplification had occurred at sites as far as 100 kilometers from the rupture zone (Hough et al., 1989; Rogers, 1991; Seed et al., 1990).

While amplification of earthquake motions can lead to stronger shaking and therefore damage to structures, there remains the question of the extent to which clays retain their initial fabric and resistance to shaking, or whether they exhibit a process similar to liquefaction in sands, losing strength as pore pressures rise. Strength degradation leading to large deformations or failures of embankments founded on soft clay could contribute significantly to the overall damage pattern following a high magnitude

earthquake. Examples of sites underlain by soft clay abound in the San Francisco Bay Area of California, where most of the downtown buildings are built on deposits that include a thick layer of Holocene bay mud, known locally as San Francisco Bay Mud (Atwater et al., 1977). Many other developments along the shores of the San Francisco Bay are built on top of this clay deposit, including solid waste fills. A major earthquake with ground motions much larger than the 1989 Loma Prieta earthquake is predicted to hit the Bay area in the next 30 years and, consequently, there is a need to develop reasonably accurate methods to predict the potential deformations in these clays in response to a high magnitude earthquake (Working Group on California Earthquake Probabilities, 1990). This information would be useful to the worldwide geotechnical community, as many parts of the world, such as major ports, are underlain by thick deposits of soft clay.

Early research in this area, starting in the 1960's, concluded that while there are some changes in pore pressures and in stress-strain properties when clays are cyclically loaded, the plasticity characteristics of clays gives them an overall inherent resistance to cyclic disturbance. This conclusion may have biased the interpretation of case histories of seismic failures, unintentionally downplaying the role of clay strength degradation. For instance, a reexamination of slope failures in the Anchorage, Alaska area during the 1964 Great Alaska earthquake showed that strength degradation in clay layers was probably a greater contributor to the failure than was originally thought (Idriss, 1985). Previous model studies using artificial clay embankments have demonstrated that failures during large magnitude earthquakes are indeed a possibility

(Kovacs et al., 1971; Arango and Seed, 1974). Furthermore, evidence suggests that deformation of the underlying soft clays at the fill margins of San Francisco Bay might have contributed to some of the observed deformations, especially along interstate 880 in Emeryville and at Treasure Island. As analysis procedures for seismic response and deformation become increasingly sophisticated, there is a clear need not only to verify the potential for strength degradation to occur, but also to quantify the magnitude of degradation and develop guidelines for laboratory testing and determination of the appropriate parameters to use in such analyses.

The purpose of this study was to quantify the strength degradation that could be expected in Holocene bay mud during a typical large magnitude earthquake expected in the San Francisco Bay Area, and to identify how degradation is affected by factors that are believed to most influence cyclic degradation. Particular focus is on those aspects of clay behavior that are pertinent to the analysis of slope stability and deformations under seismic loading.

A review of the research literature in this area, presented in Chapter Two, shows that strength degradation in clays has been demonstrated time and again, but that most research has focused on a narrow aspect of response, such as small strain properties, the influence of initial shear stresses, or post-cyclic behavior. The variations in choice of test material and in laboratory test types and procedures make it extremely difficult to piece together a comprehensive understanding of how a particular soil will respond to cyclic loading over the entire range of initial conditions and load characteristics.

This study is an attempt to provide such a comprehensive understanding of one soil, Holocene bay mud from the San Francisco Bay. Some discussion is also made of the variation of physical properties of Holocene bay mud across the bay area, and how results of the study may be extrapolated to shed light on the response of other, similar, clay deposits. Thus, a laboratory testing program was carried out to provide an understanding of the clay properties, slope geometry, stress conditions, and load characteristics that could lead to large deformations or failure in clay slopes under seismic loading.

The logical choice, from a testing perspective, to use uniform amplitude cycles in the laboratory creates an issue regarding the use of the results to model the response of clays to real, irregularly patterned earthquake loads. It is necessary to develop a rational approach for comparing earthquake time histories with equivalent uniform laboratory cycles. Seed and Idriss (1982) presented a numerical study that compared earthquake magnitude with equivalent uniform average shear stress cycles. The present study focuses on strain-controlled testing, because recent research has elucidated the strain-dependence of cyclic resistance and degradation. Chapter Three considers the amplitude and number of cycles of shear strain in earthquake time histories, based on one-dimensional site response analysis.

Holocene bay mud is the youngest deposit in a sequence of sediments infilling a structural depression occupied by the San Francisco Bay. The dynamic depositional environment over time within the bay has contributed to a spatial variability of soil

characteristics, described in Chapter Four. The study was limited to specimens from one location, Hamilton Air Force Base in Novato, California. The benefit of using specimens from one location is that it promotes the consistency and therefore the quality of the data, and it is believed that testing specimens from other locations would only be useful once there was a solid database of test results from the original location. Comparisons are made with results from other studies, for purposes of both validation and for extrapolation of the conclusions.

The test program is outlined in Chapter Five. All of the tests were performed using the UC Berkeley Simple Shear Device in strain-controlled mode. The simple shear testing apparatus provides a better approximation of an earthquake's horizontal shear stresses than does the pervasive triaxial device. Specimens were tested in monotonic, cyclic, and post-cyclic modes, and the effects of laboratory consolidation history were explored through a program modeled after the SHANSEP procedure (Ladd and Foott, 1974). The influencing factors that were varied are strain amplitude, number of cycles, and strain rate.

The test results are presented in Chapter Six. The monotonic test program is used to determine the undisturbed monotonic shear strength of Holocene bay mud with which to normalize all subsequent cyclic and post-cyclic results. Dynamic strength characteristics are presented in terms of shear modulus degradation and change in damping ratio, and the mobilization of shear stress at large strains. Changes in monotonic strength after cyclic loading, as they are influenced by the varied parameters

and by allowing pore pressures to dissipate, are submitted. Finally, the development of pore pressures in monotonic, cyclic, and post-cyclic tests is presented.

Chapter Seven presents a summary of the overall results and conclusions of the present study and a discussion of their implications for analyses of slope deformations in seismic loading, along with recommendations for further studies.

CHAPTER 2 REVIEW OF PREVIOUS STUDIES

2.1 Case Histories

Although much of the focus concerning damage from earthquakes relates to liquefaction of sandy soils, many observed failures have involved cohesive soils as well. Saturated cohesive soils were the cause of failure in the banks of the All America Canal during the 1940 El Centro Earthquake (Seed, 1967). Weak cohesive layers supporting dikes along the Agano River in Niigata City, and along the Mogami, Ada, Oyama, and Kyoden rivers near Sakata City were responsible for cracks, subsidence, and embankment slips in the 1964 Niigata Earthquake in Japan (Yokomura, 1966; Kawakami and Asada, 1966). Failed fill slopes with clay foundations were also observed in the 1978 Miyagiken-oki earthquake (Hyodo, 1993). More recently, deformations in underlying soft clays probably contributed to some of the observed failures associated with the 1989 Loma Prieta earthquake along Interstate 880 and at Treasure Island in the San Francisco bay area.

Landslides in cohesive soils have also been triggered by earthquakes; examples include submarine slumps triggered by seismic activity on the Israeli continental slope (Almagor and Wiseman, 1977). In the Niigata Earthquake of 1964, a cut slope at the entrance to a railway tunnel on the Uetsu-Line failed in deeply weathered mudstone and soft clayey soil after a series of aftershocks and rainfall (Yamada, 1966). Many landslides have been in slopes of clays alternating with silt and sand seams, and the cause of the failures has often been attributed to liquefaction of the silt and sand layers.

A large landslide on the San Pedro River near Lake Rinehue slid along a surface in a clay deposit with numerous silt and sand seams in the 1960 Chilean Earthquake (Seed, 1967). In the Great Alaskan Earthquake of 1964, a number of slides occurred in cohesive soils along the Anchorage bluffs. In addition, major slides in the Anchorage area - the L-Street Slide, the 4th St. Slide, and the Government Hill Slide - as well as the Turnagain Heights Slide, are thought to have been caused by liquefaction of sand layers (Seed, 1967). Reevaluation of two of these slides, the L-Street Slide and the 4th St. Slide, has shown that strength degradation in cohesive layers was probably at least partially responsible for triggering the failures (Idriss, 1985; Moriwaki and Vicente, 1986).

2.2 Response of Clay Under Monotonic, Cyclic, and Post-Cyclic Loading

2.2.1 Monotonic Response and Normalization Parameters

Determining the change in strength due to cyclic loading requires that the pre-shear, “static” strength of the clay can be reasonably estimated. This is done by testing the clay in a slow monotonic test under conditions that are as close as possible to the conditions in the cyclic test. It has been observed that for many normally consolidated clays, there is a direct relationship between consolidation stress and undrained strength that takes the form of Equation 2.1, where the constant is determined experimentally. The ratio S_u/σ_{vc}' is called the *undrained strength ratio*. For most clays, the undrained strength of a normally

consolidated laboratory specimen from any depth and at any consolidation stress can be determined from Equation 2.1.

$$S_u/\sigma_{vc}' = \text{constant} \quad 2.1$$

A general relationship between undrained strength ratio and plasticity was recognized by Skempton (1957) with the following empirical relationship describing the correlation between vane shear strength and plasticity:

$$S_u(\text{VST})/\sigma_{vc}' = 0.11 + 0.0037\text{PI} \quad 2.2$$

Figure 2.1 is a plot of Skempton's original data together with data from three Holocene bay mud sites from the San Francisco bay area. A similar relationship should hold for other types of tests.

The effects of sampling disturbance on natural clays are such that the strength derived from laboratory testing at in situ stresses may be either lower or higher than the in situ value (Ladd and Foott, 1974; La Rochelle et al., 1981; Baligh et al., 1987; Gens, 1994). Sources of disturbance include drilling operations, pushing of a piston sampler, mechanical distortion during penetration of the sampling tube, mechanical distortion and suction during retrieval, relief of total stresses, and disturbances during transportation and trimming. Even without mechanical disturbance, a soil sample will experience stress changes that affect its strength and modulus.

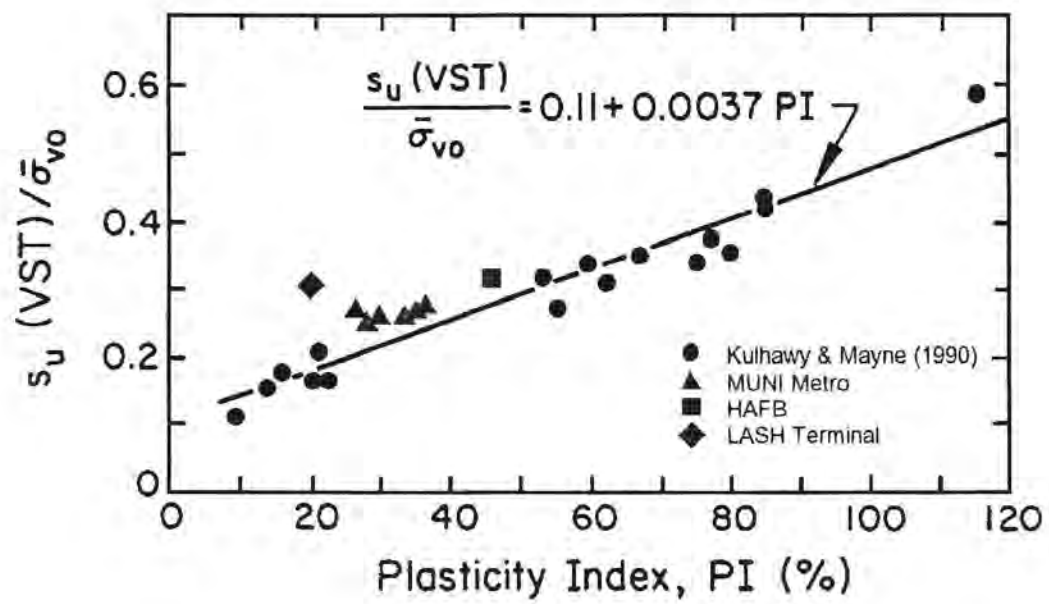


Figure 2.1 Vane shear strength vs. plasticity index. Adapted from Kulhawy and Mayne (1990).

Analysis of sampling disturbance within the conceptual framework of a yield surface for clays (La Rochelle et al., 1981; Jamiolkowski, 1985; Gens, 1994) has provided a basis for evaluating and quantifying sampling disturbance under different stress paths, and for various overconsolidation ratios. From these experimental studies it is clear that disturbance disrupts the clay particle assemblages or structure, and that the large-strain behavior of a clay represents a fully destructured, or intrinsic behavior of the particular clay. Processes that lead to the development of structure of the clay beyond that indicated by intrinsic behavior are aging and cementation, leaching processes, overconsolidation, and desiccation.

Laboratory tests performed using recompression techniques, wherein the clay is reconsolidated to the in-situ stress state prior to shearing, invariably reflect both stress change and mechanical disturbances; most commonly because there is some expulsion of water upon reconsolidation that decreases the void ratio from the in-situ value. The SHANSEP method (Ladd and Foott, 1974) was offered as a means to "erase" the effects of sampling disturbance. In this method the normally consolidated, or mechanically overconsolidated, state of the soil is recreated by first consolidating the clay well past the in-situ stress state onto the virgin portion of the consolidation curve.

The SHANSEP model is frequently used to characterize the undrained monotonic strength corresponding to a particular testing stress path, yielding a single value of S_u/σ_{vc}' . Table 2.1 summarizes the values of S_u/σ_{vc}' for Holocene bay mud from San Francisco from monotonic tests reported in various studies.

Study	Location	Sublayer	S_u/σ_{vc}'	S_u/σ_{vc}'	τ/σ_{vc}'	S_u/σ_{vc}'	ϕ_{tc}'	S_t
			ICU Triaxial	ACU Triaxial	ACU DSS	Field Vane	(degrees)	sensitivity
HAMILTON AIR FORCE BASE								
Noorany (1963)				.37-.40 $K_c = 2$				8-10
Singh (1966)								
Duncan (1965)			.36	.35 $K_c = 1.8$		.32	34.3 $K_c = 1.8$	
Seed & Chan (1966)				.39 $K_c = 2.5$				
Lacerda (1976)							31	
Denby (1978)			.31	.35-.38 $K_c = 1.4$			29.5 $K_c = 1.4$	
Idriss et al. (1978)			.40					
MARINA DISTRICT, SAN FRANCISCO								
Kayen et al. (1990)	Marina District Winfield Scott School	Subunit 3.1 Subunit 3.2						6
Sitar & Salgado (1993)	Marina District (same boring as Kayen et al.)	Subunit 3.1			.24 $K_c = 1.7$			
SAN FRANCISCO WATERFRONT								
Aisiks & Tarshansky (1969)	BART alignment							5-10
Dames & Moore (1989)	downtown SF	11-27 meter depth	.30	.29 $K_c = 2.1$	.23 $K_c = 1/K_0$	.23	32	3-7
BURLINGAME								
Kirkgard & Lade (1988, 1991, 1993)	near SFO		.38-.40	.26-.47 ¹			36-40 ¹	

Table 2.1 Undrained Shear Strength of Bay Mud, San Francisco Bay Area, California (normally consolidated specimens)

Study	Location	Sublayer	S_u/σ_{vc}'	S_u/σ_{vc}'	τ/σ_{vc}'	S_u/σ_{vc}'	ϕ'_{tc}	S_t
			ICU Triaxial	ACU Triaxial	ACU DSS	Field Vane	(degrees)	sensitivity
LASH TERMINAL / ISLAIS CREEK								
Duncan & Buchignani (1974)	LASH terminal					.31-.32		4-8
Clough & Reed (1984)	Islais Creek	Davidson Ave	.33				26	
DUMBARTON BRIDGE - WEST								
Lodde (1982), Stokoe & Lodde (1978)	Dumbarton " "	clayey silt grv.sand/silt blk.silt/clay						
AGNEW - SAN JOSE								
Lodde (1982), Stokoe & Lodde (1978)		brn./gr. clay						
MARE ISLAND								
Houston & Hermann (1980)	tidal flats, nr. Mare Island	2.4 meter depth	.37-.44					
¹ cubical triaxial apparatus; intermediate principal stress varied								

Table 2.1 (continued) Undrained Shear Strength of Bay Mud, San Francisco Bay Area, California (normally consolidated specimens)

Ladd and Foott (1974) recognized that SHANSEP is not applicable to clays with a high sensitivity. The sensitivity of bay mud across all sites usually falls between approximately 3 and 8, indicating moderate sensitivity. A sensitive clay may experience one or several changes in stress-strain characteristics at different points along the virgin compression line, corresponding to different levels of structural collapse, and thus the normalized strength is not a constant. Therefore, the SHANSEP procedure will give reasonable results only for clays that are relatively unstructured. For a highly structured clay, the interplay of destruction of clay structure and decrease in void ratio from stress redistribution is complex, and the choice of laboratory preparation and consolidation techniques must be approached carefully.

Denby (1978) performed a series of ACU triaxial tests on bay mud from Hamilton Air Force Base following the SHANSEP procedure at a consolidation stress ratio, K_c , of 1.4 (corresponding to an in-situ value of $K_0 = 0.7$). The effective consolidation pressures were 1.5, 2.5, and 4 times the actual overburden stress. As shown in Figure 2.2, the normalized stress-strain curves decrease with increasing consolidation stress. Denby determined an average undrained strength ratio of 0.35 for the four tests. Denby noted that a SHANSEP program requires better than average equipment, consistent technique, and control of environmental variables. Additionally, the difficulty in interpreting the preconsolidation pressure for test samples will be reflected in the test data.

For clays that exhibit normalized behavior, Ladd et al. (1977) proposed the following general equation.

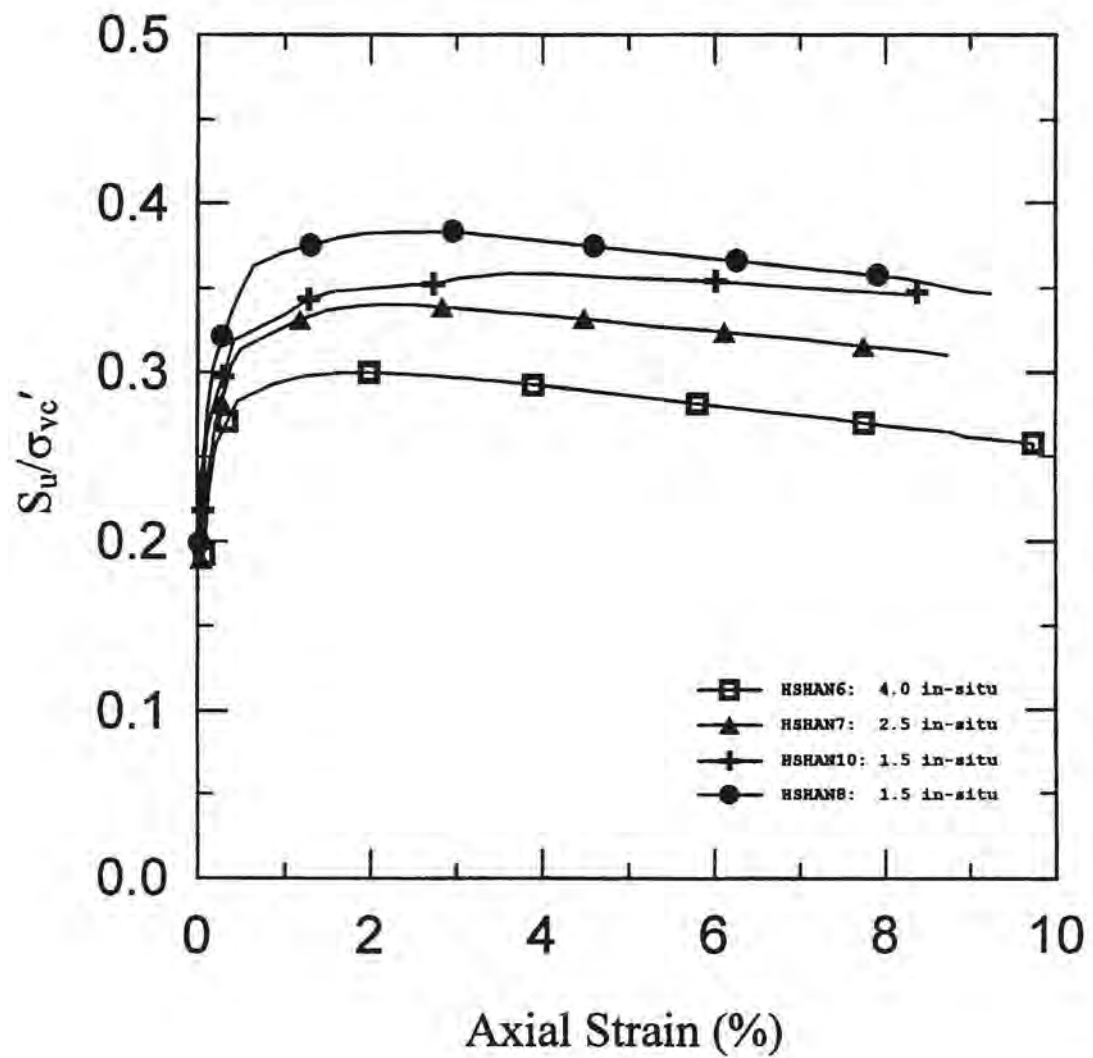


Figure 2.2 Triaxial test results for bay mud, Hamilton Air Force Base. Data reported by Denby (1978).

$$\frac{\left(\frac{S_u}{\sigma'_{vc}}\right)_{OC}}{\left(\frac{S_u}{\sigma'_{vc}}\right)_{NC}} = OCR^m \quad 2.3$$

The empirically-derived value of the exponent m is approximately 0.8, but varies somewhat with the particular clay, and a better fit occurs if m decreases with increasing OCR (Kulhawy and Mayne, 1990). The value of the exponent m determined by Dames and Moore (1989) from simple shear tests on bay mud from the MUNI Metro Turnaround Facility is approximately 0.85.

Using a data base of test results on 48 clays, Kulhawy and Mayne (1990) developed an empirical relationship between the strengths measured after isotropic and anisotropic consolidation. The tests included specimens that were consolidated under true K_0 conditions, estimated K_0 stresses, and other anisotropic stress ratios. Figure 2.3 shows this data, along with values obtained for selected bay mud sites. For most clays, the strength under anisotropic conditions was 0-20% lower than the isotropic strength. However, Denby (1978) found that the strength from anisotropically consolidated tests for bay mud from Hamilton Air Force Base was slightly higher than that obtained from isotropically consolidated tests in triaxial compression. In contrast, Duncan (1965) found it to be slightly lower, and the data collected for the MUNI Metro Turnaround Facility also shows that the strength from anisotropically consolidated tests is slightly lower than from isotropically consolidated tests.

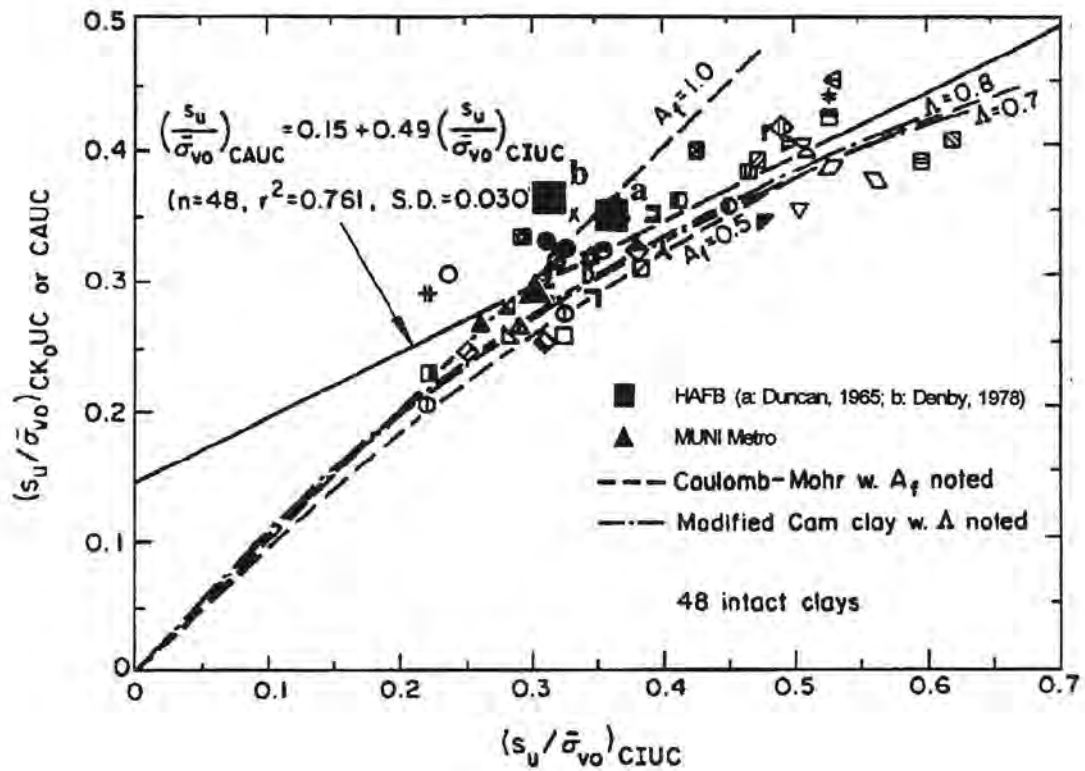


Figure 2.3 Influence of anisotropy on undrained triaxial strength of clays. Adapted from Kulhawy and Mayne (1990).

Most laboratory tests to determine the monotonic strength of clays are performed so that

failure is reached in a matter of minutes to hours; for triaxial testing, the conventional strain rate ranges from 0.5% to 10% per hour. However, it is generally recognized that tests performed at a "fast" rate of loading produce strengths that are higher than the normal "slow" tests. In a fast test, the pore pressures do not have a chance to equilibrate, while in a slow test, undrained creep becomes an issue. Because strain rate effect is a log-linear function of the rate of loading, it is also called the "viscous" effect.

Data compiled by Kulhawy and Mayne (1990) for 26 different soils shows about a 10% increase in strength per log cycle of strain rate increase (Figure 2.4). Ishihara (1981) analyzed and extended previously published data, and found an overall increase in strength of about 15% per log cycle of time to failure. Lacerda (1976) studied the behavior of bay mud from Hamilton Air Force Base over a range of strain rates (0.04%/hour to 210%/hour) and found that an order of magnitude of increase in strain rate produced an increase in maximum deviator stress of 9% in ICU triaxial compression tests and UU tests. These results were independent of confining stress.

The different stress paths, boundary conditions, and intermediate principal stresses that can be applied in monotonic shear test apparatus produce a wide but systematic range of normalized strengths, as demonstrated in Table 2.1. Kulhawy and Mayne (1990) summarized empirical data relating normalized strengths for various monotonic tests on different clays. Figure 2.5 shows relationships between effective stress friction angle in a triaxial compression test (ϕ'_{tc}) and undrained shear strength from other test types. These

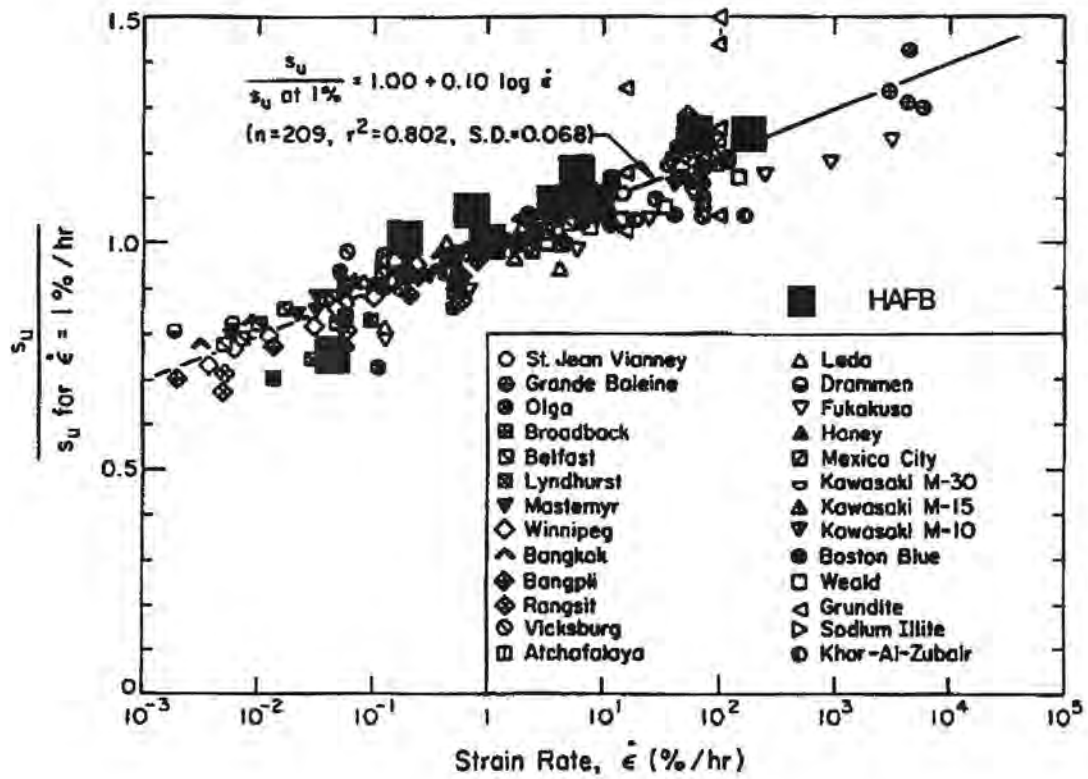


Figure 2.4 Influence of strain rate on undrained strength of clay. Bay mud data from Lacerda (1976). Adapted from Kulhawy and Mayne (1990).

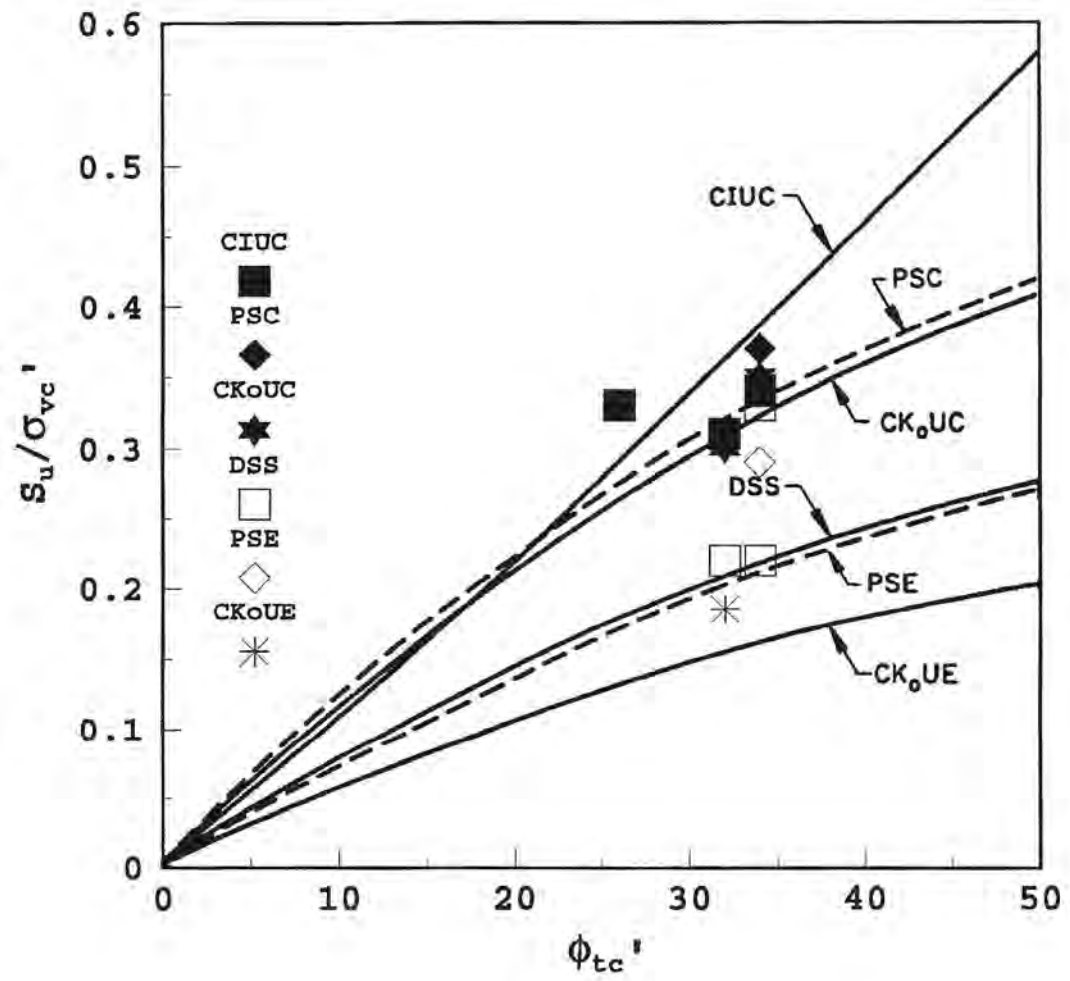


Figure 2.5 Influence of test type on undrained strength ratio of clays. Adapted from Kulhawy and Mayne (1990).

relationships were derived from linear regressions or from application of a modified Cam Clay model to empirical data (Kulhawy and Mayne, 1990). Data from tests on bay mud from various sites has been plotted on this figure. Most of the data support the relationships developed by Kulhawy and Mayne (1990), but there is some scatter to the data, particularly for the ICU triaxial and simple shear tests.

2.2.2 Response Under Cyclic Loading

Cyclic strength is influenced by the same factors which affect the monotonic strength: plasticity, sensitivity, initial shear stresses, consolidation stresses, stress history, strain rate, and test conditions. However, cyclic loading differs from monotonic loading in several significant ways. Seismically-induced strain rates are typically much higher than those in conventional monotonic loading, and the strain rate varies throughout a cycle. Partial or complete stress reversals may occur, and the loading history is typically quite irregular. Most importantly, the repetitive nature of the load causes a degradation of both stiffness and strength of the soil with increasing number of cycles and with increasing cyclic strain.

2.2.2.1 Threshold Levels

The concept of threshold level refers to the amplitude of strain or stress that separates modes of behavior in the clay. At very low strain levels, the structure of the soil is undisturbed, and linear elastic, recoverable stress-strain behavior is observed. Vucetic (1994), in a comprehensive review of threshold studies, refers to the strain which divides

essentially linear behavior from the nonlinear, as the linear cyclic threshold shear strain, denoted by γ_{li} . The shear modulus does not degrade under repeated loading below this strain level. For practical purposes, the threshold strain can be defined as the cyclic strain, γ_c , at which $G_s/G_{max} = 0.99$. The linear cyclic threshold shear strain is approximately 0.001 percent, but increases with plasticity index (Figure 2.6).

As the strain level increases from approximately 0.001 to 0.1 percent, nonlinear but still essentially elastic behavior occurs, with only small changes in pore pressure. The shear modulus essentially does not degrade in this strain range. Isenhower and Stokoe (1981) measured the shear modulus of bay mud at Hamilton Air Force Base ($PI = 45$) in a combination resonant column/torsional shear device. They found that below about 0.01 percent shear strain, the shear modulus was nonlinear but cyclic degradation did not occur, and the linear cyclic threshold shear strain was 0.001 percent.

At higher levels of strain, dilatancy behavior appears, with permanent changes to the clay structure and development of residual pore-water pressures. The stiffness may rapidly decline and failure may occur if the strains are large enough or are applied repeatedly. Vucetic (1994) identified a volumetric cyclic threshold shear strain, γ_{lv} , as "the cyclic strain amplitude above which a significant permanent volume change or a permanent pore-water pressure change may occur in the soil, while below it the soil microstructure remains practically unchanged and consequently such volume and pore-water pressure changes are negligible." Thus the volumetric cyclic threshold strain divides the nondegrading from the degrading cyclic behavior. The term "volumetric" is used to emphasize that the threshold

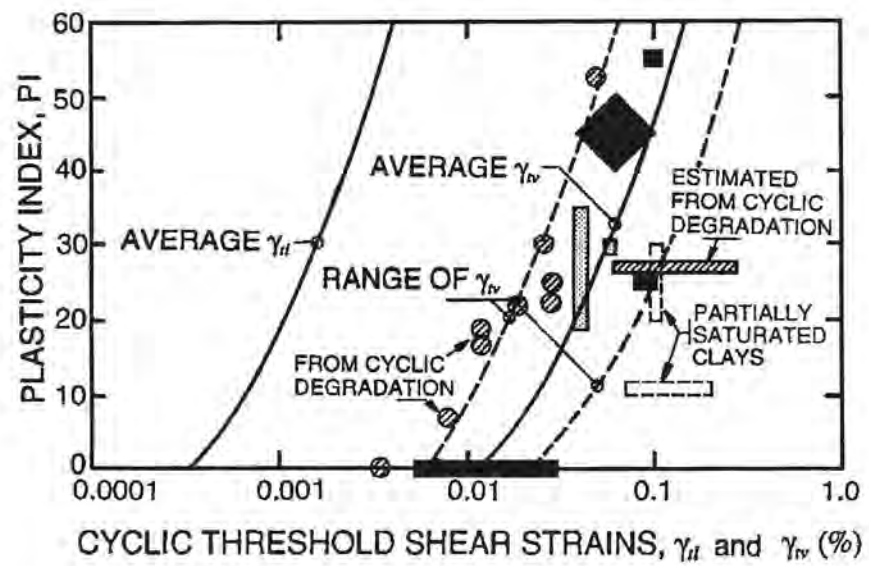


Figure 2.6 Influence of plasticity index on threshold shear strain levels. Adapted from Vucetic (1994).

level is fundamentally related to volume change behavior, or dilatancy. This threshold strain is higher for clays than for sands, and in fact shows a strong correlation with plasticity index (Figure 2.6). The volumetric cyclic threshold strain corresponds to a reduction in shear modulus of about 35 percent for all soils. Studies on clays have indicated that the volumetric cyclic threshold strain ranges from approximately 0.04 percent to as high as 0.3 percent (Ishihara, 1981; Matsui et al., 1980; Vucetic, 1994). The value of volumetric threshold shear strain of bay mud determined in the resonant column/torsional shear studies by Isenhower and Stokoe (1981) is greater than 0.02 percent.

Another threshold is the stress below which a great (or infinite) number of cycles may be applied without failure being approached. This concept is particularly useful for offshore applications, where the cyclic load from ocean waves may consist of long period, low amplitude waves of tens or hundreds of cycles. The limiting value has been called the Critical Level of Repeated Loading, or CLRL (Houston and Hermann, 1980). Houston and Hermann (1980) defined this as the limiting stress below which either residual strain or pore pressure may develop, but to a limited extent without failure (defined as 10% cyclic axial strain).

Houston and Hermann (1980) performed a series of tests on bay mud from tidal flats off Mare Island, under stress-controlled, ICU cyclic triaxial conditions. The CLRL for bay mud without initial shear stresses was determined to be about 58% of its monotonic strength. Plasticity index was found to be a significant factor when comparing the results

on bay mud with those of five other marine soils. The bay mud at the location sampled, near Mare Island, has a plasticity index of 45-68. In comparison, the CLRLs for the other five clays and their PIs are as follows: Atlantic Calcareous Ooze (3%, nonplastic); Reconstituted (Loose) Calcareous Ooze (12%, nonplastic); Pacific Calcareous Ooze (18%, nonplastic); Atlantic Hemi-Pelagic (22%, 16-37); Pacific Hemi-Pelagic (22%, 20-32%); Pacific Pelagic Clay (50%, 42-74%). These results illustrate that clays with high plasticity show an increased resistance to cyclic degradation compared with clays of lower plasticity. Thus, normalized cyclic strength tends to increase with increasing plasticity index, a result that was also found by Ishihara and Yasuda (1980).

2.2.2.2 Shear Modulus

In a monotonic test, the initial shear modulus is represented by the tangent modulus, corresponding to the slope of the straight-line portion of the shear stress versus shear strain curve. In a cyclic test, the shear modulus is defined by examining individual cycles, or hysteresis loops. The initial tangent modulus of the first loop is called the low-strain, or maximum, shear modulus ($G_{\max}$), although it is usually determined from in-situ shear velocity profiles or by low-strain vibration tests such as the resonant column test. The low-strain shear modulus corresponds to the modulus at a strain of approximately 10^{-4} percent or lower (Sun et al., 1988); $G_{\max}$ for a particular strain rate is represented as a constant below this amplitude. This reflects the linear cyclic threshold shear strain, γ_{tl} , below which the clay behaves in a linear elastic fashion. The maximum modulus value has been noted in previous studies (Anderson and Richart, 1976; Stokoe and Lodde, 1978).

However, Lango (1991) suggests that the threshold value may simply reflect the relatively low resolution of the y-axis in a semi-logarithmic plot, while Isenhowe and Stokoe (1981) observed that the threshold did not exist when tests were performed using a torsional shear device which, unlike a resonant column device, could decouple strain rate and strain amplitude effects at lower strains.

The secant shear modulus, or simply shear modulus, for a cycle is defined by the slope of the secant between the origin and the tip of the loop. In Figure 2.7, the shear modulus is represented by the equation:

$$G_t = \tau_t / \gamma_t \quad 2.4$$

The backbone curve is the trace of the tips of cyclic stress-strain hysteresis loops over a range of strains, in both directions of loading. The secant from the origin to any point along the backbone curve is the shear modulus at that strain level, thus the backbone curve describes the relationship between the shear modulus and strain. This relationship is also, more directly and perhaps more commonly, described by the modulus reduction curve. The modulus reduction curve plots shear modulus vs. strain, with the shear strain usually on a logarithmic scale and the shear modulus normalized to the low-strain modulus value. The same information is included in the modulus reduction curve and the backbone curve. A family of curves is necessary to describe the degradation of shear modulus with number of cycles, or to reflect the influence of other factors such as plasticity index. Figure 2.8 shows two interpretations of the influence of plasticity index on the modulus reduction

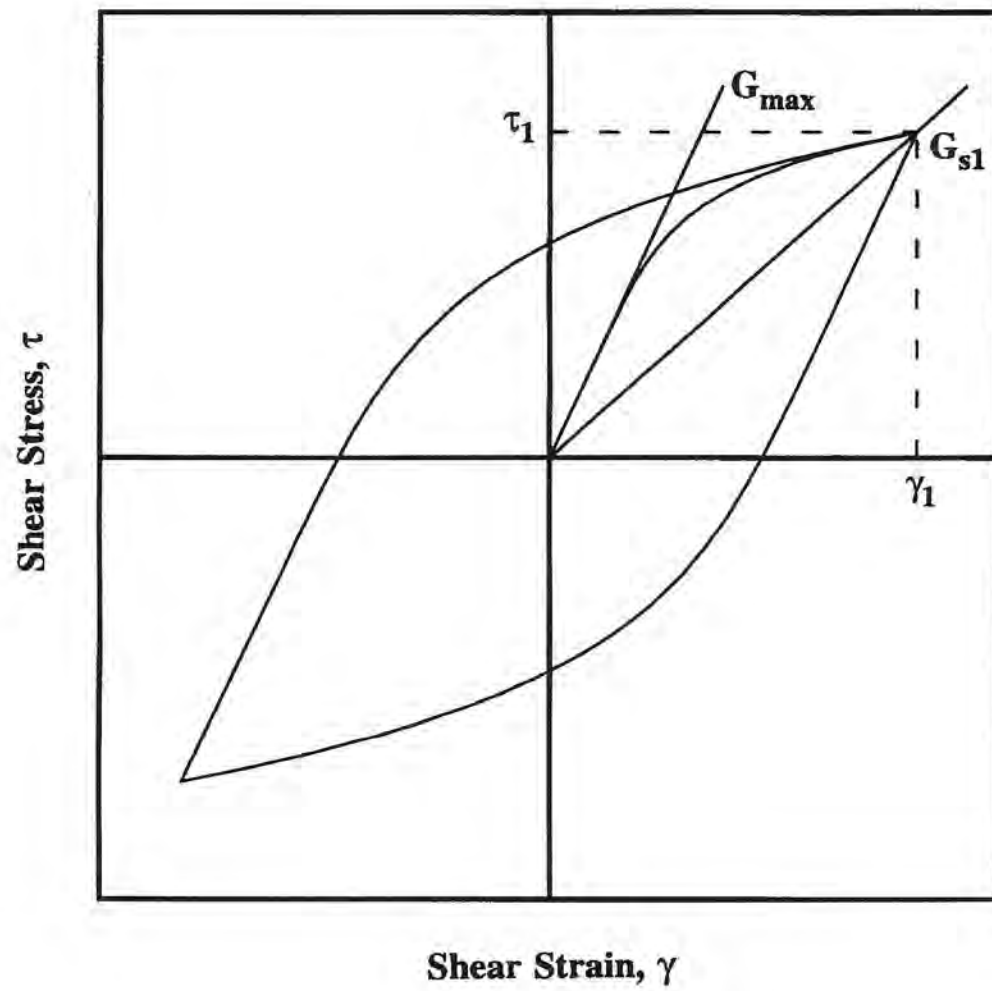
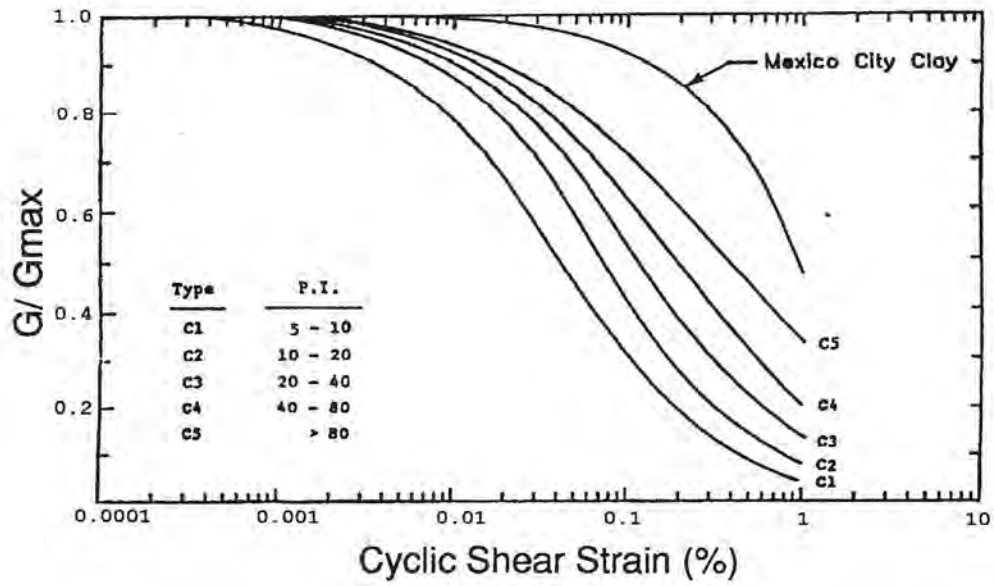


Figure 2.7 Shear modulus relationship in a cyclic test.

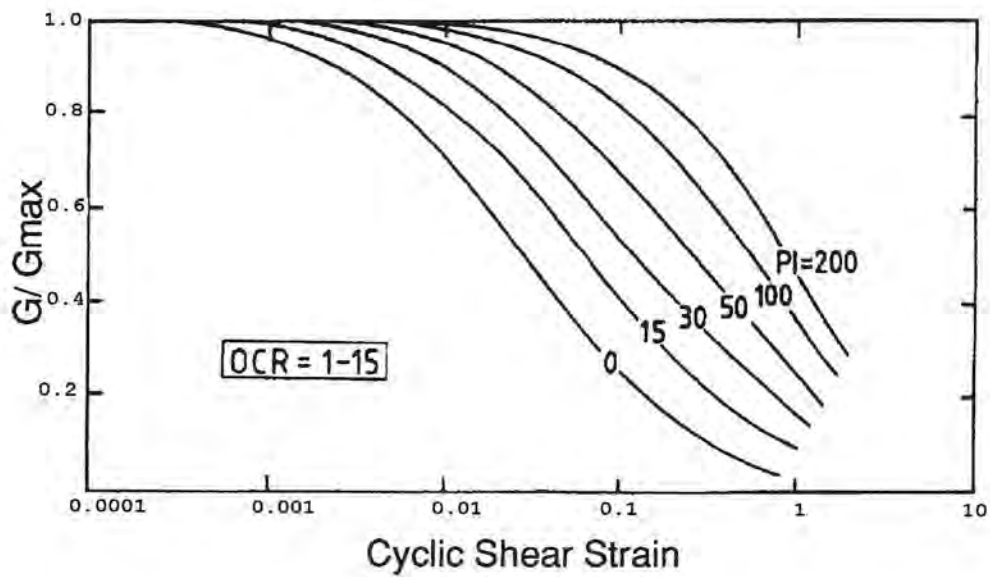
curves for clays, derived by Sun et al. (1988) and Vucetic and Dobry (1991). The two plots show similar trends with plasticity index.

A number of studies in the last two decades have presented thorough summaries of what was known about the dynamic shear modulus of clays at that particular time. These include Seed and Idriss (1970), Hardin and Drnevich (1972a), Sun et al. (1988), Lango (1991), and Vucetic and Dobry (1991).

The laboratory determined shear modulus is generally lower than field values. The factor believed to contribute the most to this effect is sample disturbance, although differences in stress states and loading paths in the field and laboratory, among other factors, also contribute to the discrepancy. Studies have shown that if the empirical relationships developed to model the modulus increase with time under consolidation load are extrapolated to an appropriate geologic time frame and used to correct laboratory shear modulus value, the corrected values are comparable to the field values. Thus, much of the degradation due to sample disturbance can be corrected for by considering aging effects (Anderson and Stokoe, 1978; Lango, 1991). The strength and stiffness of a clay that has been degraded by cyclic loading may also be regained over time, as observed by Anderson and Richart (1976). The rate of regain is slower than the time required for dissipation of excess pore pressures, suggesting that interparticle bonds need to be reformed for the clay to regain its original properties.



(a)



(b)

Figure 2.8 Influence of plasticity index on modulus reduction. (a) Adapted from Sun et al. (1988) (b) Adapted from Vucetic and Dobry (1991).

The plasticity index has been found to have more influence over the shape of the modulus reduction curve than any other factor (Zen et al., 1978; Sun et al., 1988; Vucetic and Dobry, 1991). Modulus reduction curves for non-plastic fine-grained materials resemble those for sands. The higher the plasticity index, the higher the resistance to degradation with shear strain, as shown in Figure 2.8. High plasticity soils also show a higher resistance to degradation with number of cycles. The effect of plasticity on the shape and position of the curve appears to be greater for lower plasticity soils, in the range of PI from 0 to 40 or 50, as noted by Kokusho et al. (1982).

While plasticity index is a good indicator of the shape and location of the modulus reduction curve, the curve will be influenced by other parameters. Modulus reduction curves developed for bay mud are presented in Figure 2.9. The modulus values were obtained by the resonant column and torsional shear apparatus, and the superposition of the data indicates that stress path and/or test type is a significant factor in modulus reduction.

Empirical relationships for small-strain shear modulus as a function of consolidation pressure and void ratio have been presented by among others Marcuson and Wahls (1975); Hardin and Drnevich (1972a); and Kokusho (1982). It has been suggested that the modulus at large strain amplitudes depends on clay strength, which is almost a linear function of the effective consolidation stress. Thus, the influence of effective consolidation stress increases with increasing strain. Lango (1991) suggests that for the entire strain range, the dependency on consolidation pressure increases from a square-root

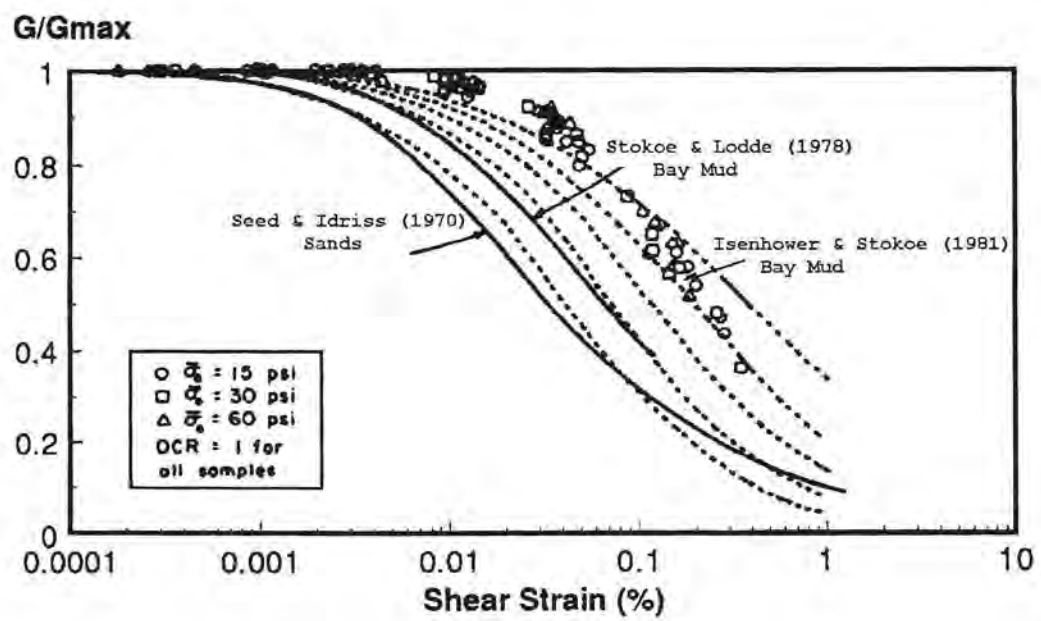


Figure 2.9 Modulus reduction relationships for bay mud. Modified from Isenhower and Stokoe (1981).

relationship at low strains to a linear relationship at high strains. Sun et al. (1988) propose that the increase in normalized modulus due to effective consolidation stress diminishes with increasing plasticity of the clay and is probably small for clays with a PI greater than about 25 and shear strains less than one percent. Similarly, Kokusho (1982) states that the effect on the modulus degradation curve due to consolidation pressure is insignificant in moderately high plasticity clays.

Lodde (1982) tested bay mud in the resonant column apparatus and found that the average modulus reduction curve for high void ratio specimens ($e > 1.8$), tested in the range of 0.01 to 0.1 percent shear strain, showed less degradation with shear strain than the curve for low void ratio specimens ($e < 1.0$). However, an inspection of the depths and locations of the samples with low void ratio indicates that differences in PI may have affected the results, and that some of the low void ratio specimens may have been tested in an overconsolidated state.

Isenhower and Stokoe (1981) investigated the effect of strain rate on the shear modulus of bay mud from Hamilton Air Force Base using a combined torsional shear and resonant column device. They found that shear modulus increases about four percent per log cycle of shear strain rate increase. These strain rate effects were demonstrated for strain amplitudes as low as 0.0001 percent. Others have shown that an order of magnitude increase in frequency results in 10% increase in $G_{\max}$ (Lango, 1991; Stokoe and Isenhower, 1980; Hicher et al., 1987). This is in contrast with researchers who have proposed that strain rate does not affect stiffness or strength below strains of 0.01 percent (Ishihara,

1981). Aggour et al. (1987) found that an increase in loading frequency increases the shear modulus across all strain values, but in the range of frequencies of interest in earthquake engineering, the effect is not significant.

The effect of long-term confinement, i.e., secondary compression, on shear modulus and damping ratio has been investigated among others by Taniguchi et al. (1978), Stokoe and Lodde (1978), and Kokusho et al. (1982). They found that the shear modulus increases as a result of secondary compression, more or less equally in all shear strain ranges, so that the shape of the reduction curve is not changed. However, Isenhower and Stokoe (1981) found that the long-term secondary compression effect for bay mud from Hamilton Air Force Base decreased with strain amplitude for shear strains between 0.01 percent and 0.1 percent.

Kokusho (1982) found that the rate of increase in shear modulus with time of secondary compression increases with plasticity index. Stokoe and Lodde (1978) suggested that the rate of increase with time during primary consolidation is mostly due to changes in void ratio, while during secondary consolidation, the rate is higher and it is due to a strengthening of the particle bonds.

Idriss et al. (1978) describe the degradation of shear modulus with cyclic loading by a degradation index, δ . This parameter represents the shear modulus at cycle N relative to the shear modulus in the first cycle of loading:

$$\delta = G_{sN}/G_{sI} \quad 2.5$$

In addition, they define a degradation parameter, t , as the rate of decrease of δ with number of cycles N :

$$t = -(\log(\delta)/\log(N)) \quad 2.6$$

The degradation parameter increases with cyclic strain amplitude and decreases with plasticity index and overconsolidation ratio.

Stokoe and Lodde (1978) noted that shear modulus decreased with number of cycles above 0.01 percent shear strain, for bay mud from near Dumbarton Bridge. The decrease in shear modulus was linear with respect to the logarithm of the number of cycles at 0.1 percent shear strain.

2.2.2.3 Damping

Beyond the elastic limit, energy is dissipated in a clay through plastic deformation, and the unloading curve follows a different path than the loading curve. The damping ratio, λ , during cyclic loading represents the total energy loss during a cycle, and is given by:

$$\lambda = \frac{1}{2\pi} \frac{\Delta E}{G \gamma_c^2} = \frac{A_c}{4\pi A_r} \quad 2.7$$

where ΔE is the energy dissipated per cycle per unit volume, given by the area within the hysteresis loop, A_C , and A_T is the area of triangle OAB , as illustrated in Figure 2.10.

Marcuson and Wahls (1978) suggested that those factors which cause a stiffening of the clay, also cause a decrease in damping ratio. Like shear modulus, the damping ratio is most significantly affected by strain amplitude. With increased applied strain, more energy is released, and the damping ratio increases. This degradation effect is commonly plotted as damping ratio versus the logarithm of shear strain. The damping ratio, like the shear modulus, is influenced by the plasticity index of the soil, as illustrated by the families of curves in Figure 2.11. The figure shows a trend of decreasing damping ratio with plasticity index, although Marcuson and Wahls (1978) show results for bentonite and kaolinite in which at low strains ($<0.0025\%$ single amplitude) the bentonite exhibits higher damping. Kokusho et al. (1982) showed some interesting results of tests exploring the effect of plasticity on the shape of the damping ratio curve: the damping ratio decreased with plasticity up to the moderate plasticity range ($PI = 40-60$); but high plasticity clays had higher damping ratios than the moderate plasticity clays. Kokusho et al. (1982) also found that the effect of plasticity was greater in the high strain range. Differences in consolidation pressure may have obscured the results.

Damping ratio has been shown to decrease with mean effective principal stress (Hardin and Drnevich, 1972a; Marcuson and Wahls, 1978; Kokusho et al., 1982). Marcuson and Wahls (1978) suggest that this influence cannot be decoupled from the void ratio decrease. Others have found that damping decreases with increasing void ratio (e.g., Hardin and

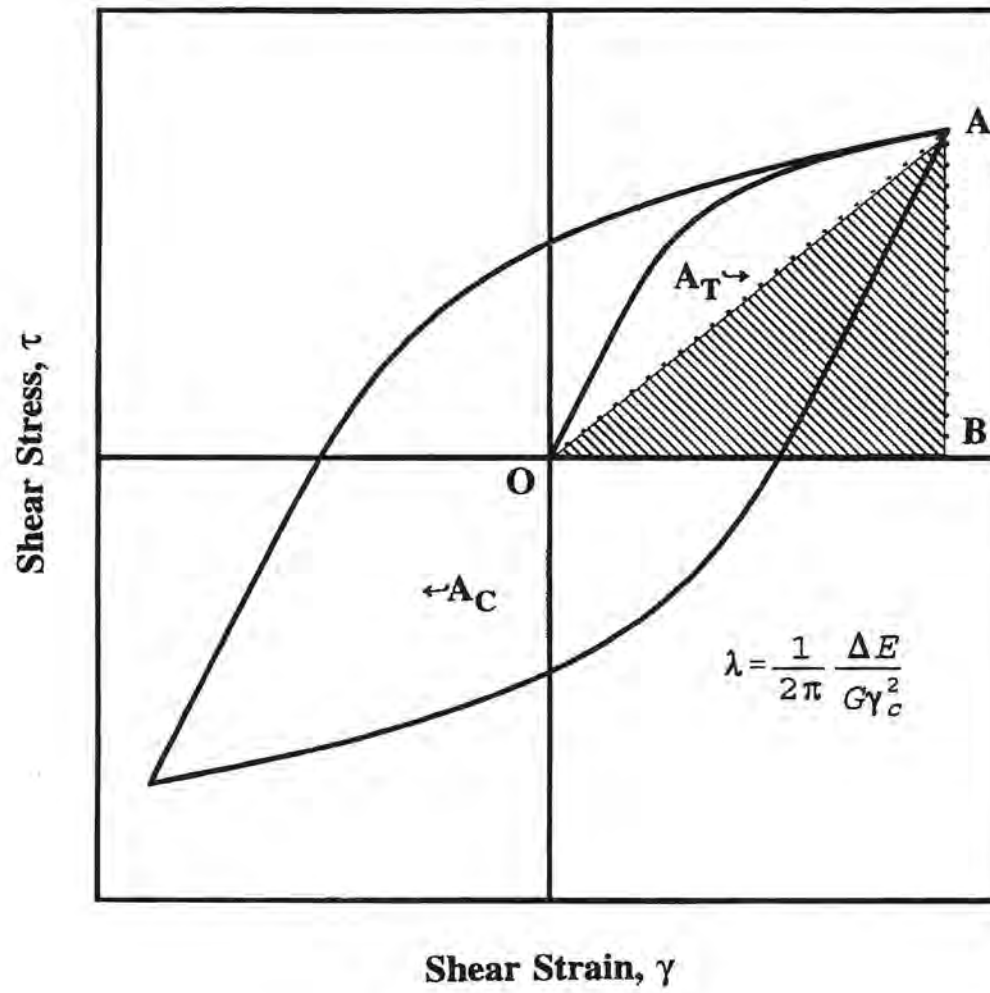


Figure 2.10 Calculation of damping ratio.

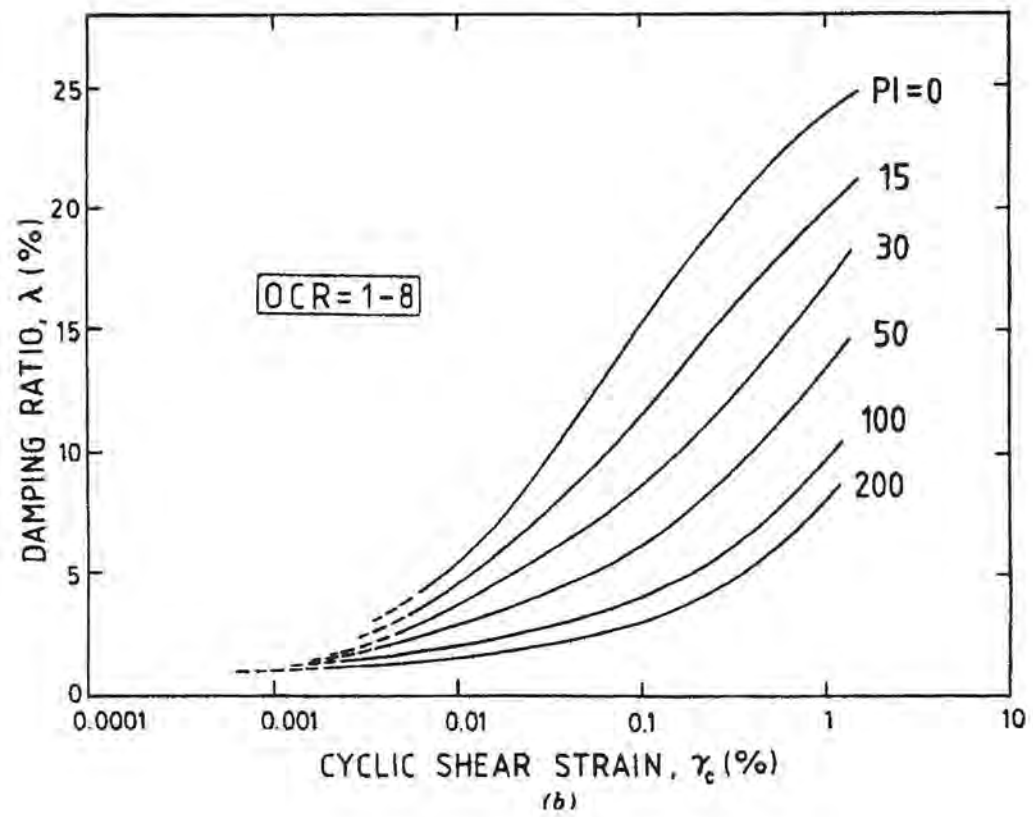


Figure 2.11 Influence of plasticity index on damping curves for clay. Adapted from Vucetic and Dobry (1991).

Drnevich, 1972a). Hardin and Drnevich (1972a) also found that the *rate* of increase of damping with shear strain increases at low effective confining stresses and high void ratios.

Stokoe and Lodde (1978) found a general trend of decreasing values of damping ratio at low strains with increasing consolidation stress for bay mud. They also found that high void ratio samples ($e_0 > 1.8$) plotted below the average curve for low void ratio samples ($e_0 < 1.0$) in the strain range from 0.02 to 0.1 percent, which appears to agree with results presented by Hardin and Drnevich (1972a); but as was mentioned in the discussion on shear modulus, these results may be better explained by differences in the plasticity index of the specimens.

Damping ratio has been shown to decrease, often linearly, with time under secondary compression (Hardin and Drnevich, 1972a; Marcuson and Wahls, 1978; Stokoe and Lodde, 1978; Kokusho et al., 1982). The decrease with time is on the order of 12 to 25% per log cycle of time past primary consolidation (Marcuson and Wahls, 1978).

2.2.2.4 Cyclic Strength

Cyclic "strength" can be defined in many ways, since in a typical cyclic test failure of the clay does not occur in the first cycle, if at all. Most often the interest is in behavior in the intermediate- to high-strain range, where the clay experiences plastic deformations and failure is possible.

In a stress-controlled test, as the number of cycles increases the peak-to-peak cyclic strains increase, eventually becoming infinite. The hysteresis loops may also migrate from the zero-strain position, usually in the direction of first loading. The average peak strain amplitude ($\gamma_{p-p}/2$) of any one cycle is the cyclic strain, and the amplitude of the migration of the cyclic strain from the zero position is the residual strain. These quantities are shown in Figure 2.12.

One definition of failure is the development of unlimited cyclic strains, determined as the asymptote to a curve of cyclic strain versus number of cycles (N). The strength of the clay is then defined as a combination of stress amplitude and number of cycles that is asymptotic to the N versus shear strain curve, as shown in Figure 2.13 for bay mud. The stress amplitude is usually designated as a cyclic stress ratio (CSR), a parameter in which the stress is normalized either by the monotonic strength or by the consolidation stress. Alternatively, failure can be defined by a smaller value of strain. Families of curves representing different failure strains could be plotted from a suite of tests as N versus CSR.

The choice of failure strain amplitude may depend on clay behavior. For example, a 3% cyclic strain level has been shown to have significance for some clays where below a certain CSR, cyclic strains would develop but then level off at or below the 3% level. If the CSR were high enough, then the accelerated cyclic strain development would begin at around the 3% cyclic strain level (Frydman et al., 1988). This is one definition of Critical Level of Repeated Loading. In other cases, the type of analysis may determine the stress or strain level of interest. For instance, in a Newmark-type slope deformation analysis, the

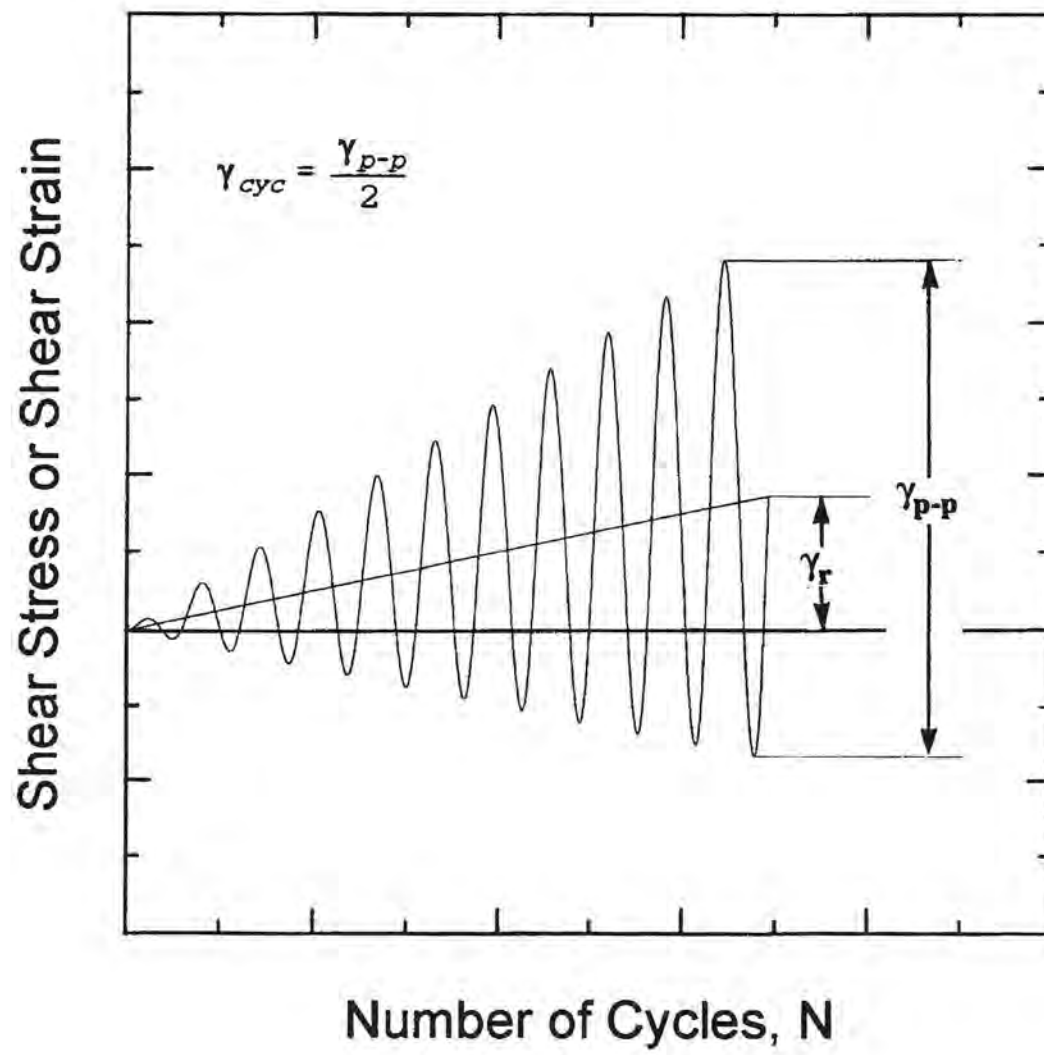


Figure 2.12 Definition of strains in a cyclic test.

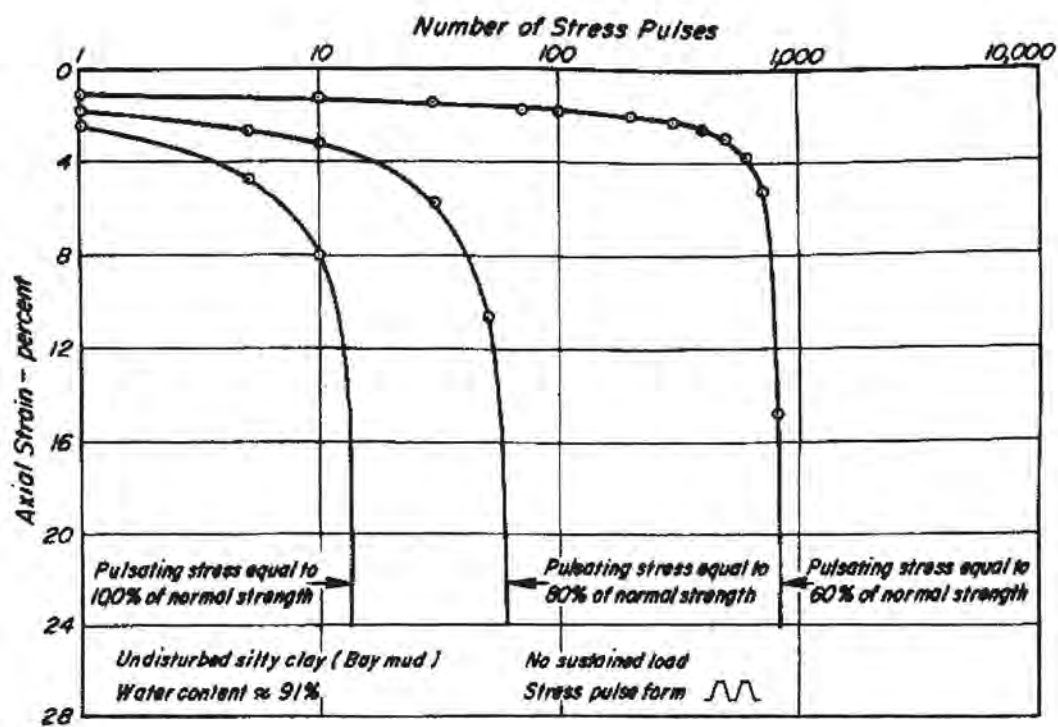


Figure 2.13 Cyclic strength curves. Adapted from Seed and Chan (1969).

strength value used for determination of the yield acceleration should be related to the stress level that initiates plastic deformation under cyclic conditions.

Defining cyclic strength for strain-controlled testing is not as straightforward as it is for stress-controlled testing. In that case, "failure" does not occur *per se*, because the strains are controlled, but a limiting value of stress will be reached as the number of cycles gets larger. The strength during each cycle of loading can be defined as the stress mobilized in the cycle at that strain level.

Plasticity is a major factor in the dynamic resistance of clays. The predominantly frictional bonds of lower plasticity clays are more easily broken than the bonds that are formed by electrochemical attraction forces in high plasticity clays, so an increase in plasticity index correlates with a resistance to cyclic degradation and pore pressure generation. However, clays that contain cementation bonds or high overconsolidation ratios tend to be brittle and reach failure with low shear strains, so generalizations based on plasticity may not be valid for these clays (Ishihara, 1981).

The relationship between sensitivity and cyclic resistance has been suggested but not conclusively shown. Sensitivity is defined as the ratio of peak undisturbed strength to remolded strength. The causes of sensitivity include metastable fabric, cementation, weathering, thixotropic hardening, leaching processes, and presence of dispersing agents (Mitchell, 1994). Practically all natural intact clays possess sensitivity to some degree. The causes of extreme sensitivity usually also produce clays that are brittle and reach peak

strength at very low strains. The leaching process associated with the development of most quick clays (sensitivity greater than 16) also reduces the plasticity; therefore, an inverse relationship between plasticity index and sensitivity might be presumed. But as noted by Houston and Hermann (1980), sensitivity and plasticity index for most soils correlate very weakly, therefore these factors are not interchangeable.

Lee (1979) reported data for tests on eastern Canadian clays with high sensitivity (380 and 35, respectively). The soils exhibited brittle failure behavior during cyclic shear, and a very thin layer of remolded soil was seen in the immediate vicinity of the failure plane. A three percent single amplitude cyclic shear strain was used to define failure in the cyclic tests. Lee (1979) plotted the results against cyclic strength curves for 19 clays investigated by Lee and Focht (1976), and showed that the normalized cyclic strengths of the highly sensitive clays are among the highest. It was noted that while the remolded strength of these soils is quite low compared to the undisturbed strength, it requires considerable effort to achieve thorough remolding. The strength after strong agitation in a plastic bag or mixing bowl was significantly lower than after a few rotations of a miniature vane. Thus, sensitivity may indicate the amount of the *potential* strength degradation during cyclic loading, but is not a measure of the peak strength or the failure strain of the soil or the energy required to break particle bonds and reach the remolded state.

Hicher and Elhosri (1981) examined the influence of clay structure by comparing the cyclic degradation in a flocculated kaolinite, to a dispersed bentonite and illite. The kaolinite with its flocculated structure was more susceptible to destructuration. The

degradation of kaolinite to 3% cyclic strain occurred in a small number of cycles (19), while illite built up cyclic strain in a large number of cycles (400). Bentonite, with its very large adsorbed water layers, required 1300 cycles to reach 3% cyclic shear strain.

Initial shear stresses will also affect cyclic resistance. In the field, stress anisotropy can develop from deposition or erosion processes, or irregular geometries of the overlying soil column. Cyclic loading will produce full stress reversal, no stress reversal, or partial stress reversal depending on the magnitude of the initial shear stresses relative to the amplitude of the cyclic load.

If a laboratory specimen is isotropically consolidated, and the cyclic load is applied by increasing and decreasing the deviator stress about the initial position, then full stress reversal occurs; that is, the shear stress on the failure plane alternates symmetrically between compression and extension (in a triaxial test). If there are initial shear stresses and they are higher than the imposed cyclic load, then even when the direction of the cyclic load is changed, the shear stress remains either in compression or extension; that is, the shear stress on the failure plane alternates between two positive (or two negative) values. This situation is known as cyclic loading with no stress reversal. Finally, if the initial shear stresses are low enough that reversal of the load also causes a change from compression to extension or vice versa, the situation is known as cyclic loading with partial stress reversal. In direct simple shear testing, all three reversal situations are possible. Initial shear can be applied through a deviator stress as for the triaxial case, and/or an additional shear stress can be applied to the horizontal plane prior to cyclic loading.

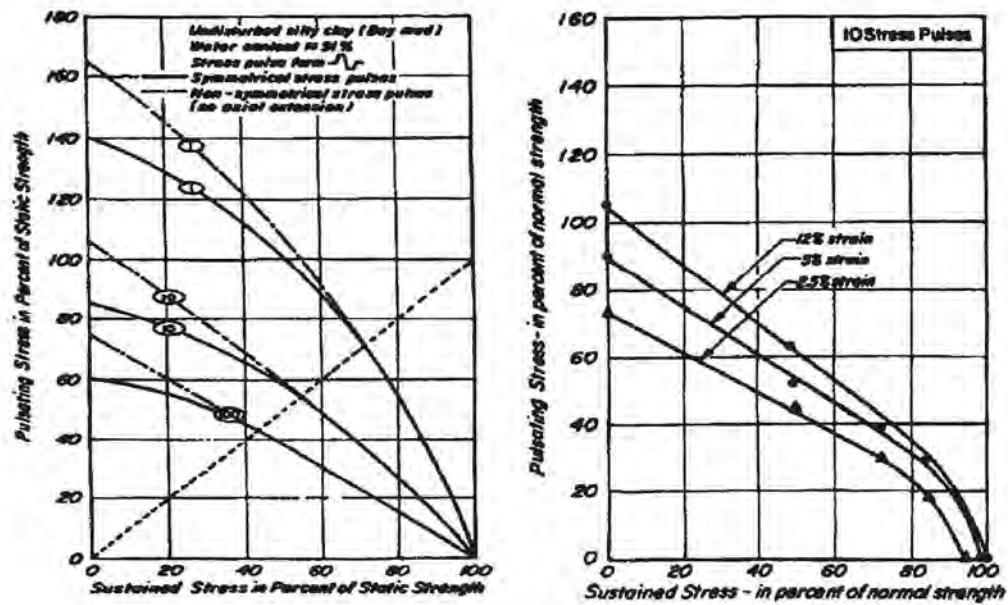


Figure 2.14 (a) Cyclic strength curves assuming failure at unlimited strain development; (b) Cyclic strength curves for different strain levels. Adapted from Seed and Chan (1966).

Many investigations have found that cyclic loading with stress reversal has a more severe impact on cyclic strength degradation than cyclic loading without stress reversal (Seed and Chan, 1966; Ishihara and Yasuda, 1980; Ishihara, 1981). Houston and Hermann (1980) found that Pacific Calcareous Ooze has a cyclic strength of 22% of the static strength in cyclic triaxial testing with full stress reversal, but this increases to 60% (combined static and cyclic stress components) when an initial shear stress of 30% of the static strength is added. They also found that stress reversal has less of an effect when the plasticity of the clay is high. Bay mud ($PI = 45$) in fact showed a slight increase in the cyclic component of strength as full stress reversal was approached.

Seed and Chan (1966) applied initial undrained shear stresses to bay mud tested in a triaxial test apparatus. They then found combinations of initial shear stress and cyclic stress needed to cause failure (defined as development of unlimited strains) as a function of number of cycles. The data in Figure 2.14a shows that the cyclic strength becomes less than the monotonic strength somewhere between one and ten cycles of loading, and that loading with stress reversal (all solid-line data above the dashed line) causes failure at somewhat lower stresses than loading without stress reversal. Figure 2.14b shows that if a less severe failure criteria is used, the strength is much lower; for example, if failure is considered to occur at less than 5% residual axial strain, then cyclic strength is lower than the monotonic strength.

2.2.3 Post-Cyclic Response

The post-cyclic behavior of clays is of interest because the loss of stiffness and strength that occurs during dynamic loading may remain long after the load is removed. A clay deposit that has adequate strength to perform well under cyclic loading may be susceptible to deformation under static load until the pore pressures generated by the cyclic loading dissipate (Seed and Chan, 1966).

Koutsoftas (1978) suggests that the loss in undrained static shear strength after cyclic loading is caused by the reduction in effective mean normal stress due to the excess pore pressures generated, while Hyde and Ward (1985) suggest that post-cyclic strength reduction is largely a strain-dependent phenomenon. Matsui (1980) demonstrated that pore pressure development and residual strain are related by a logarithmically linear relationship, indicating that these conclusions are not at odds with each other.

Thiers and Seed (1968, 1969) measured the post-cyclic strength of bay mud after 200 strain-controlled cycles in an NGI direct simple shear device and a cyclic triaxial apparatus. They concluded that 200 cycles of 2-3% shear strain reduced the monotonic strength by only about 10%, and strains greater than one percent were required to cause any significant degradation in post-cyclic strength.

Thiers and Seed (1969) compared the results of post-cyclic monotonic tests on bay mud with those on Anchorage Silty Clay. Figure 2.15 shows that bay mud, which has a

higher plasticity index ($PI = 45$, $\epsilon_f = 3-5\%$) has the highest post-cyclic strength, followed by the Zone 3 Anchorage Silty Clay ($PI = 14$, $\epsilon_f = 2.5\%$), and the Zone 2 Anchorage Silty Clay ($PI = 14$, $\epsilon_f = 1.0\%$). Zones 2 and 3 of the Anchorage Silty Clay appear to have similar properties except for the different failure strains, suggesting that other factors such as aging effects or gradation are responsible for the difference in the post-cyclic behavior.

The post-cyclic secant modulus is much more affected by cyclic loading than the ultimate strength value (Thiers and Seed, 1969). The secant modulus at 1% shear strain was taken as the reference value. This modulus was reduced as much as 40-50% after 200 cycles at 2-3% shear strain; for 1% shear strain cycles, the reduction was 20%.

Sitar and Salgado (1994) obtained almost identical results in tests on samples from San Francisco's Marina District. Ultimate strength values after 1% or 2% shear strain cycles showed somewhat of a decrease. The post-cyclic shear modulus was reduced by approximately 10-20% after the 1% shear strain cycles, and 40-50% after the 2% shear strain cycles. The Marina District samples were subjected to only 30 cycles at this high strain, while the samples tested by Thiers and Seed (1969) were subjected to 200 cycles. The similarity of the results suggests that strain level is more important than number of cycles in the degradation of shear modulus.

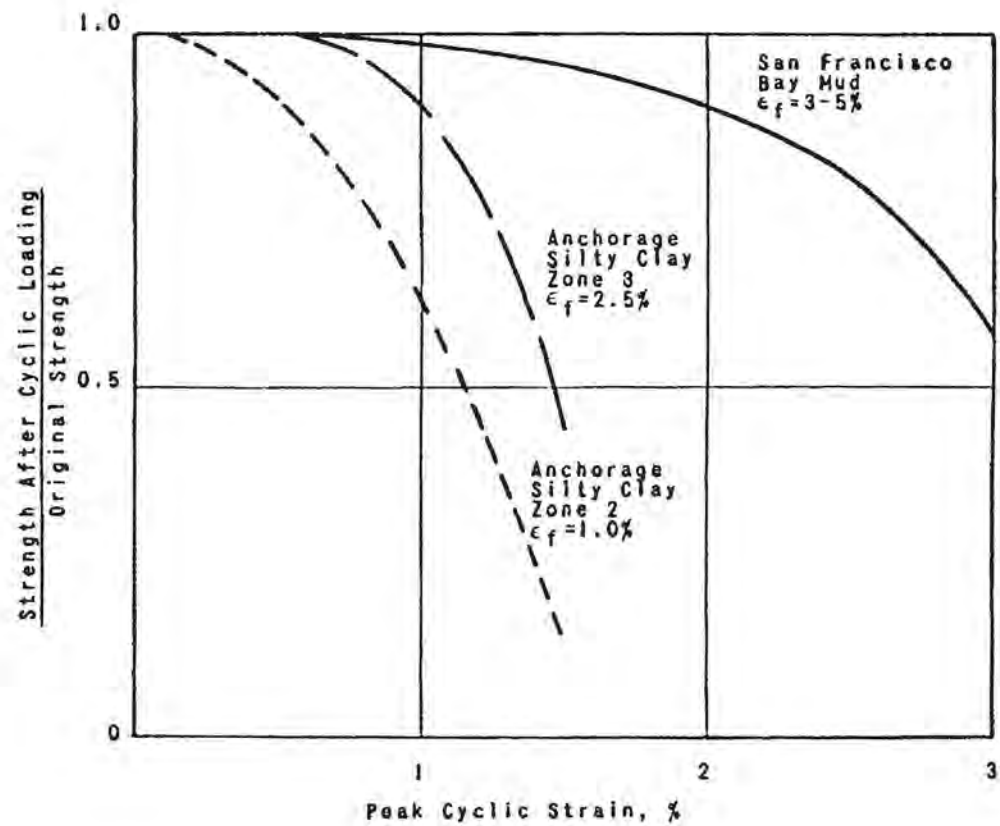


Figure 2.15 Post-cyclic normalized strength of three soils. From Theirs and Seed (1969).

2.2.3.1 Apparent Overconsolidation Hypothesis

In a typical ACU cyclic triaxial test, the clay begins with some initial isotropic effective stress due to negative pore pressures that have developed during sampling (Point A on Figure 2.16a). Then the clay is consolidated to the final anisotropic consolidation stress state (Point B on Figure 2.16a).

Cyclic loading at an amplitude high enough to cause pore pressure generation is accompanied by a decrease in mean effective stress. In an undrained test, constant volume is maintained and the void ratio of the clay is unchanged. The clay at some point during cyclic loading is represented on both Figures 2.16a and 2.16b by Point C.

If the cyclic test is stopped and a post-cyclic monotonic test is performed without allowing drainage, the available strength of the clay is represented by the stress path C-D in Figure 2.16b. This strength is lower than the strength that would have been measured if the monotonic test had been performed before cyclic loading (path B-E).

Several researchers (Matsui, et al., 1980; Hyde and Ward, 1985; Azzouz, et al., 1989; Yasuhara, 1994) have proposed that the monotonic strength measured after cyclic loading corresponds to the strength of a clay that has been initially consolidated to a higher consolidation stress (Point F on Figure 2.16b), then mechanically overconsolidated to the same effective stress as at Point C. The difference between the two clays is that the one that has been “apparently” overconsolidated to Point C by

cyclic loading, has done so at constant volume, and if drainage were allowed, a decrease in void ratio would result; whereas the clay that has been mechanically overconsolidated to Point C no longer has excess pore pressures and thus remains at Point C even with drainage. Furthermore, significant plastic straining can result in the cyclic loading case, while plastic strains are minimal in the process of mechanical unloading.

The Apparent Overconsolidation Ratio (AOCR) Hypothesis (Azzouz, et al., 1989) suggests that if the effective stresses are known at any point during cyclic loading, and the initial stresses and maximum past effective stress are known, then from a knowledge of monotonic soil behavior, the available strength can be predicted. This model applies to mechanically overconsolidated as well as normally consolidated clays. The model accounts for the change in strength due to change in effective stress in cyclic loading, but not for the degradation effects on the clay structure. It is thus likely that the model works best for high plasticity clays and at low levels of strain.

The Apparent Overconsolidation Ratio for both cyclic loading and mechanical unloading is defined by the current stress state and the corresponding point on the virgin compression line at constant void ratio, not by the maximum past pressure on the virgin compression line.

$$AOCR = \frac{\sigma'_{v, \text{virgin, at constant void ratio}}}{\sigma'_{v, \text{current}}} \quad 2.8$$

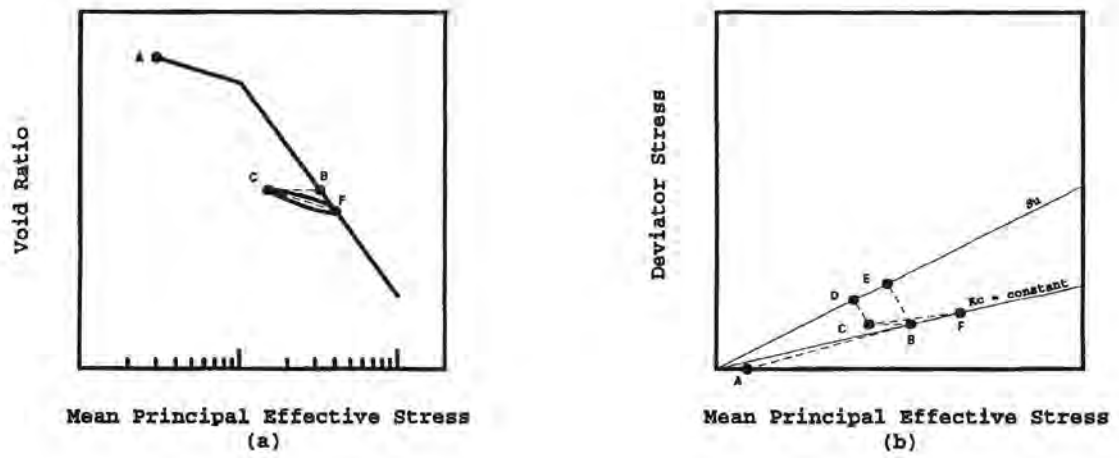


Figure 2.16 The Apparent Overconsolidation Hypothesis.

The post-cyclic strength is predicted by

$$\left(\frac{Su}{\sigma'_v} \right)_{post-cyclic} = \left(\frac{Su}{\sigma'_v} \right)_{static, NC} (AOCR)^n \quad 2.9$$

This relationship has the same form as that for predicting the monotonic strength of overconsolidated clays using the normalization procedure proposed by Ladd et al. (1977). The exponent in the above equation has been designated by the letter n to distinguish it from the SHANSEP parameter m . Based on the definition of AOCR and the assumption that at the same final effective stress prior to monotonic loading the cyclically loaded and mechanically unloaded strengths will be the same, the parameter n can be predicted from a knowledge of m and the geometry of the compression curve:

$$n = m \frac{\ln(\sigma'_{vi} / \sigma'_{v, post-cyclic})}{\ln(\sigma'_{v, max} / \sigma'_{v, post-cyclic})} \quad 2.10$$

As a predictive tool, the AOCR hypothesis will become most useful when effective stresses (or pore pressures) can first be predicted as a function of initial conditions and loading. Most techniques for predicting response to undrained cyclic loading have been formulated as total stress methods, but some models that incorporate the prediction of pore pressures during cyclic loading have been developed. These methods generally involve curve-fitting of empirical data.

CHAPTER 3 NUMERICAL STUDIES OF SHEAR STRAIN TIME HISTORIES

Holocene bay mud is found in the soil profile along much of the bay margins of the San Francisco bay area, varying in thickness from zero to 40 meters. Ground response analyses using records from the 1989 Loma Prieta earthquake confirmed that profiles containing bay mud are capable of amplifying ground motions. Peak horizontal bedrock accelerations were amplified two to three times, and spectral ordinates were amplified up to three to six times, with pronounced amplification occurring for motions at periods greater than 1.0 second (Dickenson, 1994).

During a large magnitude earthquake, soil nonlinearity may lead to a progressive softening, and while peak horizontal rock accelerations tend to deamplify at accelerations greater than approximately 0.35g, large ground displacements may occur. Dickenson (1994) calculated that the maximum acceleration that may be transmitted to Holocene bay mud layers around the San Francisco bay area is approximately 0.35g to 0.50g, depending on the soil profile. Higher accelerations may occur where a stiffer layer of soil overlies the soft bay mud. Soil nonlinearity becomes an important factor at a slightly lower acceleration level (corresponding to less than full mobilization of shear stress in the soil layers) of 0.15g to 0.30g. For comparison, the peak ground accelerations observed in the 1989 Loma Prieta earthquake at ten sites studied by Dickenson were between 0.12g and 0.33g. The thickness of Holocene bay mud at these sites varied from 4 feet to 65 feet, and the bedrock accelerations ranged from 0.07g to 0.12g. A review of response studies following the earthquake suggested that the shear

strains that were induced approached or only slightly exceeded the transition to nonlinear behavior (Dickenson, 1994). Higher bedrock accelerations, on the order of $a_{\max, \text{rock}} = 0.3g$ or higher, are expected in a design level event in the San Francisco bay area.

3.1 Analysis of Strain Time Histories from Response Analyses

A suite of analyses using the one-dimensional, equivalent linear response code SHAKE was conducted to estimate the amplitude and number of cycles of shear strain to be expected in a moderate to large earthquake in the bay area.

Two of the profiles used in the studies were chosen from those developed by Dickenson (1994), and are representative profiles for the local bay area soils. These profiles consist of a fifteen-foot layer of medium dense to dense (non-liquefiable) sandy fill, overlying a layer of Holocene bay mud, and a stiffer, older Old Bay Clay. In the first profile, the bay mud is twenty feet thick and the Old Bay Clay is 50 feet thick. In the second profile, the bay mud is forty feet thick and the Old Bay Clay is 400 feet thick. The response of these two profiles is representative of the range studied by Dickenson. A third profile representing soil conditions at Hamilton Air Force Base was also developed for this study. All three profiles with the thicknesses and soil properties for each layer are included in Appendix A.

A linearly increasing shear wave velocity function was used for the clay layers, as suggested by Dickenson (1994), based on local shear wave velocity measurements:

$$\text{Holocene bay mud} \quad V_s \text{ (fps)} = 4.2z + 180 \quad 3.1$$

$$\text{Old Bay Clays} \quad V_s \text{ (fps)} = 2.5z + 500 \quad 3.2$$

The shear wave velocity of the top sandy fill layer ranges from 700 to 800 feet per second. The shear wave velocity of the bedrock was varied over depth from 2500 to 5000 feet per second to model a weathered bedrock profile.

Input ground motions were chosen to represent earthquakes of local magnitude between 5.6 to 7.6, at varying distances from the fault rupture. The input rock motions for this study had scaled peak accelerations of 0.30g or 0.50g. A collection of recorded bedrock motions was assembled to represent earthquakes of the required magnitude at varying distances to produce the desired $a_{\max}$ levels. The motions are presented in Table 3.1 and the acceleration response spectra for 5% damping are shown in Figure 3.1.

The validity of an equivalent linear analysis to predict acceleration response for input analysis at 0.30g was verified by Dickenson (1994). Suitable consideration of the dynamic properties and limiting shear strength of the soil was found to be important. At higher levels of shaking ($> 0.30g$) the dynamic shear strength can be approached or exceeded in the analysis, and the results in this range are greatly affected by the constitutive relationship used for stiffness and shear strain. Dickenson mediated the

Earthquake	Mag. M_L	Station	Distance (km)	Comp.	a_{max} (g) ^a	T_p (sec)
Parkfield 6/27/66	5.6	Temblor	7	S25W	0.30 (0.347)	0.38
Loma Prieta 9/17/89	7	Presidio	80	90E	0.30 (0.199)	0.48
Tabas 9/16/78	7.4	Dayhook	7	Long.	0.30 (0.386)	0.2
Mexically Valey	6.4		8.5	N45E	0.50 (0.625)	0.23
Loma Prieta 9/17/89	7.25	Gilroy No.1	14	0E	0.50 (0.426)	0.19

a) scaled peak acceleration, and (recorded or synthesized peak acceleration)

Table 3.1 Rock motions used in SHAKE analyses

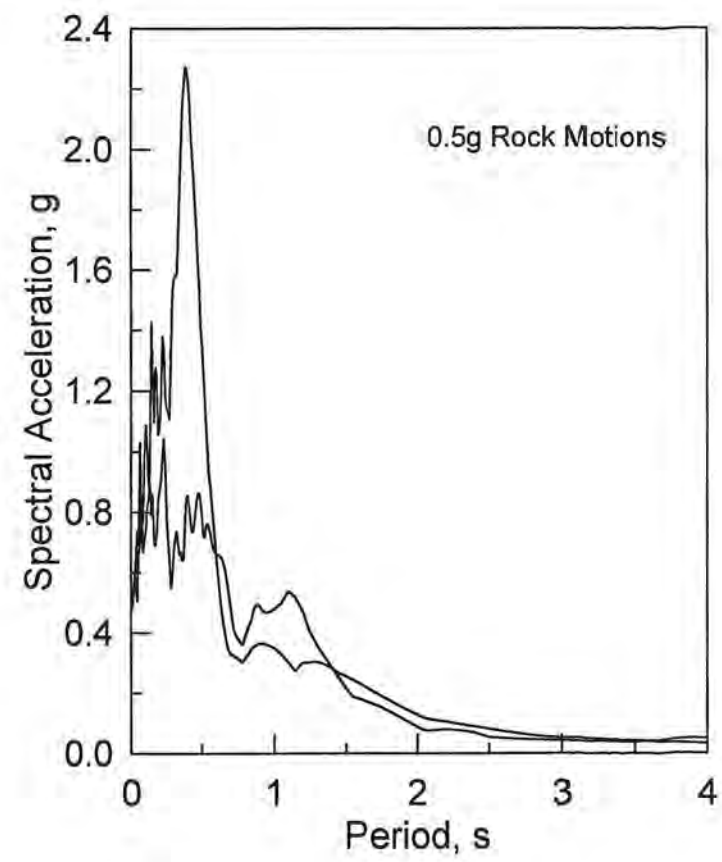
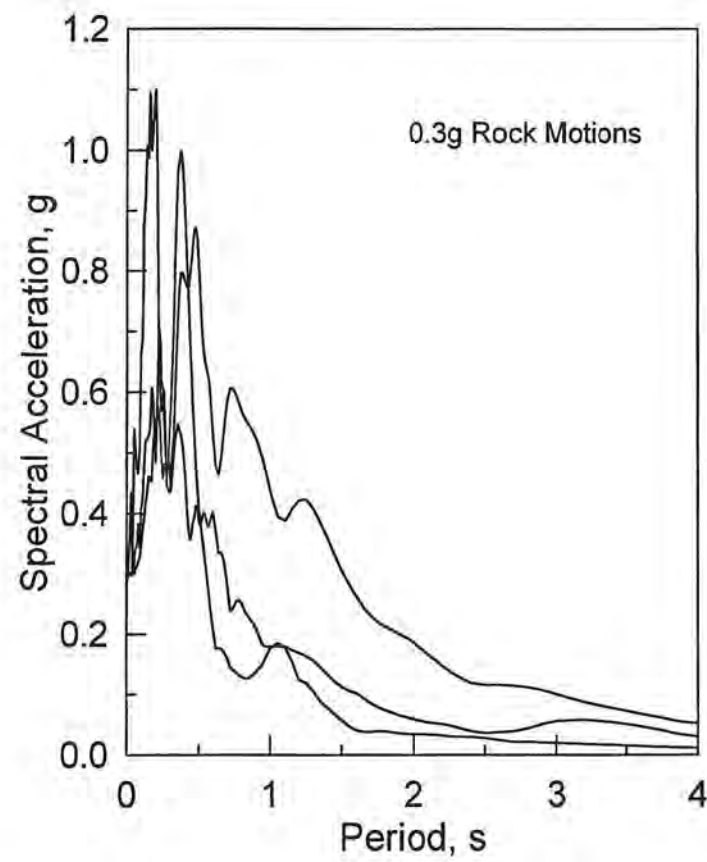


Figure 3.1 Response Spectra for Bedrock Motions Used as Input for Response Analyses

problem of excessive shear stresses in the soil layers by using a “modified” relationship between $G/G_{\max}$ and shear strain to compute the “correct” acceleration response. In the present analysis, since shear strains were desired instead of acceleration response, the $G/G_{\max}$ relationships were used unmodified.

Table 3.2 shows the results from the response analyses. The number of peaks (not cycles) within the specified ranges of shear strain was tallied from the shear strain time histories of the soil layer that exhibited the greatest peak shear strain amplitude. The results show that both the soil profile and the input rock motion have an effect on the amplitude and number of cycles of shear strain in bay mud during a moderate to large earthquake. Strains greater than 1% were observed in seven of the twelve analyses. Ten peaks greater than 1% shear strain (equivalent to five “cycles”) were observed in the analysis of Profile I with the Tabas Dayhook input motion scaled to a peak acceleration of 0.5g.

The results of the numerical study indicate that Holocene bay mud may experience shear strain levels exceeding 1% in a moderate to large earthquake, and that the number of cycles of loading greater than 0.5% shear strain could be from zero to ten or more. Therefore, cyclic tests that are designed to capture the seismic response of clays must be performed at high shear strain amplitude and number of cycles to accurately reflect the expected conditions.

0.3g motions		Number of peaks within cyclic strain range				
		0.25-0.5%	0.5-0.75%	0.75-1.0%	1.0-1.5%	1.5-2.0%
PROFILE I: 15 m fill - 20 m bay mud - 50 m old bay clay						
LP Presidio	8	2	2	1	1	
Tabas Dayhook	24	5	0	0	0	
Parkfield Temblor	5	2	2	0	0	
PROFILE II: 15 m fill - 40 m bay mud - 400 m old bay clay						
LP Presidio	10	0	1	1	1	
Tabas Dayhook	21	0	0	0	0	
Parkfield Temblor	8	1	1	0	0	
PROFILE III: Hamilton Air Force Base						
LP Presidio	0	14	14	5	2	
Tabas Dayhook	31	6	1	0	0	
Parkfield Temblor	7	3	2	1	0	
0.5g motions		Number of peaks within cyclic strain range				
		0.25-0.5%	0.5-0.75%	0.75-1.0%	1.0-1.5%	1.5-2.0%
PROFILE I: 15 m fill - 20 m bay mud - 50 m old bay clay						
LP Gilroy No. 1	5	1	1	3	0	
Tabas Dayhook	11	5	11	4	6	
Mexically Valley	6	8	2	4	0	

Table 3.2 Summary of shear strain time history analyses.

CHAPTER 4 DEPOSITIONAL HISTORY AND CHARACTERIZATION OF HOLOCENE BAY MUD FROM SAN FRANCISCO BAY

4.1 Depositional History and Distribution

Holocene bay mud, known locally as San Francisco Bay Mud, is the youngest deposit in a sequence of sediments infilling a structural depression occupied by the San Francisco Bay. Treasher (1963) suggested that the bay mud was deposited during Holocene in an estuarine environment, and referred to the unit as "younger bay mud" to distinguish it from similar deposits that originate during the Pleistocene. He identified two members, a "soft" member and a "semiconsolidated" member. The soft member (called member A-1 by Trask and Rolston, 1951) is a soft mud with occasional interbedded thin sand layers. The underlying semiconsolidated member (member A-3, Trask and Rolston, 1951) is a silty clay, firmer and less sandy than the soft member, and with a higher apparent state of overconsolidation, higher unit weight, lower water content, and higher compressive strength. Atwater (1977) assigns this member to an earlier alluvial regime rather than the younger estuarine bay mud. The upper unit coarsens from west to east along a southern crossing of the bay, Alameda to north of Hunter's Point. A third member (A-2), a fine- to medium-grained silty sand, is sometimes sandwiched between the two clay members.

Deposition of bay mud began when the rising sea level, caused by melting of the late Pleistocene (Wisconsin) glaciers, reached the entrance to the present bay about 11,000 years ago (Atwater et al., 1977). During the Wisconsin glacial stage and low water level, streams had eroded valleys in the underlying Merritt Sands and San Antonio formation. The bay mud subsequently filled the depression in this eroded surface. Evidence suggests

that the valley floor has been subsiding in the southern part of the bay since before the Wisconsin glacial stage (more than 200,000 to 300,000 B.P.). In addition to creating the structural depression, the subsidence has ensured that the underlying formations were not completely eroded during the low sea level stand (Atwater et al., 1977). The rising sea entered the Golden Gate about 10,000-11,000 years ago. Between 9,500 and 8,000 years ago, sea level rose about 2 cm/yr, advancing landward about 30 m/yr along a stream draining the southern San Francisco Bay and reaching what is currently Menlo Park by 8,000 years ago. From 8,000 to 6,000 years ago the rate decreased approximately ten-fold, and from 6,000 years ago to the present it has risen 0.1 - 0.2 cm/yr (Atwater et al., 1977). As a result of the tectonic subsidence and differential erosion of the underlying deposits, the thickness of bay mud varies from zero to as much as 40 meters (Rogers and Figuers, 1991). Structure contours of the base of the bay mud are presented in Figure 4.1, and contours of the thickness of the bay mud in Figure 4.2.

Along the western edge of the bay the sediments were deposited in an increasingly more saline environment with younger age. Atwater et al. (1977) suggests that the increase in salinity towards the top of the bay mud is caused by landward migration of the sea and growth of the estuary, which caused a greater proportion of saltwater to enter the bay. As a result the depositional environment for bay mud has not remained constant in time. In addition, there is significant spatial variability in the thickness and character of the deposits, depending on its position relative to the bay margins. Thus, there can be a variation in properties within even a single layer over short distances, similar to the variability of most natural soil deposits.

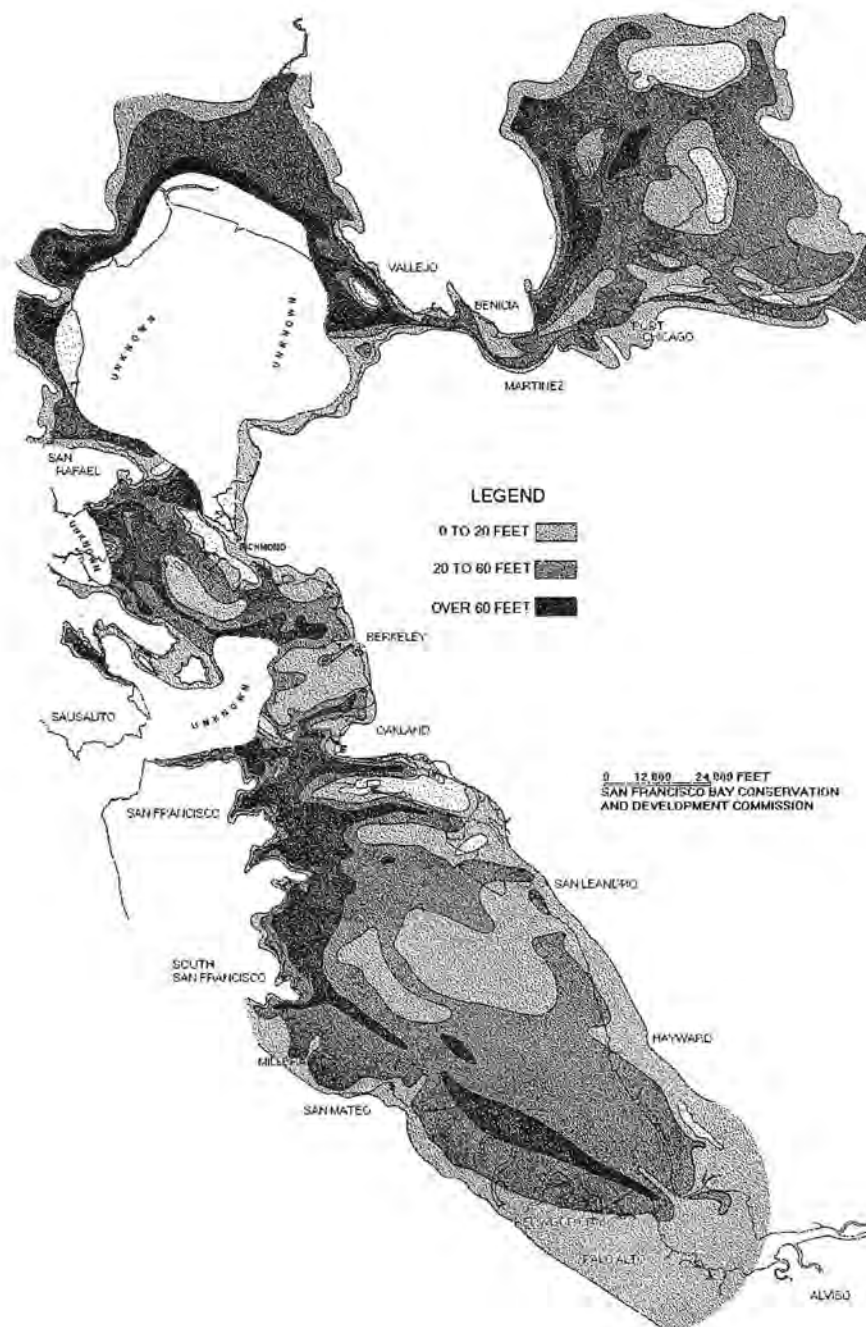


Figure 4.2 Thickness contours of bay mud. Adapted from Goldman (1969).

4.2 Distribution and Variability of Engineering Characteristics

A recent study performed by Dames and Moore (1989) on the MUNI Metro Turnaround Facility project in San Francisco illustrates the variability in engineering properties from different depositional regimes. The bay mud along the project alignment is overlain by 5 to 12 meters of fill that was placed along the bay margin, mostly during the 19th century. The thickness of the bay mud increases towards the waterfront, from approximately 18 meters to over 27 meters, with a maximum thickness of 37 meters.

The bay mud in this area contains several layers that are found consistently across the site. At the top of the profile is a layer one or two meters thick that is non-sandy, of high plasticity, and has an average unit weight of approximately 15.7 kN/m^3 . Below is a layer approximately six meters thick that contains variable amounts of sand, either mixed with the finer fractions or present as thin fine sand layers. This layer has a slightly higher average unit weight of 17.1 kN/m^3 , and slightly higher average undrained shear strengths from vane shear tests and UU triaxial tests. The bay mud below a depth of approximately 18 meters contains only traces of sand and appears quite uniform, with an approximate average unit weight of 16.6 kN/m^3 .

The undrained shear strength profile generally shows monotonically increasing strength with depth, as seen in the vane shear strength profile of Figure 4.3a. The exceptions are the sandy layers and layers that have been further consolidated by the presence of a buried seawall 125 years old, which have both higher average strengths and more variable

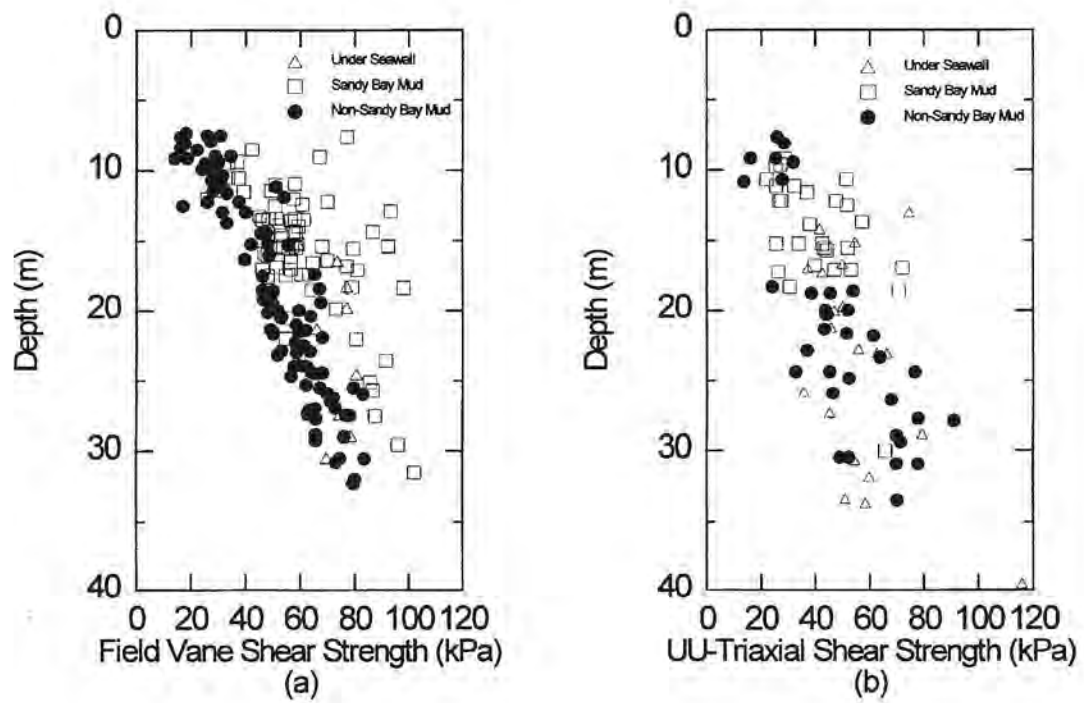


Figure 4.3 Field vane and UU-triaxial shear strength profiles at MUNI Metro Turnaround Facility, San Francisco. Data from Dames and Moore (1989).

strengths. If the test data on bay mud with sand and bay mud underneath the seawall are eliminated, a significantly more uniform strength profile is obtained; for example, approximately 65% of the data falls within $\pm 20\%$ of the average value if the full data set is used, while 86% of the data falls within $\pm 20\%$ of the average value if only non-sandy bay mud outside the seawall is used. In addition, Figure 4.3b shows that UU-triaxial strength data is not adequate for identifying these influences, due to the stronger influence of sample disturbance on the test results.

Table 4.2 on the next two pages contains a summary of the index properties of bay mud collated from various studies performed between 1966 and the present, including those compiled by Bonaparte and Mitchell (1979). The table represents a large database of test results on bay mud. Bay mud generally behaves according to the principles established for other clays; however, differences in local depositional environments and site history have produced a significant variability in index characteristics, strength, and stress-strain behavior among the samples of sediments classified as bay mud. Therefore, it is important to recognize that “representative” bay mud characteristics are representative only of selected well-studied zones, and do not represent the entire deposit.

Figure 4.4 shows the variability of the bay mud from various sites in terms of Atterberg Limits. As can be seen, while there is a clustering of points in the range of liquid limit from 50 to 70%, some samples have liquid limits as low as 37%, while some are as high as 107%. Plasticity index varies from a low of 14 to a high of 64.

Study	Location	Thickness of Bay Mud, meters	Sublayer	Sample Depth meters	Liquid Limit	Plasticity Index	In-situ w/c, %	Liquidity Index	Unit Weight kN/m ³	Silt/Clay %
HAMILTON AIR FORCE BASE										
Singh (1966)		18			88-90	45-55				--/60 ²
Duncan (1965)		18			88	45	88		14.6-15.1	
Seed & Chan (1966)		18			88	45	91			
Thiers & Seed (1968, 1969)		18		4-7	88	45	85-96			
Lacerda (1976)		18		5-8		50	88-93			41/59 ²
Denby (1978) ³		18		0-18		34-51	52-99	.44-1.3	13.5-15.6	
Idriss et al. (1978)		18			81	49	89-95		14.5-14.6	
MARINA DISTRICT, SAN FRANCISCO										
Kayen et al. (1990)	Marina District Winfield Scott School	4	Subunit 3.1 Subunit 3.2	7-9 9-12	59 --	30 --	48 36-55	0.63 --	17.1 16.7	67/33 21/12
Sitar & Salgado (1993)	Marina District (same boring as Kayen et al.)	4	Subunit 3.1	7-9	55	25	52-55	.88-.99	16.7	
SAN FRANCISCO WATERFRONT										
Aisiks & Tarshansky (1969)	BART alignment				86	62				
Dames & Moore (1989)	downtown SF			6-37	35-70	15-40	50	0.8	16.5	
BURLINGAME										
Kirkgard & Lade (1988, 1991, 1993)	near SF Airport	9-15		6	85	37	98	1.4	14	40/60 ¹

Table 4.1 Index Properties of Holocene Bay Mud, San Francisco Bay Area, California

Study	Location	Thickness of Bay Mud, meters	Sublayer	Sample Depth meters	Liquid Limit	Plasticity Index	In situ w/c, %	Liquidity Index	Unit Weight kN/m ³	Silt/Clay %
LASH TERMINAL / ISLAIS CREEK										
Duncan & Buchignani (1973)	LASH terminal	24-30		6-15 15-30	55-60 40-55	20 20	75-80 58-60	1.6-2 1.2-1.6	14.9-16.5 14.9-16.5	
Clough & Reed (1984)	Islais Creek Channel	12-30	upper YBM organic layer lower YBM	5-9 9-11 11-21	95-105 -- 68-85	59 -- 38-45	85-100 92-154 55-84	0.7-0.9 -- 0.7-0.9	14.3-14.5 13.5-14.9 15.1-16.3	
DUMBARTON BRIDGE - WEST										
Lodde (1982), Stokoe & Lodde (1978)	Dumbarton " " Ravenswood		"clayey silt" "grav.sand/silt" "blk.silty clay"	1-5 24-29 3-10	67-98 22-28 50-89	33-63 -- 19-51	87-93 6-22 64-92	.65-1.6 -- 0.6-1.8	11.8-14.8 17.1-21.4 15.6	-- -- 68/32
AGNEW - SAN JOSE										
Lodde (1982), Stokoe & Lodde (1978)			"brn./gr. clay"	3-5	46-50	20-27	32-33	0.3-0.4	18.9	39/42
MARE ISLAND										
Houston & Hermann (1980)	tidal flats, nr. Mare Island				94-100	48-58	68-129			60/36 ¹
¹ silt = 0.074 mm-0.005 mm; clay = < 0.005 mm										
² silt = 0.074 mm - 0.002 mm; clay = < 0.002 mm										
³ includes test data from upper dessicated zone										

Table 4.1 (continued) Index Properties of Holocene Bay Mud, San Francisco Bay Area, California

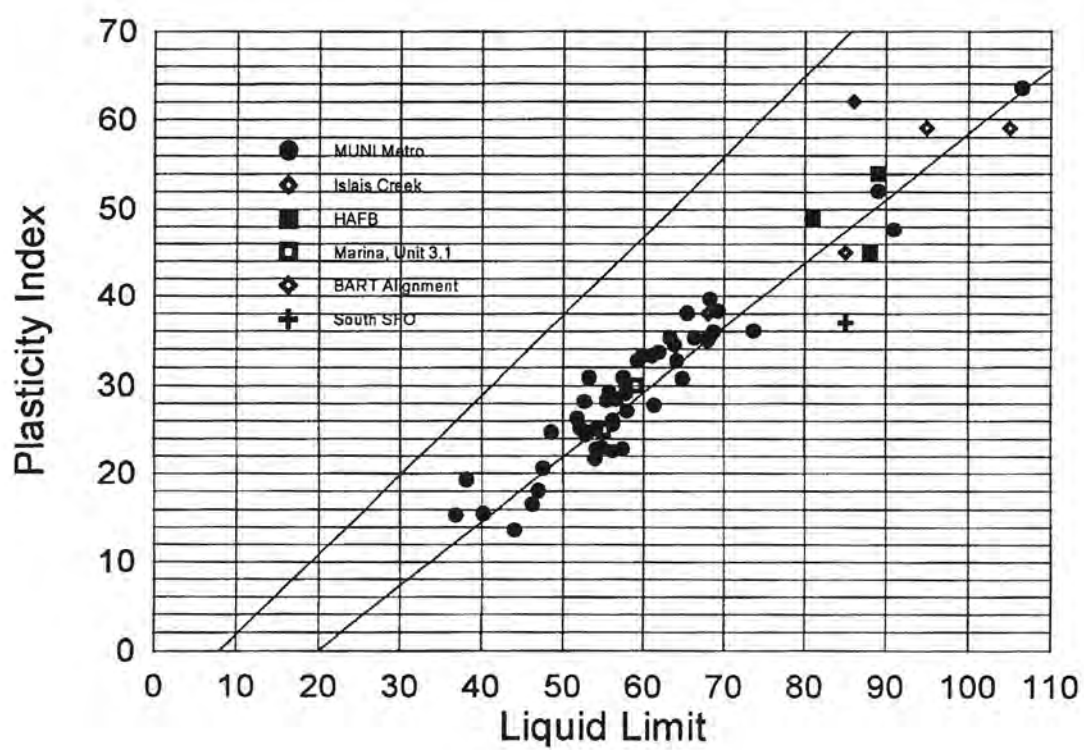


Figure 4.4 Plasticity chart for bay mud, various sites and depths.

4.3 Holocene Bay Mud at Hamilton Air Force Base

Hamilton Air Force Base (HAFB) is located on the western shore of San Pablo Bay, in Marin County, California (Figure 4.5). A site near the rifle range has been used since the 1960's as a geotechnical test site for research and classroom activities. The site has also been included as one of 40 National Geotechnical Experimentation Sites in the database funded by the National Science Foundation and Federal Highway Administration. All specimens used in this study were sampled from the HAFB test site.

Bonaparte and Mitchell (1979) summarized the monotonic behavior of bay mud from Hamilton Air Force Base obtained from a variety of laboratory and in-situ tests. In-situ tests included the vane shear test, cone penetration test, self-boring pressuremeter, and Glotzl load cell. Laboratory strength tests consisted mostly of various types of triaxial tests in addition to laboratory vane tests and plane strain tests. Index test and compression test results were also summarized. Missing from the survey is any data from simple shear tests or from dynamic or cyclic testing, although information on creep tests was included.

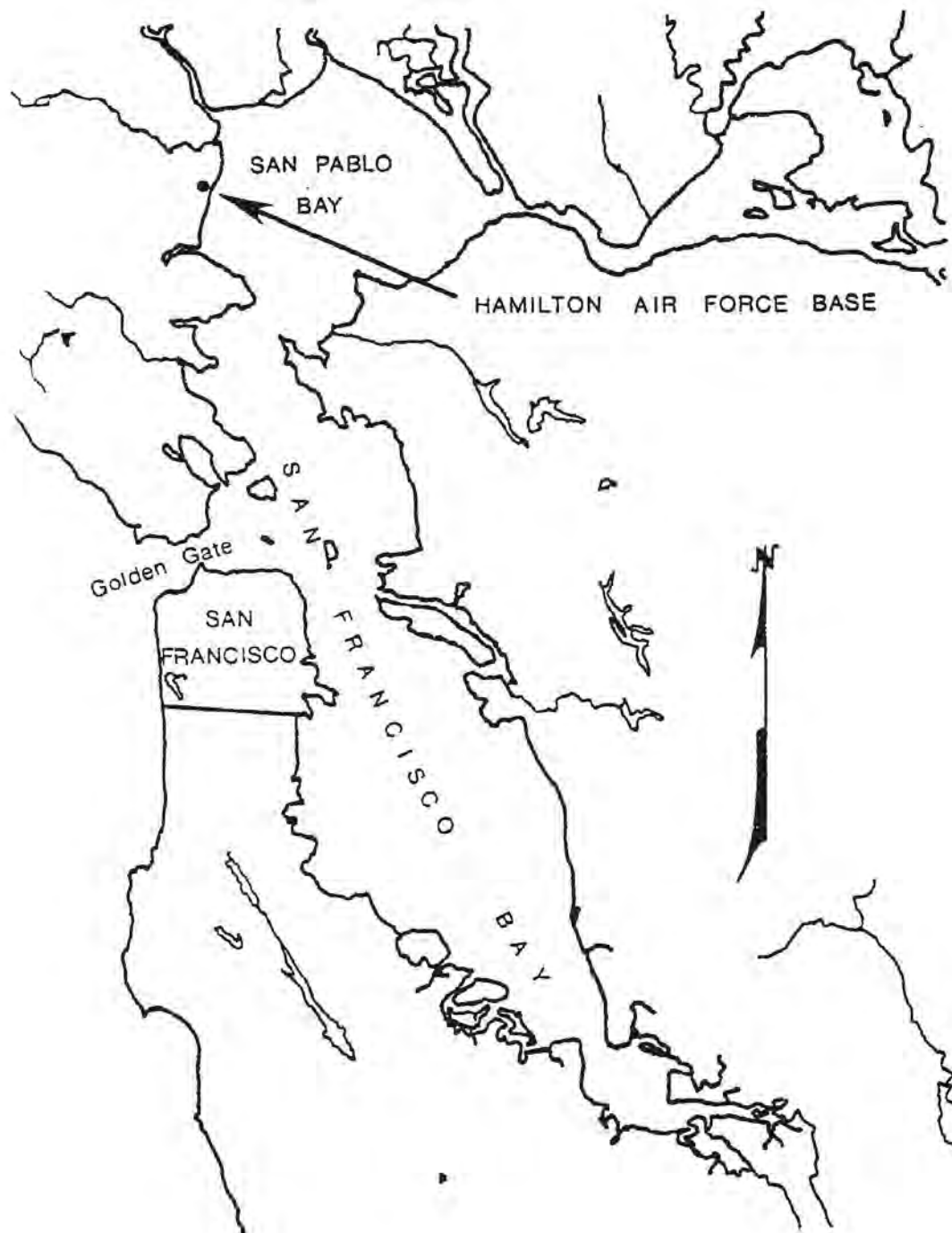


Figure 4.5 Geotechnical test site at Hamilton Air Force Base. Adapted from Isenhower (1979).

CHAPTER 5 SAMPLING, SPECIMEN PREPARATION, AND TEST PROCEDURES

5.1 Sampling Procedures and Specimen Preparation

Boreholes at Hamilton Air Force Base (HAFB) were advanced by hand-augering, and samples were obtained with a fixed-piston, hand-pushed sampler attached to 5-inch diameter, thin-walled sampling tubes. Plastic end caps were sealed to the tube ends with rubber o-rings and the tubes were then stored in a wet room until needed for testing. The specimens were extruded using a hydraulic piston, with the soil being pushed out in the same direction as during sampling to keep the shear stresses in the same direction.

5.2 Consolidation Tests

Three consolidation tests were performed, two using the incremental loading procedure and one using the constant rate of strain procedure. The purpose of the consolidation tests was to confirm the relationship between preconsolidation pressure and depth that had been established during past research and classroom activities on HAFB bay mud samples. Confidence in the preconsolidation profile was necessary for consistent reconsolidation procedures in simple shear testing. For each test, specimens were trimmed into a consolidation ring using a special trimming device to approximately 0.8 inches in height and 4 inches in diameter, discarding the trimmings or using them for a water content determination.

For the incremental loading tests, the specimen was installed in the consolidometer and a seating load was applied to prevent swell. Then the specimen was inundated with water. Increments of vertical load were applied to the specimen using a load increment ratio of 1.75, with the final load estimated to be greater than four times the preconsolidation pressure. One cycle of unloading/reloading was also applied after the specimen was well on the virgin compression curve. These load increments were applied approximately every 24 hours. For each increment, a data acquisition program recorded at prescribed intervals the time, vertical displacements, and vertical loads from a load cell and linear variable displacement transducer mounted on the specimen. After the last increment, the specimen was quickly unloaded and the final water content was determined.

For the constant rate of strain test, the specimen was installed in the consolidometer and a seating load was applied to prevent swell. Then the specimen was inundated with water. The vertical load was applied at a constant rate of displacement of approximately 0.0006 inches/minute by engaging a gear mechanism. The rate of displacement was chosen to keep the induced pore pressures in the specimen between 5 and 30% of the applied vertical stress. The load was increased until a vertical stress greater than twice the estimated preconsolidation pressure was applied. Two cycles of unloading/reloading were performed after the specimen was well on the virgin compression curve. During the test, a data acquisition program recorded at prescribed intervals the time, vertical displacements from a linear variable displacement transducer, vertical loads from a load cell, and pore pressures from an effective stress transducer mounted on the specimen. After the last increment, the specimen was quickly unloaded and the final water content was determined.

5.3 Monotonic and Cyclic Simple Shear Tests

5.3.1 UC Berkeley Simple Shear Device

The University of California at Berkeley simple shear device uses air as a chamber fluid to apply hydrostatic pressures to a membrane-enclosed specimen. An anisotropic state of stress can be obtained by applying an additional load through the vertical piston. The horizontal shear stress is applied through the horizontal piston, which moves the bottom cap along a track bearing while the top cap remains stationary. The pistons are actuated by pneumatic servovalves, which are controlled through a software program called ATS (Automated Testing System; Sousa and Chan, 1991). Drain lines exit from the top and bottom caps and connect to a volume change device, allowing for measurement of water flowing into and out of the specimen, and changes in effective stress. The chamber pressure and back pressure are both manually controlled. The simple shear apparatus is illustrated in Figure 5.1.

The quantities that can be measured in any test include chamber pressure, effective stress, volume change, horizontal and vertical displacement, vertical load, and horizontal load. The vertical displacement transducer is attached to the top of the vertical piston. The horizontal displacement transducer is mounted between the top and bottom caps, inside the chamber. Two vertical load cells are mounted beneath the specimen inside the chamber. The horizontal load cell is in series with the horizontal piston.

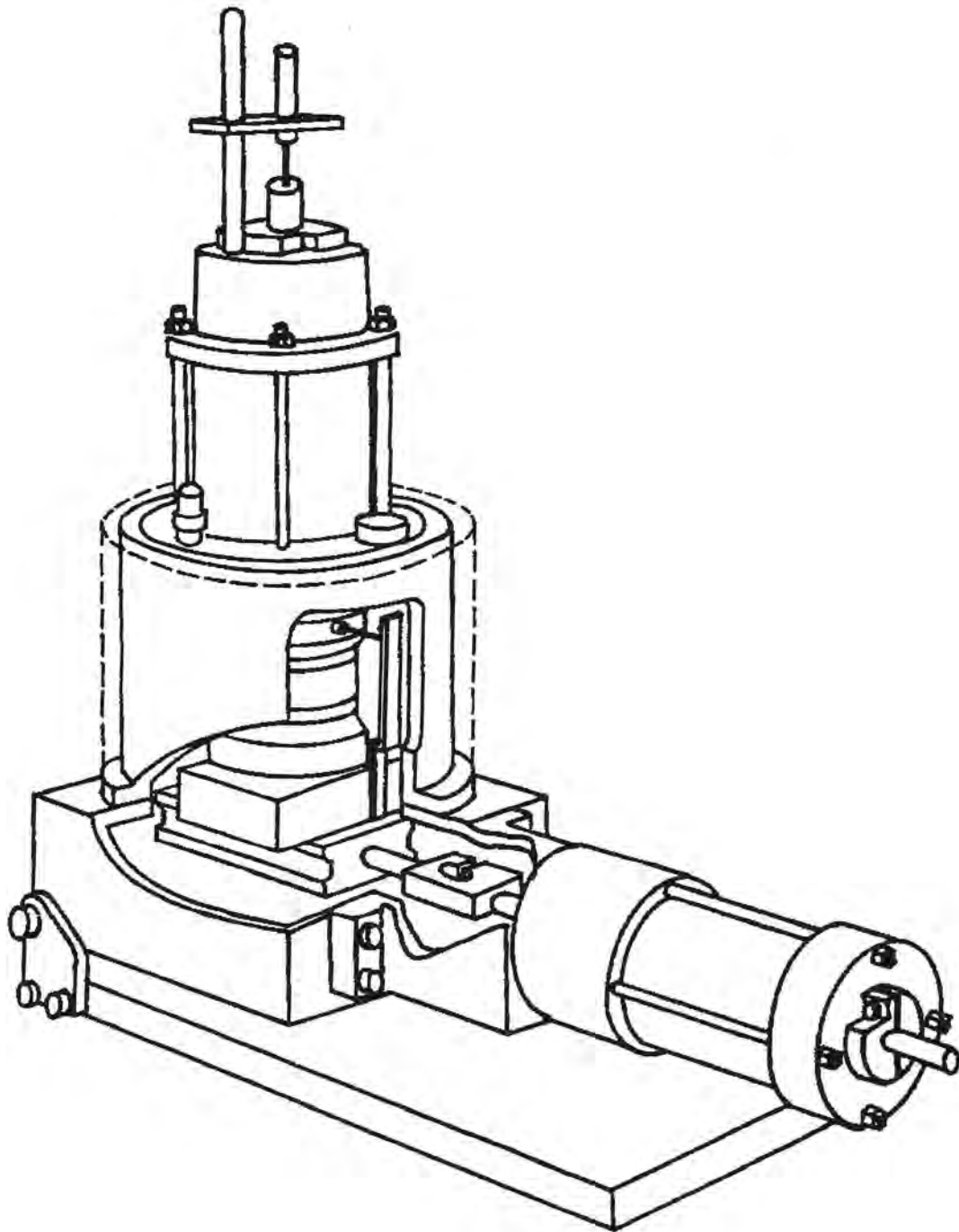


Figure 5.1 UC Berkeley simple shear device. Adapted from Rumpelt (1990).

5.3.2 Stress States in Simple Shear Tests

The simple shear test is less frequently employed in geotechnical engineering than the triaxial test, but it has certain advantages, especially with respect to studies into the response of soils to earthquake loading. The simple shear mode of deformation simulates a vertically propagating shear wave in a horizontal soil profile more faithfully than the triaxial test. In addition, the simple shear mode of deformation allows for a continuous rotation of the principal stresses, whereas in the triaxial test the principal stresses are always horizontal and vertical.

Most simple shear devices are one of two types: the Norwegian Geotechnical Institute device uses a cylindrical specimen encased in a wire-wrapped membrane and consolidated to the K_0 condition through the vertical loading piston prior to horizontal simple shear loading; the Cambridge device is a rigid rectangular box with hinges at the edges to allow for movement. Other apparatus with unique configurations have been devised for research, but not routine, purposes.

In all cases, the devices do not act in pure shear, as the shear force is applied horizontally but not on the vertical faces of the specimen. For equilibrium, the complementary shear stresses must be acting in the interior of the specimen, so the convention is to assume that the complementary shear stresses exist and are equal to the horizontal stresses. This assumption has been confirmed by stress analysis (Duncan and Dunlop (1969), Budhu (1984) and Wroth (1984)).

The basic design of the UC Berkeley Simple Shear device is similar to the NGI device, except that the specimen is encased in a flexible membrane instead of a wire-wrapped membrane. The flexible membrane allows for independent control of the horizontal (radial) stresses, at the expense of not having a strict strain-controlled boundary. Instead, the boundary is one of plane stress.

Rumpelt (1990) proposed idealized stress states for the UC Berkeley Simple Shear device, adapted from the method of interpretation for Cambridge devices developed by Budhu (1984). The interpretation neglects edge and shape effects, and assumes complementary shear stresses on the interior of the specimen, equal on average to the applied horizontal shear stress. The effective principal stresses are calculated from the known applied horizontal and vertical stresses as follows:

$$\sigma_1 = \sigma_v + \sigma_h / 2 + (((\sigma_v - \sigma_h) / 2)^2 + \tau^2)^{1/2} \quad 5.1a$$

$$\sigma_2 = \sigma_h \quad 5.1b$$

$$\sigma_3 = \sigma_v + \sigma_h / 2 - (((\sigma_v - \sigma_h) / 2)^2 + \tau^2)^{1/2} \quad 5.1c$$

5.3.3 Test Procedures

Sixteen cyclic simple shear tests were performed, with 10 post-cyclic monotonic tests. In addition, 8 monotonic tests were performed without prior cyclic loading.

For each test, specimens were trimmed using a special trimming device to approximately 1.0 inch in height and 2.8 inches in diameter, discarding the trimmings or using them for a water content determination. The specimen was enclosed in two latex membranes, with vacuum grease as an extra seal between the two membranes and underneath the o-ring seals. A vacuum of not more than one-third of the final horizontal consolidation stress was applied and the specimen was mounted inside the chamber.

To saturate not only the specimen, but also the drain lines and any pockets of air at the caps or between the specimen and the membrane, an external vacuum was applied to the chamber, and both the internal and external vacuum pressures were increased at a rate that kept the effective stress in the specimen constant. The pressures were increased until essentially a full vacuum pulled on the inside of the specimen, and this stress was left on the specimen until the air moving from the specimen was very slow or essentially stopped, in some cases overnight. Then the vacuum pressures were decreased and a positive chamber pressure was applied to the specimen without internal vacuum, again keeping the effective stress within the specimen constant. A back pressure was applied in small increments with increasing chamber pressure, and B-value measurements were taken at intervals until a B-value of 0.95 or above was achieved.

After back pressure saturation resulted in a satisfactory B-value, the specimen was taken from its isotropic stress state to its final anisotropic consolidation state. In the UC Berkeley simple shear apparatus, the horizontal stresses are controlled through the cell pressure, while the vertical stresses are controlled through a combination of the cell

pressure and vertical piston load. The lateral earth coefficient is thus chosen as one of the testing parameters. This is in contrast with the commonly used Norwegian Geotechnical Institute (NGI) apparatus, which controls the vertical stresses through the vertical piston load, while the horizontal *strains* are held essentially zero by the confinement of a wire-wrapped membrane around the specimen. In the NGI apparatus, the lateral earth coefficient is equal to the “at rest” state by virtue of the strain boundary condition. No additional lateral stresses are applied.

To determine the value of the coefficient of lateral earth pressure to be used in the test series, the literature on research of Holocene bay mud was reviewed. The in situ lateral stresses of the Holocene bay mud at HAFB were investigated by self boring pressuremeter and Glotzl load cell, as summarized by Bonaparte and Mitchell (1979). The coefficient of lateral earth pressure at rest, K_0 , measured by the two techniques fell between 0.5 and 0.7 in the normally consolidated zone, with the Glotzl cell giving measurements that were higher than the pressuremeter test.

Lacerda (1976) investigated the value of K_0 of Holocene bay mud from HAFB, using an experimental technique. Consolidating specimens in a triaxial apparatus, he performed a series of ten tests in which the increments of horizontal and vertical stress were applied to the specimen while the ratio of horizontal to vertical stress was kept constant. The size of the stress increments was also varied between tests. The lateral stress ratios were chosen to bracket the estimated in situ value for K_0 , and the stress increments were chosen to investigate the influence of pore pressure gradients within the specimen during

consolidation. The experimental value of K_0 was determined as the lateral stress ratio in which the lateral strains were essentially zero at the end of primary consolidation in any given increment. The lateral strains were represented by the difference between axial strains and volumetric strains in each increment, and the lateral stress ratio for zero strain difference was found by plotting the results for each increment and interpolating between tests. The results of the experimental series suggested that the in situ value of K_0 is approximately 0.58.

For the current series of tests, the coefficient of lateral earth pressure during consolidation, K_c , was targeted to be approximately 0.58 to 0.60. In practice, the final values of K_c ranged from 0.54 to 0.64, although a subset of monotonic tests was performed with K_c ranging from 0.41 to 0.44. The anisotropic state was reached in small increments, with any one increment of vertical load not allowed to exceed one-third of the shear strength of the soil in the previous consolidation step, estimated from normalized shear strength parameters. The increment size was chosen to not only avoid failing the specimen in rapid shear, but also to minimize the development of a significant pore pressure gradient within the specimen during consolidation. Each increment was applied to the specimen for a minimum of one hour. When the final consolidation stage was reached, the specimen was held at this state for approximately 12 hours before subsequent testing was performed. This “rest” period was estimated to be between one and two log cycles of secondary compression.

The final vertical consolidation stress, σ_{vc}' , in the monotonic tests varied from 0.5 to 5.3

times the maximum past consolidation stress of the specimens, including both normally consolidated and overconsolidated tests. All monotonic tests (both without any cyclic loading and after cyclic loading) were controlled through the ATS software. The tests were performed at a constant rate of horizontal displacement equal to approximately 0.125% of the vertical height per minute. The vertical height was kept constant throughout the test, and the drain lines were closed. At the end of the test, the specimen was weighed and put into the oven to determine its water content.

The final vertical consolidation stresses in the cyclic tests were all approximately twice the maximum past consolidation stress of the specimen. Cyclic tests were also controlled through the ATS software. The vertical height was kept constant throughout the test, and the drain lines were closed. The frequency was 1 Hz in most of the tests, and 0.5, 2, and 4 Hz in others. The number of cycles varied from 1 to 50. In some tests, the drainage lines were opened after the cyclic loading to allow pore pressures to dissipate prior to post-cyclic testing. In most of the tests, the post-cyclic test was performed quickly after cyclic loading without pore pressure dissipation.

CHAPTER 6 TEST RESULTS

6.1 Consolidation Tests

The purpose of consolidation testing was to confirm the preconsolidation stress profile developed for Holocene bay mud at Hamilton Air Force Base (HAFB) by Bonaparte and Mitchell (1979). It was necessary to derive the relation between preconsolidation stress and depth so that the OCR of laboratory test samples could be estimated. The data from Bonaparte and Mitchell (1979) was augmented with results of consolidation tests performed for this study and test results obtained by students in laboratory classes (CE 177 and CE 270L) taught at UC Berkeley in 1992-1994. The test parameters for the two incremental load tests and one constant rate of strain test are presented in Table 6.1. The preconsolidation stress profile is presented in Figure 6.1. The profile shows that the bay mud has a desiccated, overconsolidated crust extending to a depth of about five meters and is normally consolidated below. A high variability was expected to occur in the data set created in the classroom environment; nonetheless, both the data from the classroom testing and the tests performed for this study are consistent with the profile proposed by Bonaparte and Mitchell (1979).

Consolidation Test	Sample depth (meters)	e_o	σ_p' (kPa)	C_c
ILC1	7.0	2.41	61	1.3
ILC2	3.3	2.19	78	1.0
CRS1	3.3	2.21	84	1.1

Table 6.1 Consolidation test results obtained in this study.

6.2 Monotonic Simple Shear Tests

Eight monotonic simple shear tests were performed without prior cyclic loading. The purpose of the monotonic testing was to establish a methodology for estimating the static strength of the specimens to be cyclically tested, as the static strength would be used as the baseline for comparison with the cyclic data. The logical method for estimating static strength was to use a normalization parameter such as in the SHANSEP approach; therefore the testing program set out to validate this approach and determine the appropriate SHANSEP parameters. Tests MONO-1, -2, -3, -4, and -8 were performed to establish the normally consolidated undrained strength ratio by testing the specimens at different points on the virgin compression curve. Tests MONO-5, -6, and -7 were performed to establish the overconsolidated undrained strength ratio, and also to evaluate the applicability of SHANSEP loading and unloading consolidation test procedures to a sensitive clay like bay mud. Test parameters for each of these monotonic tests are presented in Table 6.2.

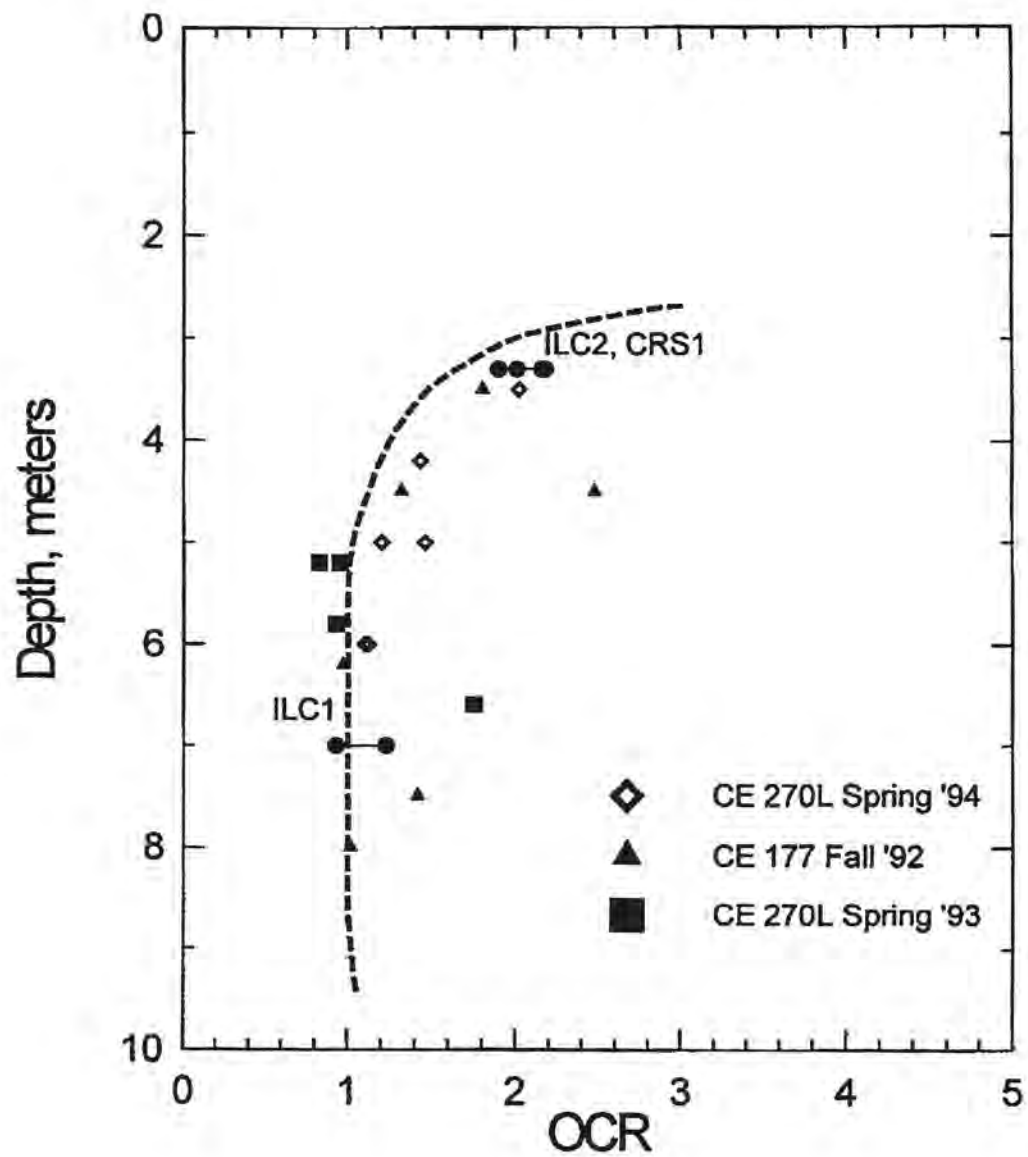


Figure 6.1 Preconsolidation stress profile at Hamilton Air Force Base, California. Dashed line shows interpretation of profile presented in Bonaparte and Mitchell (1979).

Test No.	Depth (meters)	w_i	w_c	τ_s/σ_{vc}'	σ_{vo}' (kPa)	est. OCR in situ	σ_{vc}' (kPa)	σ_{vc}'/σ_p'	σ_{hc}' (kPa)	K_c
MONO-1	5.5	0.916	0.727	0	48	1.05	68	1.3	28	0.41
MONO-2	5.5	0.949	0.762	0	48	1.05	67	1.3	29	0.43
MONO-3	5	0.882	0.724	0	46	1.05	254	5.3	112	0.44
MONO-4	7	0.895	0.626	0	56	1.05	160	2.7	69	0.43
MONO-5	3.1	0.807	0.799	0	36	2	40	0.6	23	0.58
MONO-6	3.2	0.820	0.776	0	37	2	59	0.5	35	0.59
MONO-7	3.2	0.861	0.661	0	37	2	160	2.2	96	0.60
MONO-8	5.2	0.935	0.765	0	47	1.05	119	2.3	71	0.60

Table 6.2 Test parameters for monotonic tests

6.2.1 Normalization Parameters – Normally Consolidated Specimens

Monotonic tests MONO-1, -2 -3, -4, and -8 were performed to determine the undrained strength ratio of bay mud under normally consolidated conditions. In the SHANSEP approach (Ladd and Foott, 1974), the undrained strength ratio, τ_f/σ_{vc}' , of any specimen tested at a consolidation stress sufficiently above the preconsolidation stress ($\sigma_{vc}'/\sigma_p' > 1.5$ -2.0) should be a constant value, with a scatter no greater than 10% of the mean value. The strength of specimens tested at a consolidation ratio, σ_{vc}'/σ_p' , between 1.0 and the “sufficient” value may be influenced by the decrease in void ratio that accompanies reconsolidation in the laboratory; but if the soil is consolidated well onto the virgin compression curve, this effect is erased.

Tests MONO-1, -2 -3, -4, and -8 were performed on specimens that were normally consolidated with a consolidation stress ratio, σ_{vc}'/σ_p' , that varied between 1.3 and 5.3. The results of all monotonic tests are tabulated in Table 6.3. The undrained shear strength shown in Table 6.3 is determined as the horizontal shear stress at 7.5% shear strain, normalized by the vertical consolidation stress. The graphical results of tests MONO-1, -2 -3, -4, and -8 are presented in Figure 6.2.

Test No.	Depth (meters)	w_c	τ_v/σ_{vc}'	σ_{vc}' (kPa)	σ_{vc}'/σ_p'	K_c	γ_f (%)	τ_f (kPa)	u_{max}/σ_{vc}'	τ_f/σ_{vc}'
MONO-1	5.5	0.727	0	68	1.3	0.41	7.5	20	0.06	0.3
MONO-2	5.5	0.762	0	67	1.3	0.43	7.5	19	0.08	0.29
MONO-3	5	0.724	0	254	5.3	0.44	7.5	56	0.1	0.22
MONO-4	7	0.626	0	160	2.7	0.43	7.5	41	0.09	0.26
MONO-5	3.1	0.799	0	40	0.6	0.58	7.5	19	0.16	0.48
MONO-6	3.2	0.776	0	59	0.5	0.59	7.5	24	-0.04	0.41
MONO-7	3.2	0.661	0	160	2.2	0.60	7.5	33	0.28	0.21
MONO-8	5.2	0.765	0	119	2.3	0.60	7.5	30	0.32	0.25

Table 6.3 Tabulated results for monotonic tests

The stress-strain curves in Figure 6.2 show that there is a consistent trend of decreasing undrained strength ratio with consolidation stress. Test MONO-8 deviates from this trend slightly, but the horizontal stress ratio, K_c , of this test was 0.60, versus a stress ratio of between 0.41 and 0.44 for the other tests, and some deviations are expected due to differences in anisotropy, minor testing variations, and natural variability. This trend of decreased undrained strength ratio with increased consolidation stress can also be

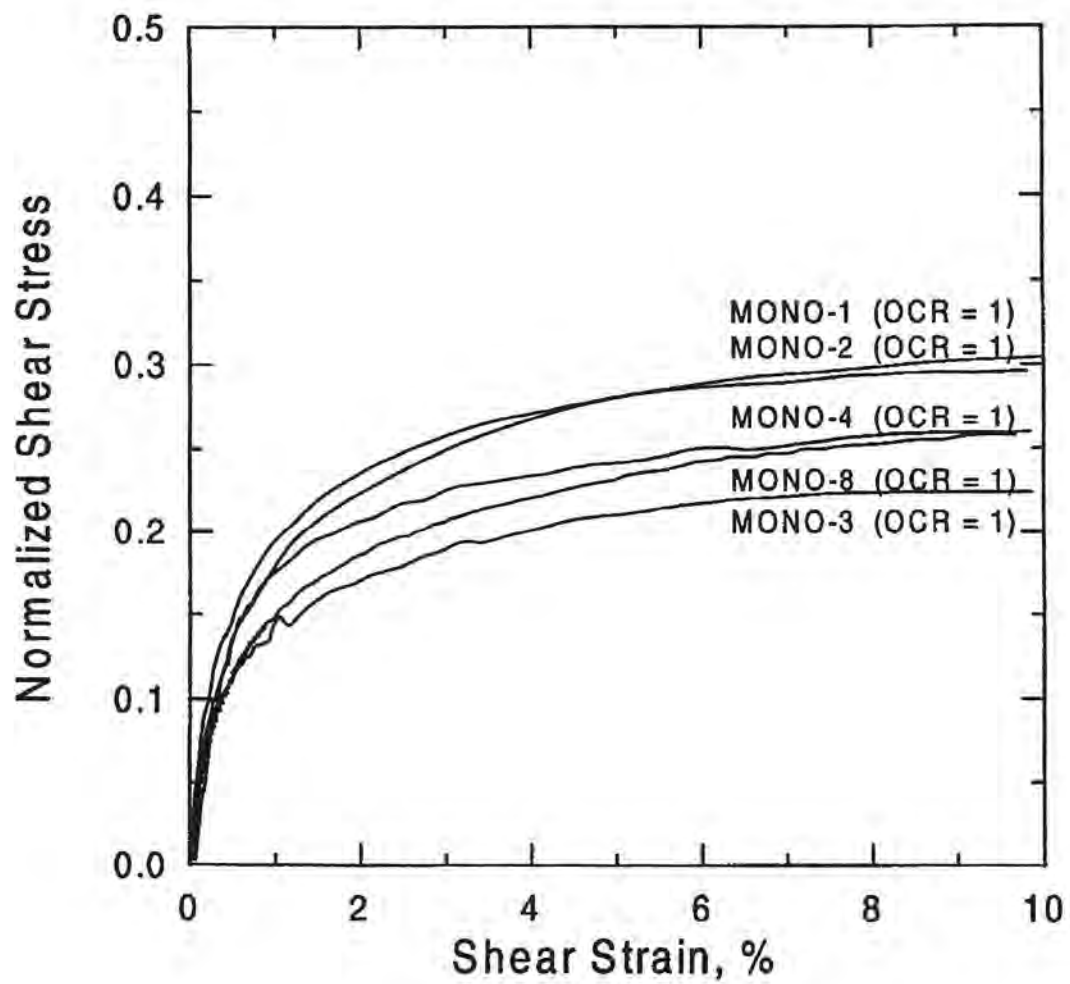


Figure 6.2 Stress-strain curves for monotonic tests MONO-1 through MONO-4 and MONO-8, Holocene bay mud from Marin County, California.

observed in data reported by Denby (1978), who performed a SHANSEP program on Holocene bay mud from HAFB in the triaxial apparatus (see Figure 2.2). Ladd and Foott (1974) suggested that if S_u/σ_{vc}' varies consistently with stress, the normalized soil parameter concept does not apply. Therefore, even though local practitioners have used normalized soil parameters to represent the strength profile of Holocene bay mud for years, this review suggests that the use of SHANSEP for bay mud might not be strictly appropriate. In general, the approach is only valid for uniform clays of low to moderate sensitivity, without cementation. Holocene bay mud from Hamilton Air Force Base is a sensitive clay with sensitivity in the range of approximately 3-8.

The monotonic testing series suggests that the SHANSEP approach should be carefully applied when modeling the strength profile of moderately sensitive clays. For bay mud from Hamilton Air Force Base, the strength of specimens tested at $\sigma_{vc}'/\sigma_p' = 1.3$ are higher, probably due to the decrease in void ratio caused by laboratory reconsolidation to the original in situ stresses. The value of $\sigma_{vc}'/\sigma_p' = 1.3$ is not sufficiently above the preconsolidation stress recommended by Ladd and Foott ($\sigma_{vc}'/\sigma_p' > 1.5-2.0$) for erasing the effects of sample disturbance. The other specimens tested between σ_{vc}'/σ_p' between 2.3 and 5.3 (MONO-3, -4, and -8) have undrained strength ratios within 16% of each other, and show a trend of decreasing strength ratio with consolidation stress. This suggests that the strength may be affected by disturbance to the soil structure, even well onto the virgin compression line.

In light of the variability of the monotonic test results, a set of initial testing conditions was chosen as a target for all of the cyclic tests, so that the baseline static strength used for comparison could be reasonably estimated. A monotonic test was performed under these conditions so that the effects of consolidation stress (magnitude and anisotropy) did not need to be reassessed for each cyclic test. Test MONO-8 was performed with initial test conditions approximately equal to the values chosen for the cyclic testing (target values: $\sigma_{vc}'/\sigma_p' = 2.0$, $K_c = 0.6$). The undrained strength ratio determined from this test was 0.25.

6.2.1 Normalization Parameters – Overconsolidated Specimens

The purpose of tests MONO-5 through MONO-7 was to establish the overconsolidated undrained strength ratio parameter of bay mud, and also to evaluate the applicability of SHANSEP loading and unloading consolidation procedures to a sensitive clay like bay mud. The tests were performed on specimens taken from one tube, which was retrieved from the shallow overconsolidated zone at Hamilton Air Force Base. Three tests were performed on specimens from this tube: one test at the in situ, overconsolidated stress, one that was mechanically overconsolidated to approximately the same OCR using the SHANSEP approach, and one that was normally consolidated onto the virgin compression curve for comparison.

The overconsolidation ratio estimated for this sample in the field was 2.0. Test specimen MONO-5 was reconsolidated to slightly over the in situ stresses ($OCR = 1.7$)

prior to shearing. Test specimen MONO-6 was first consolidated onto the virgin compression line, and then unloaded back to an overconsolidation ratio of 2.0, following the SHANSEP procedure. Specimen MONO-7 was normally consolidated in the laboratory to a consolidation stress ratio of $\sigma_{vc}'/\sigma_p' = 2.2$ prior to shearing. The test results are presented in Figure 6.3.

Figure 6.3 demonstrates that the strength of an overconsolidated specimen depends on the method of achieving the overconsolidated state (by testing at the in situ stresses, or by mechanically unloading the specimen by the SHANSEP method). Test MONO-5, which was not consolidated to the virgin compression line, had a higher strength at a lower OCR (= 1.7) than MONO-6, which was loaded onto the virgin compression line and then unloaded to OCR = 2.0. As in the monotonic testing, the higher strength at low consolidation stresses is probably due to the decrease in void ratio caused by laboratory reconsolidation below the virgin compression line. The SHANSEP methodology is designed to erase the effects of disturbance that is inherent in testing at in situ stresses; however, the monotonic test series provides evidence that the stress achieved on the virgin compression line relative to the original preconsolidation stress affects the undrained strength ratio. In the mechanical overconsolidation process required for SHANSEP testing of overconsolidated specimens, the specimen is first consolidated onto the virgin compression line to a somewhat arbitrary ratio of consolidation stress to original preconsolidation stress. This ratio is usually between 1.5 and 4.0, but within this range (except for sensitive clays) the same results are expected. For MONO-5, the preconsolidation stress at in situ conditions is $\sigma_p' = 67$ kPa; for

MONO-6, which had the same in situ preconsolidation stress, the specimen was first consolidated to a new $\sigma_p' = 118$ kPa. The ratio for test MONO-6 prior to unloading was $\sigma_{vc}'/\sigma_p' = 1.8$, within the range of approximately 1.5 to 4.0 recommended for SHANSEP. However, even though the same OCR could be achieved by consolidating to higher or lower stresses initially, according to the results of the monotonic testing, different strengths would result. The implications are that for sensitive clays, if a series of tests is performed at different OCR's, then the consolidation stress prior to unloading should be at a consistent ratio of stress to in situ preconsolidation stress prior to mechanical unloading. This should ensure a consistent strength profile with OCR; however, the "correct" strength for an overconsolidated, sensitive clay is unknown.

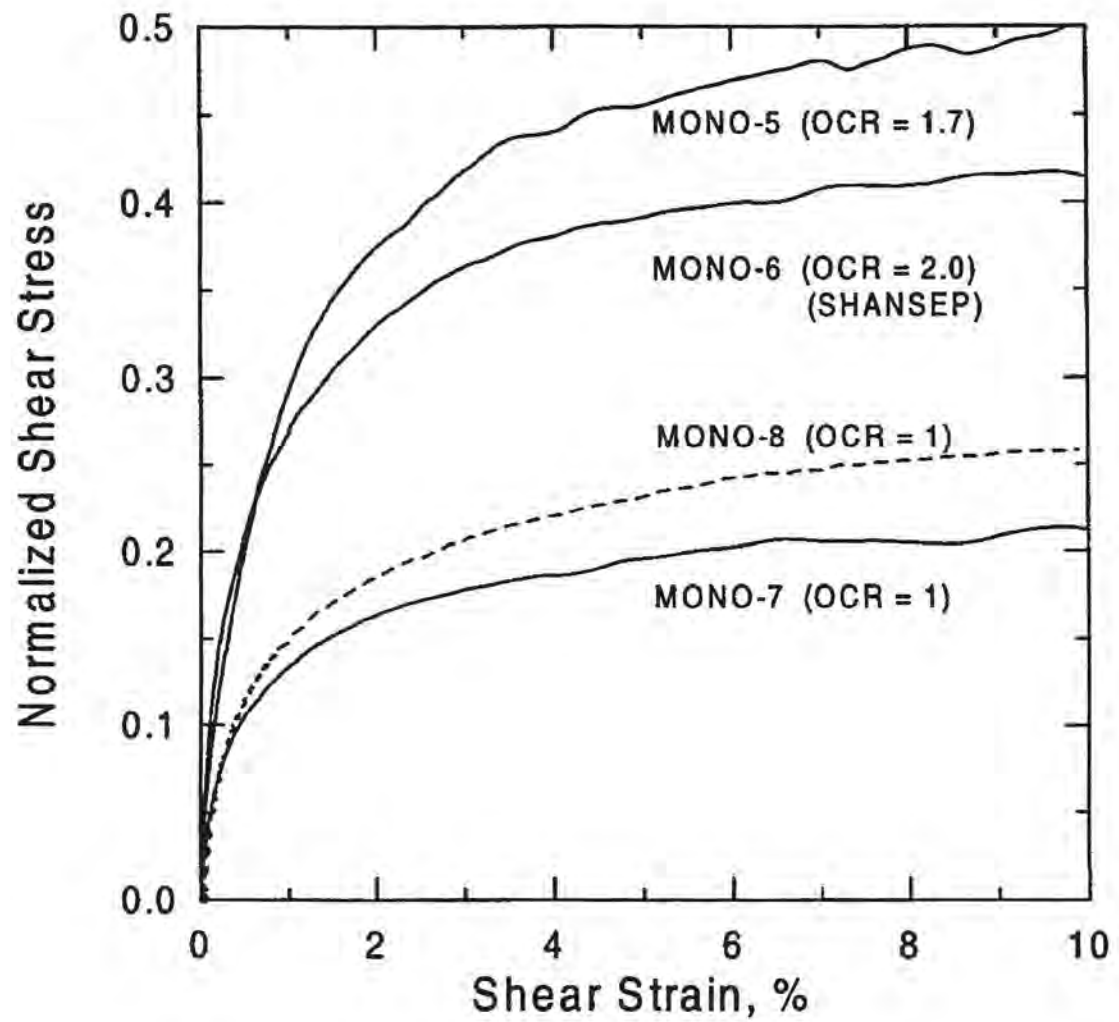


Figure 6.3 Stress-strain curves for monotonic tests MONO-5 through MONO-7, Holocene bay mud from Marin County, California.

The relationship between undrained strength in the normally consolidated and overconsolidated states is derived from Equation 2.3.

$$(S_u/\sigma_{vc}')_{OC} = (S_u/\sigma_{vc}')_{NC}(OCR)^m \quad 6.1$$

Ladd et al. (1977) suggested that the empirically derived value of m should be approximately 0.8. The value of m determined for samples of bay mud from the MUNI Metro Turnaround Project in San Francisco (Dames and Moore, 1989) was 0.85. The specimens were overconsolidated in the laboratory to OCR's between 1.5 and 4 prior to shearing in a monotonic simple shear test.

For tests MONO-5 and MONO-6 overconsolidated specimens, it was assumed that the normally consolidated undrained strength ratio was represented by the strength of specimen MONO-7, $\tau_f/\sigma_{vc}' = 0.21$. The value of m in E 6.1 is estimated to be between 1.01 and 2.6 for specimen MONO-5 tested near the in situ stress, with a most likely value of 1.2. The parameter m must be presented as a range for this specimen because the preconsolidation stress (and thus the OCR) is difficult to estimate from consolidation testing; the preconsolidation stress profile is estimated as a range of possible values. The value of m for the SHANSEP-consolidated specimen MONO-6, which can be more precisely known due to the SHANSEP overconsolidation procedure, is 0.97.

6.3 Cyclic Simple Shear Tests

Sixteen cyclic simple shear tests were performed to characterize the dynamic response of bay mud. The test parameters for each cyclic test are presented in Table 6.4. Each specimen was consolidated in the laboratory to the virgin compression region, with the consolidation stress ratio σ_{vc}'/σ_p' between 1.2 and 2.5, with most between 2.0 and 2.3. The earth pressure coefficient in the tests ranged from 0.45 to 0.64, with most between 0.59 and 0.61. Single amplitude cyclic shear strain amplitudes varied from 0.12% to 5.76% of the height. The frequency varied between 0.5 Hz and 4 Hz, and the number of cycles varied from 5 to 50. The quantities that were measured during cyclic tests were chamber pressure, effective stress, volume change, horizontal and vertical displacement, vertical load, and horizontal load.

6.3.1 Threshold Levels

The threshold levels identified by Vucetic (1994), γ_{ll} and γ_{lv} , are low-strain parameters. The tests performed for this study concentrated on larger strain behavior, in the range of 0.1% to 5% single amplitude shear strain. Some cyclic tests were performed below 0.1%, but the results of these tests are less accurate, because of the limitations of the testing apparatus and measuring devices.

The linear cyclic threshold shear strain (γ_{ll}) divides essentially linear behavior from the nonlinear. The linear cyclic threshold shear strain of Holocene bay mud from Hamilton

Test	Depth (meters)	w_i	w_c	τ_o/σ_{vc}'	σ_{vo}' (kPa)	est. OCR	σ_{vc}' (kPa)	σ_{vc}'/σ_p'	σ_{hc}' (kPa)	K_c
CY-1	7.2	0.916	0.727	0	57	1.05	126	2.1	71	0.56
CY-2	7.3	0.942	0.780	0	57	1.05	121	2.0	71	0.59
CY-3	7.3	0.949	0.762	0	57	1.05	116	1.9	73	0.63
CY-4	4.3	0.882	0.724	0	42	1.3	121	2.2	65	0.54
CY-5	4.3	0.895	0.626	0	42	1.3	135	2.5	86	0.64
CY-6	5.8	0.929	0.741	0	50	1.05	117	2.2	71	0.61
CY-7	4.4	0.920	0.768	0	43	1.15	118	2.4	72	0.61
CY-8	4.5	0.884	0.726	0	43	1.15	115	2.3	71	0.62
CY-9	4.5	0.835	0.706	0	43	1.15	114	2.3	67	0.59
CY-10	4.1	0.896	0.784	0	41	1.2	106	2.2	65	0.61
CY-11	7.0	0.905	0.862	0	56	1.05	71	1.2	32	0.45
CY-12	7.0	0.908	0.833	0	56	1.05	72	1.2	33	0.46
CY-13	5.3	0.885	0.713	0	47	1.05	112	2.3	66	0.59
CY-14	5.3	0.866	0.713	0	47	1.05	110	2.2	66	0.60
CY-15	4.3	0.889	0.737	0.09	42	1.3	134	2.5	79	0.59
CY-16	4.3	0.880	0.760	0	42	1.3	109	2.0	65	0.60

Table 6.4 Test parameters for cyclic tests

Air Force Base tested by Isenhower and Stokoe (1981) was approximately 0.001 percent. This strain is much lower than those applied in the present study.

The volumetric cyclic threshold shear strain (γ_{tv}) divides nonlinear but nondegrading cyclic behavior from degrading behavior in which the shear modulus is permanently decreased by cyclic loading. Above the volumetric cyclic threshold shear strain, significant permanent volume change (in drained loading) or pore pressure generation (in undrained loading) occurs. The volumetric cyclic threshold shear strain in specimens of Holocene bay mud from Hamilton Air Force Base tested by Isenhower and Stokoe (1981) was greater than 0.02 percent.

Figures 6.4 and 6.5 show the development of pore pressures in the cyclic tests performed for this study. The results suggest that below approximately 0.02% to 0.04% shear strain, a limited amount of pore pressure development develops, but the pressure does not increase even beyond 10 or 15 cycles of loading. Vucetic (1994) suggested that the volumetric cyclic threshold strain corresponds to a reduction in shear modulus of about 35 percent for all soils. Figure 6.6, showing the modulus degradation curve of the specimens from Hamilton Air Force Base, confirms that a volumetric cyclic threshold strain of 0.02% to 0.04% shear strain corresponds to approximately a 35 percent reduction in shear modulus.

Other threshold limits have been suggested, such as the Critical Level of Repeated Loading presented by Houston and Hermann (1980). Tests to determine the CLRL of

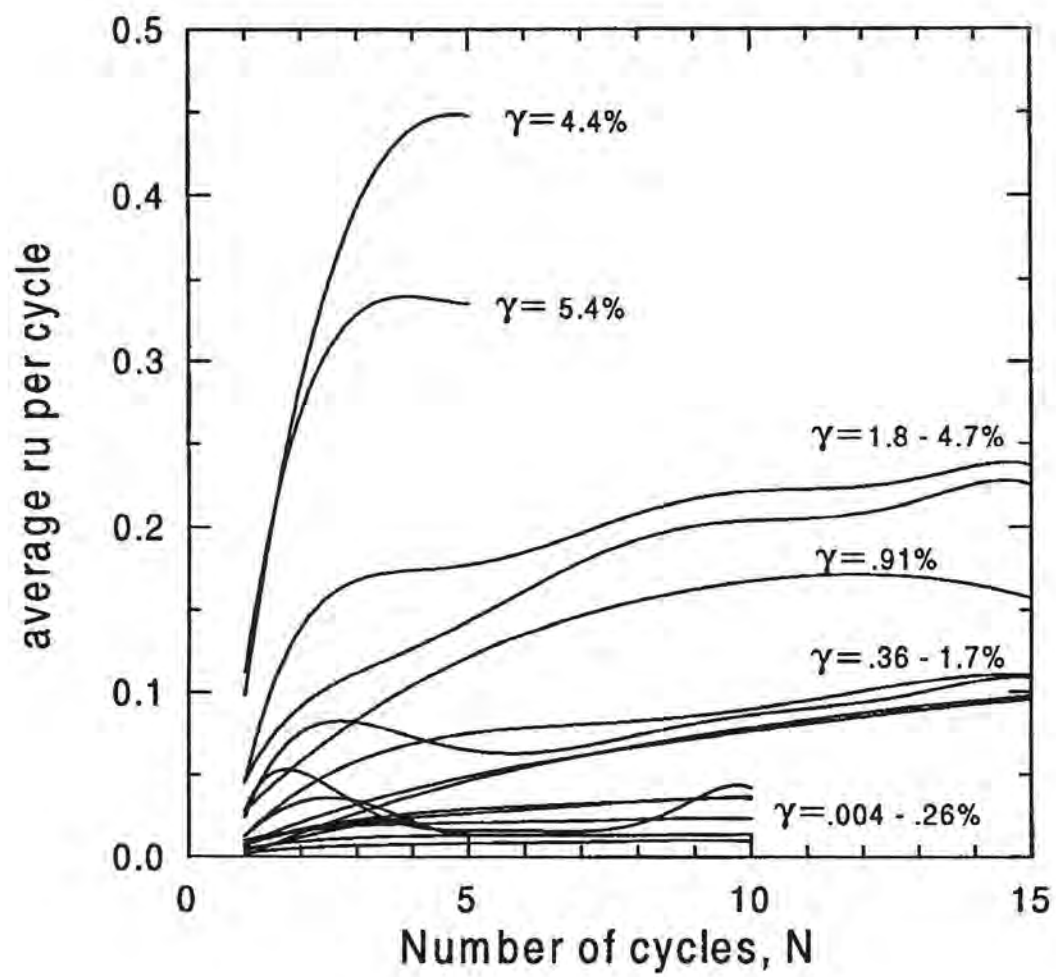


Figure 6.4 Development of pore pressures as a function of cycle number during cyclic loading; $r_u = \Delta u_{exc} / \sigma'_{vc}$.

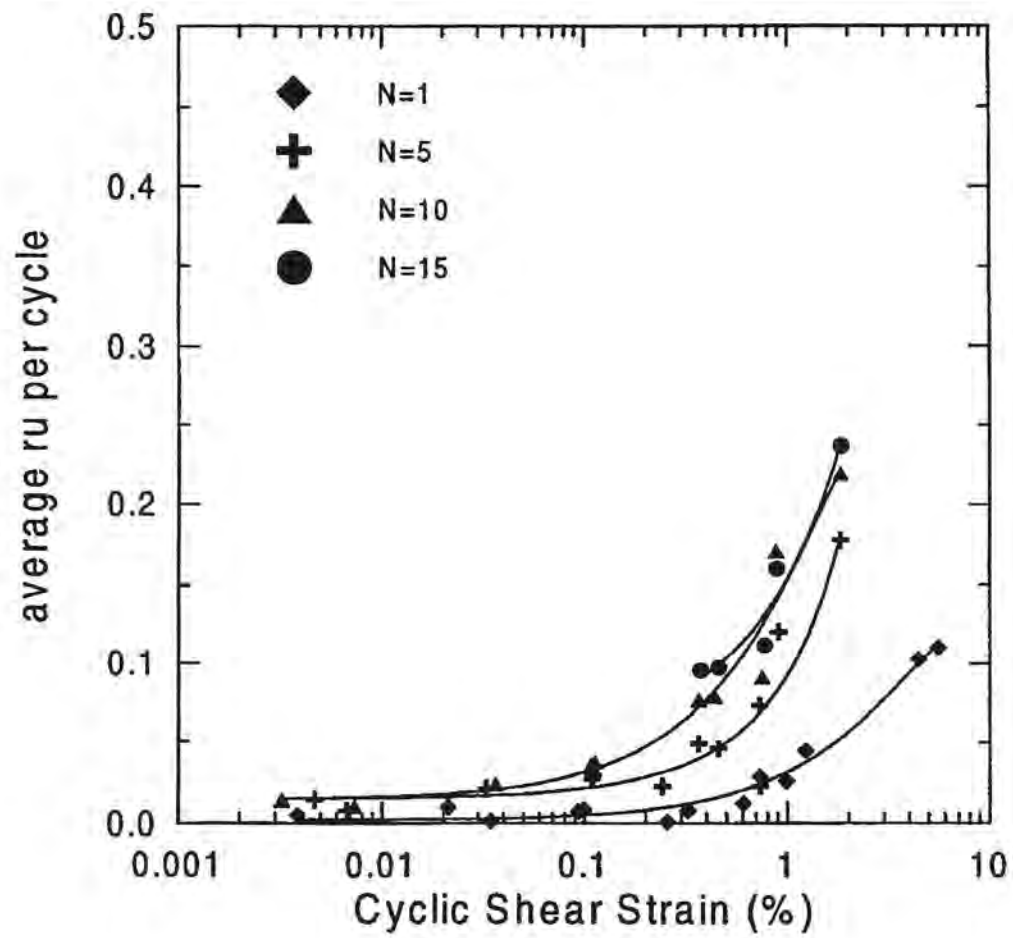


Figure 6.5 Development of pore pressures as a function of strain during cyclic loading;
 $r_u = \Delta u_{exc} / \sigma'_{vc}$.

Holocene bay mud from Hamilton Air Force Base were not performed as part of the current study.

6.3.2 Modulus Degradation

The shear modulus was calculated for each cycle of loading, and the results for the first cycle are presented in Figure 6.6. The small-strain shear modulus, $G_{\max}$, was calculated as a function of vertical consolidation stress from empirical relations developed by Dickenson (1994). Dickenson developed the following correlation between depth and V_s for normally consolidated bay mud, based on local shear wave velocity data.

$$V_s \text{ (fps)} = 4.2z + 180 \quad 6.2$$

The shear wave velocity was estimated by back-calculating a “depth” corresponding to the consolidation stress of the laboratory specimen. The small strain shear modulus was then calculated as

$$G_{\max} = V_s^2 \rho \quad 6.3$$

Figure 6.7 presents the modulus degradation curves with data superimposed from two previous bay mud studies. The data from the present study appears to be consistent with the shear modulus values determined by resonant column by Stokoe and Lodde (1978) on samples from Hamilton Air Force Base ($PI = 45$). The data presented by Isenhower

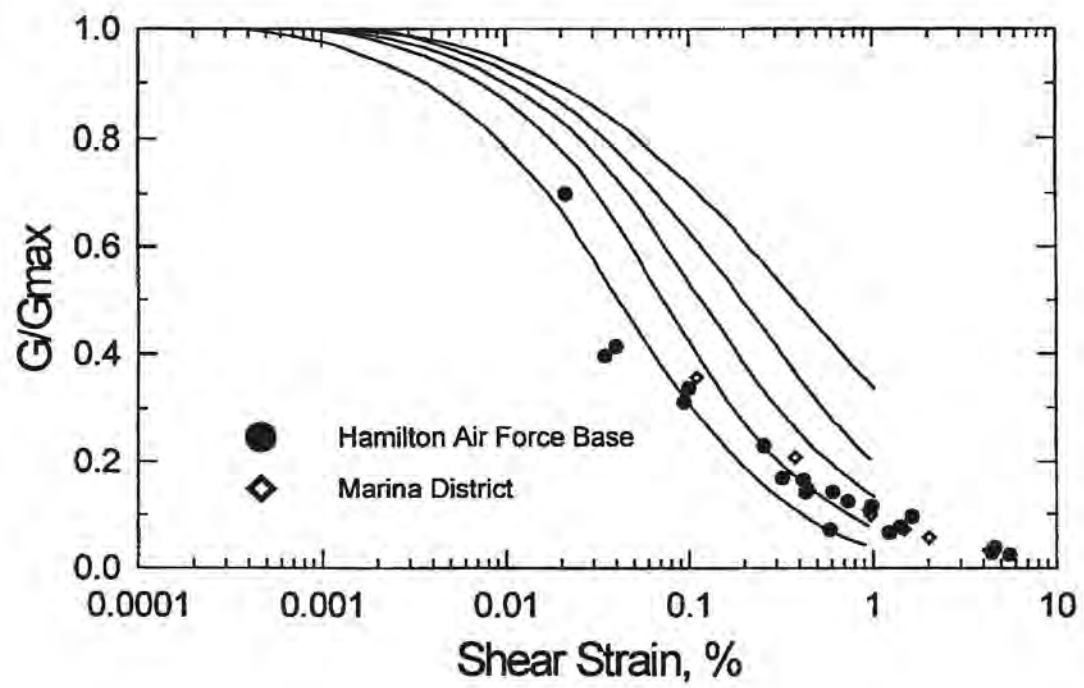


Figure 6.6 Modulus reduction curve for Holocene bay mud, HAFB and Marina District, first cycle of loading.

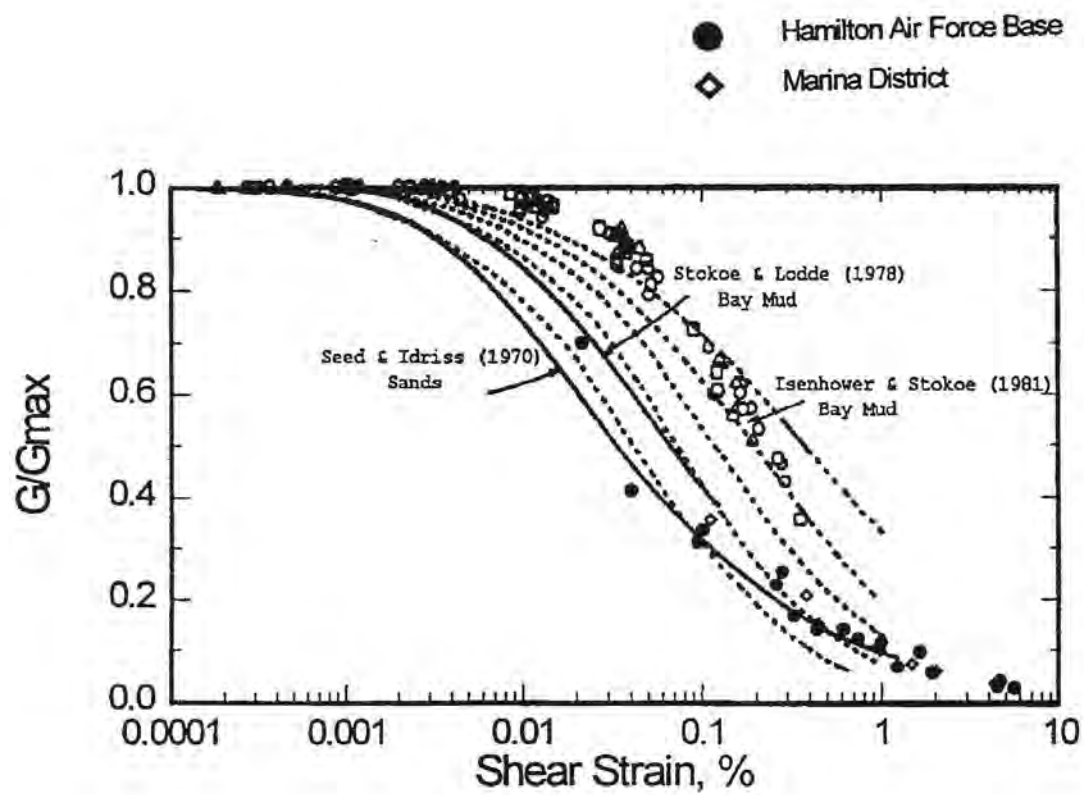


Figure 6.7 Modulus reduction curve for Holocene bay mud, different sites, first cycle of loading.

and Stokoe (1981) was obtained by a resonant column/torsional shear apparatus on specimens from near Dumbarton Bridge ($PI = 33$). The data from this locale shows linearity out to high shear strain amplitudes, indicating a difference either in soil characteristics from this location or else an effect of stress path or test type on the measurement of shear modulus. The agreement between the tests on samples from Hamilton Air Force Base suggests that soil characteristics may be a stronger factor than stress path or test type, as the Stokoe and Lodde data was obtained from a resonant column device and the data from the present study is from cyclic simple shear testing. It should also be noted that the modulus degradation of specimens from Hamilton Air Force Base is more severe than is suggested by the empirical correlation with PI developed by Sun et al. (1988), as the data lie nearer to the contours for $PI = 10-20$ than between the contours for $PI = 20-40$ and $PI = 40-80$, as would be expected (PI at Hamilton Air Force Base = 45).

Figure 6.8 shows the modulus degradation as a function of the number of cycles of cyclic loading. Data points for the first cycle shear modulus from both the Hamilton Air Force Base specimens and the Marina District specimens are compared with the shear modulus after 30 or greater numbers of cycles. The effect of cyclic degradation on the shear modulus is within the range of the scatter of the data for each cycle.

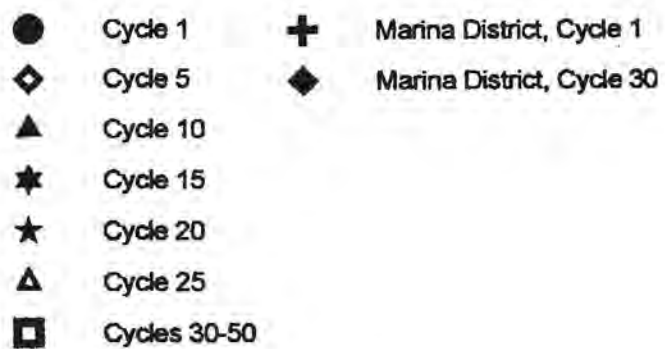
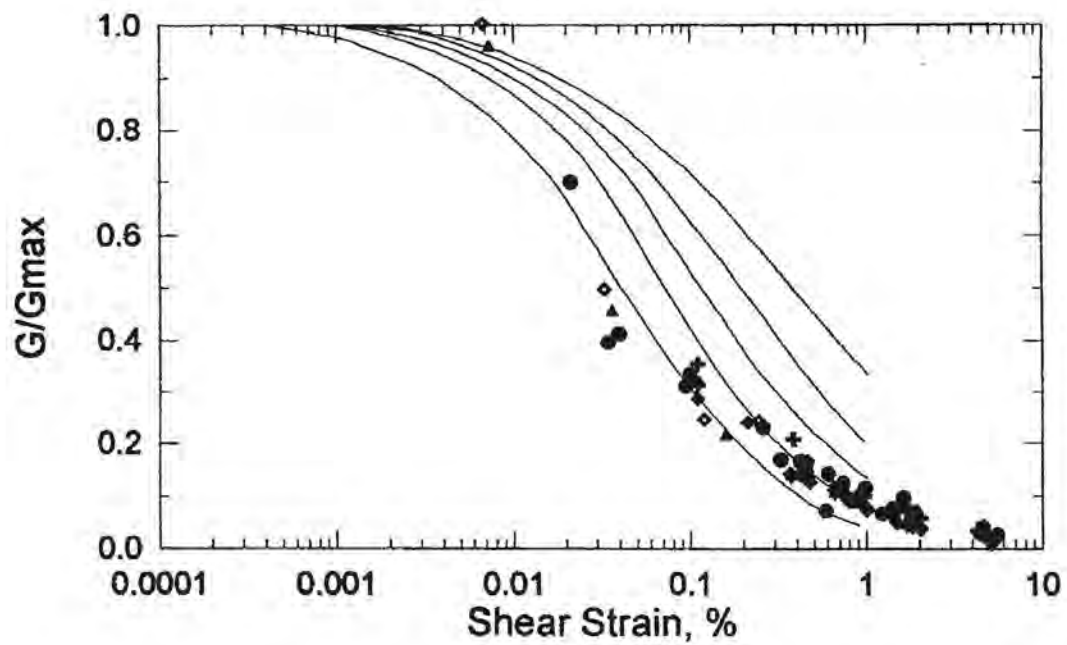


Figure 6.8 Modulus reduction curve for specimens from Hamilton Air Force Base, effect of cycle number

6.3.3 Damping Ratio

The damping ratio for the first cycle of loading are presented in Figure 6.9. The limits shown are those suggested by Seed and Idriss (1970). While there is scatter to the data, practically all of the points are within the limits expected for cohesive soils.

6.3.4 Mobilized Shear Stress During Cyclic Loading

Figure 6.10 shows the normalized shear stress, τ/σ'_{vc} , at the “tips” of the first cycle of loading (labeled “N = 1” on the figure) for each of the cyclic tests on specimens from HAFB. The curve fit to these data points follows a hyperbolic form (Duncan and Chang, 1970). The dashed line superimposed on the figure is the average stress-strain curve from the two monotonic simple shear tests that best represent the monotonic strength, based on the initial conditions (σ'_{vc}/σ'_p , K_c) of the cyclic tests. The mobilized strength in the first cycle of the cyclic tests is up to 40-50% higher than in the monotonic tests.

To compare the monotonic data with the cyclic loading data, the strain rate in each test must be estimated. For the monotonic tests, the shear strain rate approached a constant rate of 0.125% per minute, or 7.5% per hour. For the cyclic tests, it is considered that the average strain rate is $f\gamma/4$. For all tests in which the strain amplitude exceeded 0.1%, the shear strain rate thus varied from a low of .0275% to a high of 1.44% per second, or 99% to 5184% per hour. This rate is between one and three orders of magnitude greater

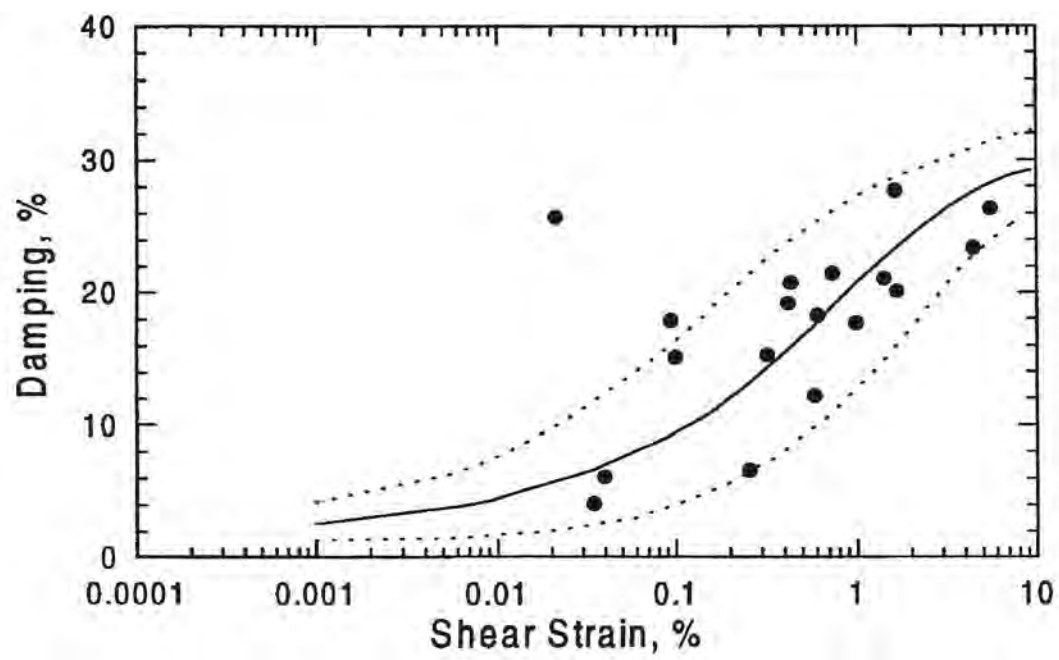


Figure 6.9 Damping ratio for specimens from Hamilton Air Force Base, first cycle of loading.

Curve fit by hyperbolic model:

$$a = G_{\max}, b = \tau_{\max}$$

$$\frac{\tau_e}{\sigma'_{vc}} = \frac{\gamma_e}{\frac{1}{a} + \frac{\gamma_e}{b}}$$

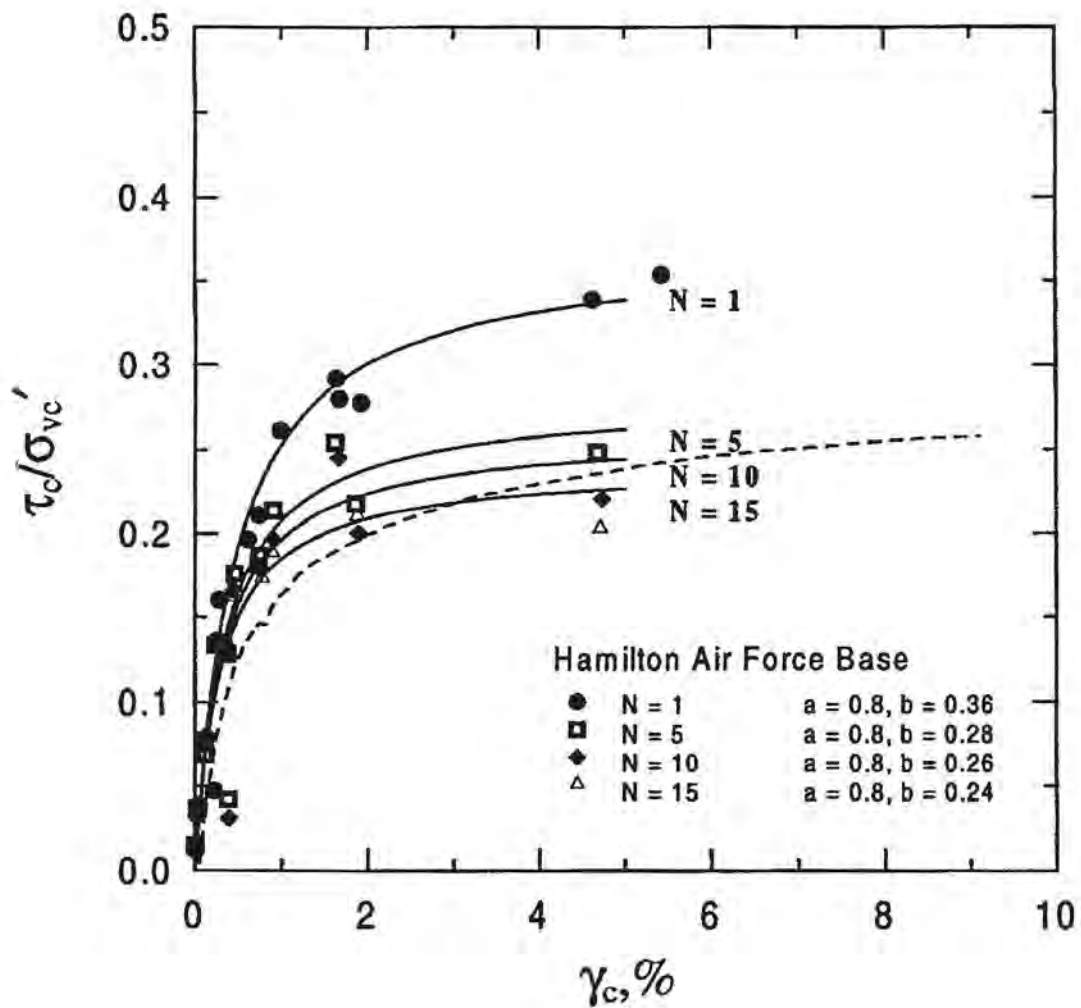


Figure 6.10 Cyclic stress-strain (backbone) curves for Holocene bay mud from Hamilton Air Force Base.

than in the monotonic tests.

The increase in mobilized shear strength in the first cycle data shown in Figure 6.10 represents approximately 13-16% per order of magnitude of increase in strain rate, compared with the results presented by Lacerda (1976) which showed an increase of approximately 9% per order of magnitude on specimens from Hamilton Air Force Base tested in ICU triaxial compression tests and UU tests. The increase of 13% is within the range of 10% to 15% predicted for all clays by Kulhawy and Mayne (1990) and Ishihara (1981), respectively.

The increase in mobilized strength due to rate increases during fast loading is offset by strength degradation due to cyclic loading. Figure 6.10 shows the normalized shear stress, τ/σ_{vc}' , at the “tips” of the first, fifth, tenth, and fifteenth cycle of loading for each of the cyclic tests. The mobilized strength in the cyclic tests is up to 40-50% higher than the monotonic value in the first cycle, but on progressive numbers of cycles the mobilized strength decreases. By the fifteenth cycle, the mobilized strength is very close to the monotonic value.

In comparison with the HAFB data, Figure 6.11 shows the normalized shear stress at the ends of cycles 1, 5, 10, 20, and 30 for cyclic simple shear loading of specimens from the Marina District in San Francisco (Sitar and Salgado, 1994). While the data conform to a hyperbolic shape in the lower strain ranges, strain softening is observed at shear strains beyond one or two percent after the first cycle of loading (the data beyond 1%

Curve fit by hyperbolic model:

$$a = G_{\max}, b = \tau_{\max}$$

$$\frac{\tau_c}{\sigma'_{vc}} = \frac{\gamma_c}{\frac{1}{a} + \frac{\gamma_c}{b}}$$

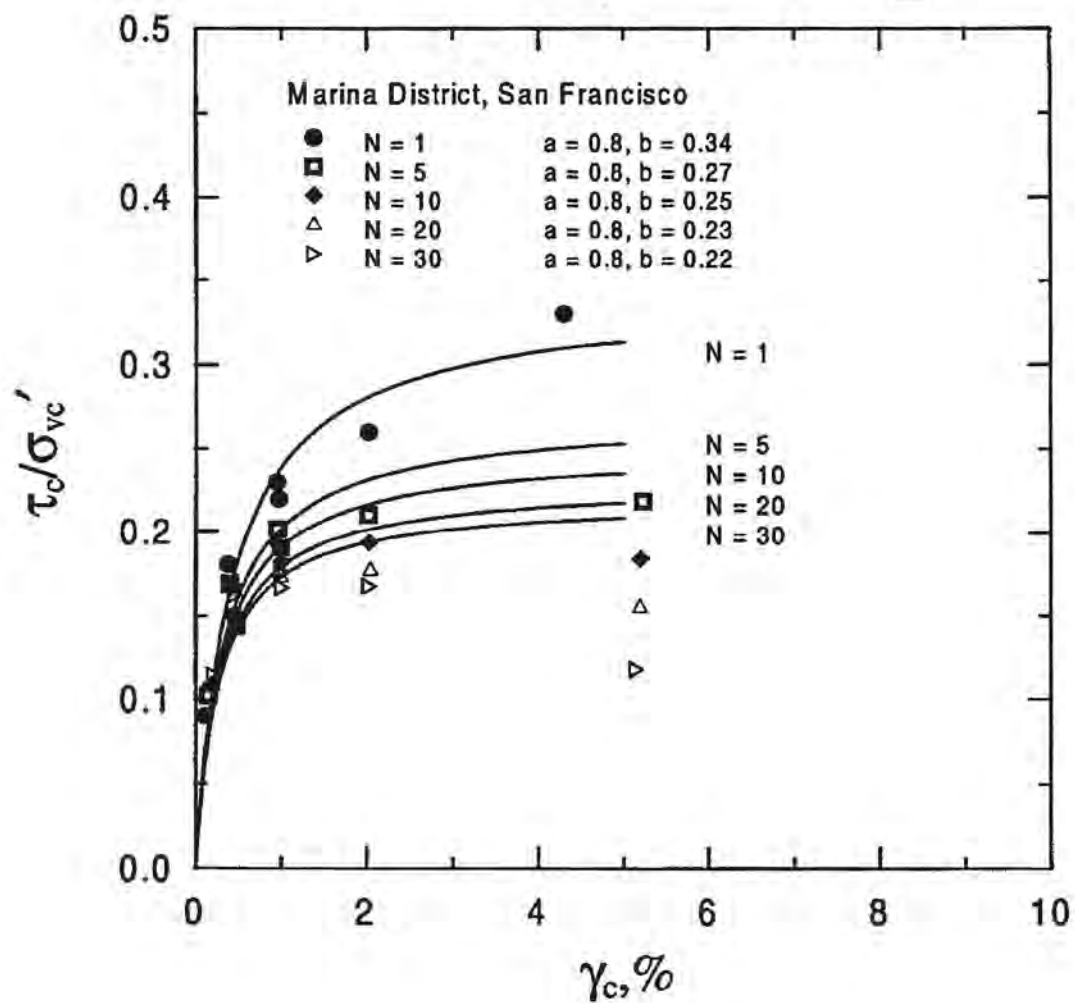


Figure 6.11 Cyclic stress-strain (backbone) curves for Holocene bay mud from the Marina District, San Francisco; normally consolidated specimens.

shear strain have not been used to fit the curves). Upon loading the clay for 30 cycles at five percent cyclic shear strain, the mobilized strength is reduced by approximately 40 percent of the predicted maximum dynamic shear stress. This represents only 50 percent of the predicted monotonic shear strength (the undrained strength ratio of normally consolidated specimens from this test series was $\tau_f/\sigma_{vc}' = 0.24$). While a loading condition this severe is not expected in a typical large earthquake, the strength degradation at smaller strains and numbers of cycles is still significant; the strength is significantly below the monotonic value at five or more cycles of two percent cyclic shear strain. As hypothesized by Sitar and Salgado, the degree of strength loss of these specimens may be related to an apparently higher silt content (67% silt at the Marina District vs. an estimated 40% silt at HAFB) and/or lower plasticity index (PI = 25 at the Marina District vs. PI = 45 at HAFB). In any event, a dramatic difference in cyclic response is observed between the Holocene bay mud from these two locations.

The normalized shear stress for two tests on overconsolidated specimens from the Marina District ($\gamma_c = 0.45$ and $\gamma_c = 1.47$) was converted to an equivalent normally consolidated value through equation 6.1 using an empirically determined value of $m = 0.7$ for these specimens, and the results are shown in Figure 6.12. A slightly lower mobilized cyclic shear strength is suggested as compared with the normally consolidated specimens. Vucetic and Dobry (1986) determined that the relationship between dynamic backbone curves at different overconsolidation ratios is not the same as given in equation 6.1, since the dynamic shear stress is a function of cyclic strain as well as

Curve fit by hyperbolic model:

$$a = G_{\max}, b = \tau_{\max}$$

$$\frac{\tau_c}{\sigma'_{vc}} = \frac{\gamma_c}{\frac{1}{a} + \frac{\gamma_c}{b}}$$

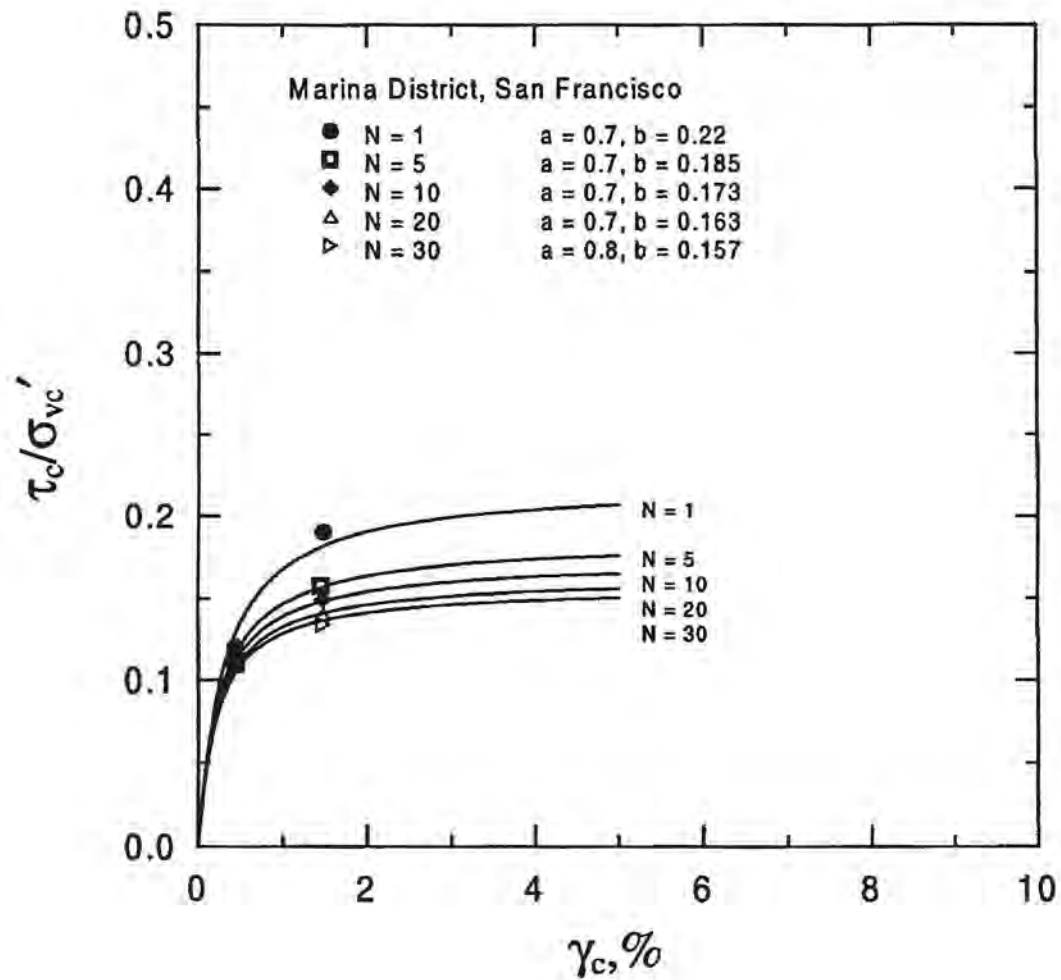


Figure 6.12 Cyclic stress-strain (backbone) curves for Holocene bay mud from the Marina District, San Francisco; overconsolidated specimens.

OCR. Thus, the apparent increased degradation in the overconsolidated specimens at relatively low strains could be an artifact of the conversion to an equivalent normally consolidated value. However, several other investigators (Andersen, 1980; Hyde and Ward, 1985) have found that overconsolidated soils have a lower cyclic resistance at equivalent cyclic stress ratios. A common suggested explanation is that in overconsolidated soils, fatigue and deterioration of the clay structure play a bigger role in inducing failure than does the reduction in effective stress.

6.4 Post-Cyclic Simple Shear Testing

Ten post-cyclic monotonic simple shear tests were performed after cyclic testing at shear strains ranging from 0.1 to 1.9 percent. The post-cyclic tests were performed under strain control at the approximate rate of 0.125%/min of the height of the specimen. Seven of the ten post-cyclic tests were performed quickly after cyclic loading, without pore pressure dissipation. A summary of the tests performed is presented in Table 6.5. Figure 6.13 shows typical test results. Results for each individual test are presented in Appendix B.

6.4.1 Comparison of Pre- and Post-Cyclic Monotonic Tests

The normalized stress-strain curves from ten post-cyclic tests are shown in Figure 6.14, including three tests in which pore pressures were allowed to dissipate between cyclic

Test	Depth (m)	σ_{vc}'/σ_p'	K_c	Cyclic Tests				Post-cyclic Tests		
				γ_{cyc}	N	f (Hz)	u_{max}/σ_{hc}'	u_f/σ_{hc}'	τ_f (kPa)	τ_f/σ_{vc}'
M-1	5.2	2.3	0.60	SLOW MONOTONIC TEST				0.32	30	0.25
M-2	7.0	2.7	0.43	SLOW MONOTONIC TEST				0.09	41	0.26
CY-1	7.2	2.1	0.56	0.75	10	1	0.1	0.24	29	0.23
CY-2	7.3	2.0	0.59	0.12	10	1	0.03	0.24	32	0.27
CY-3	7.3	1.9	0.63	0.73	15	1	0.19	0.18	33	0.28
CY-4	4.3	2.2	0.54	0.74	5	1	0.12	0.12	32	0.27
CY-5	4.3	2.5	0.64	0.36	15	1	0.16	0.01	35	0.26
CY-6	5.8	2.2	0.61	0.91	15	1	0.31	0.26	31	0.26
CY-7	4.4	2.4	0.61	0.44	15	2	0.17	0.27	30	0.25
CY-8	4.5	2.3	0.62	1.86	15	0.5	0.42	0.34	28	0.24
CY-9	4.5	2.3	0.59	0.21	15	4	0.05	0.24	29	0.25
CY-10	4.1	2.0	0.61	1.43	15	0.1	0.50	0.38	26	0.25

σ_{vc}' vertical effective consolidation stress
 σ_p' maximum past consolidation pressure
 K_c ratio of horizontal to vertical stress
 γ_{cyc} single amplitude cyclic shear strain
N number of cycles
f frequency of loading

u_{max} average excess pore pressure
in last cycle of loading
 σ_{hc}' horizontal consolidation stress
 u_f excess pore pressure at failure
 τ_f shear stress on horizontal plane
at failure

Table 6.5 Summary of test parameters for post-cyclic testing.

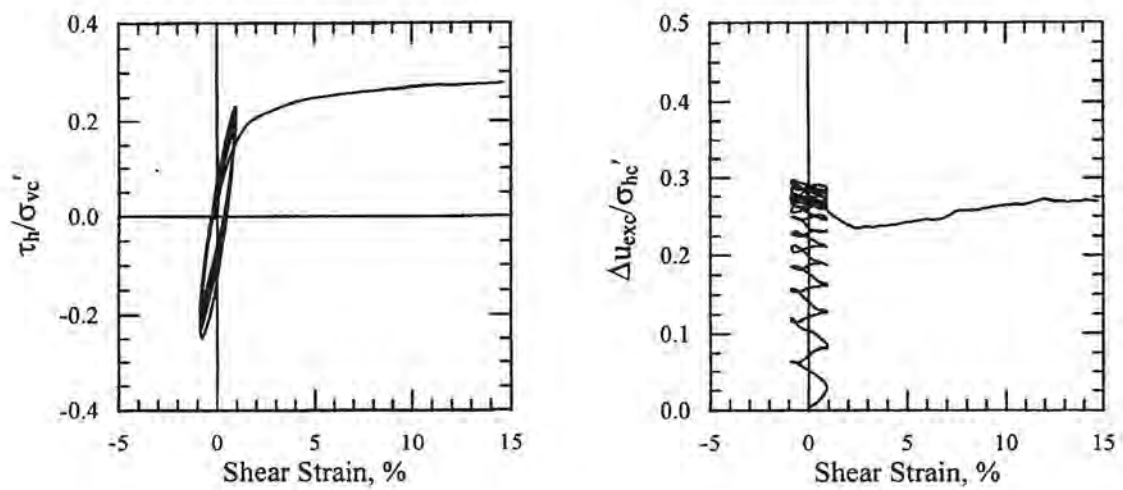


Figure 6.13 Typical test results of strain-controlled cyclic and post-cyclic simple shear testing of Holocene bay mud (for this test, $\gamma_c = 0.91\%$, $N = 15$, $f = 1$ Hz).

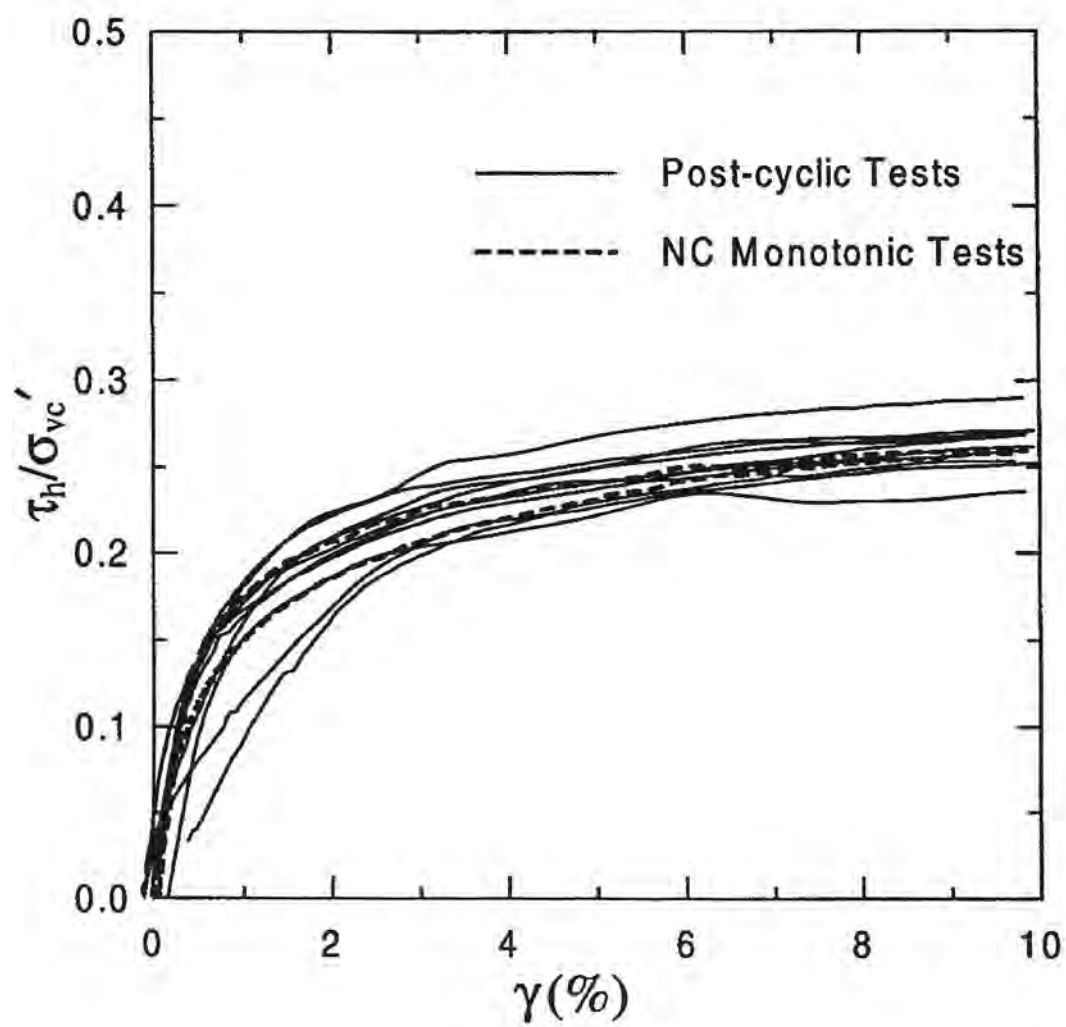


Figure 6.14 Normalized stress-strain curves for post-cyclic tests on Holocene bay mud from Hamilton Air Force Base.

loading and subsequent monotonic shear. The shear stress on the horizontal plane has been normalized by the vertical effective stress at the beginning of cyclic loading. The dotted lines are stress-strain curves from monotonic tests, and represent the pre-shear condition.

Most of the stress-strain curves have a similar shape, with 2/3rds of the maximum shear stress developed before the strain reaches 1%. The exceptions to this are tests CY-8 and CY-10, which were subjected to cyclic strains in excess of 1% and developed excess pore pressures due to cyclic loading greater than 40% of the horizontal effective consolidation stress. The post-cyclic monotonic shear modulus is significantly reduced in these tests, a result that has been noted by others (Thiers and Seed, 1968; Matsui et al., 1992). The normalized shear stress at 7.5% shear strain has been chosen to represent the failure condition, as the increase in shear stress tends to level off beyond this point. Henceforth, the “post-cyclic strength” will refer to the normalized shear stress at 7.5% shear strain. No strain-softening behavior was observed in any of the simple shear tests. The strength in the pre-shear, monotonic tests shown in Figure 6.12 is approximately 0.25, with post-cyclic strengths ranging from 0.23 to 0.27, or within $\pm 10\%$ of the pre-shear value.

In Figure 6.15, cyclic shear strain is plotted against post-cyclic strength for all tests in which pore pressures were not allowed to dissipate. The number of cycles is about the same in all tests, either 10 or 15. There is a small decrease in post-cyclic strength as the cyclic strain increases from 0.12% to 1.86%.

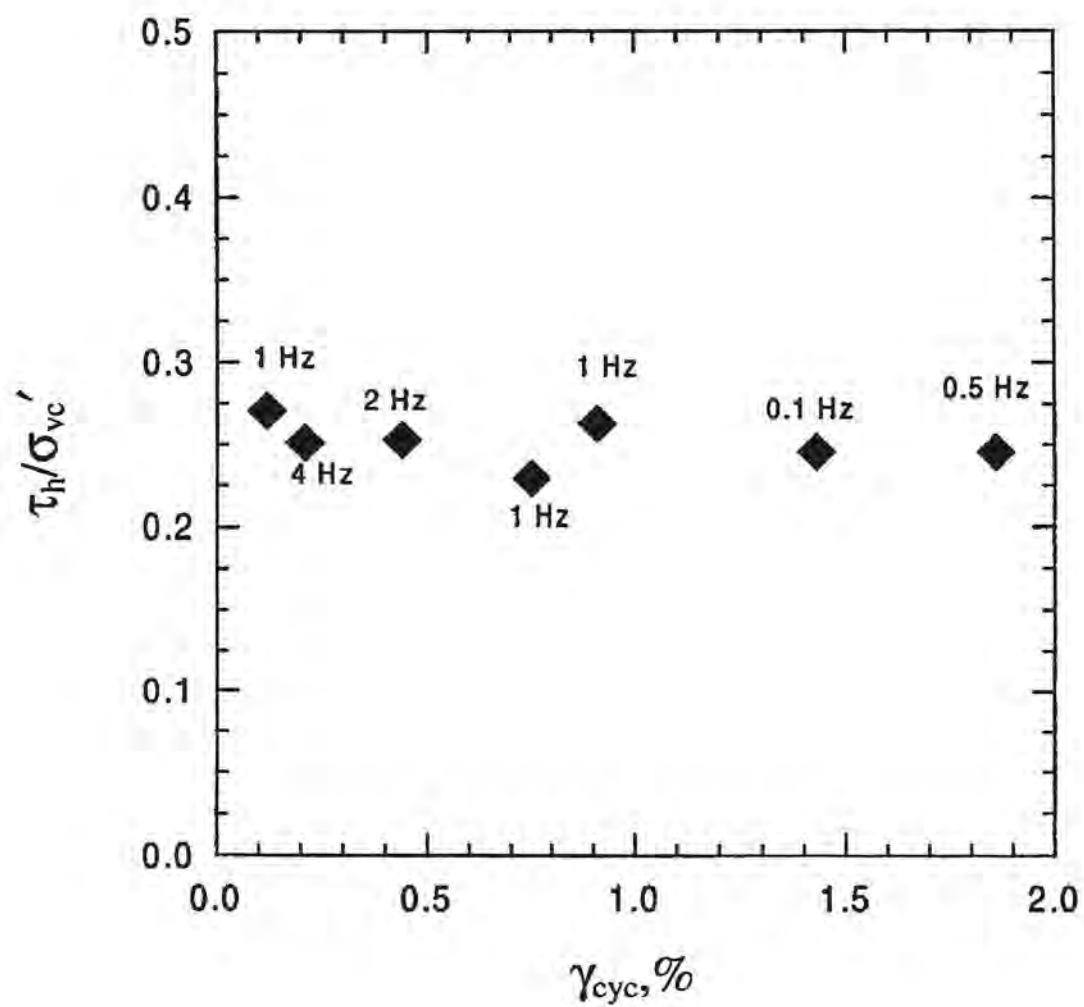


Figure 6.15 Cyclic shear strain effects on post-cyclic strength.

The application of cyclic loads at different frequencies will result in different loading times for specimens subjected to similar strain amplitudes and number of cycles. Because of the viscous nature of clay soils, it is expected that the varying loading times (strain rates) would produce a systematic variation in cyclic degradation and thus post-cyclic strength. In Figure 6.15, the frequency of cyclic loading is labeled beside each data point. Any frequency effect on the post-cyclic strength would be demonstrated by contours of strength at the various levels of frequency, with the highest frequencies producing the highest post-cyclic strengths. The data presented in Figure 6.15 do not support a frequency-dependent relationship between cyclic strain and post-cyclic strength, within the range of frequencies tested.

The average rate of shear strain during cyclic loading is proportional to the product of frequency and cyclic strain amplitude, $f \cdot \gamma$ (for single amplitude strain, the average strain rate is $f \cdot \gamma / 4$). Arango and Seed (1974) proposed the use of $f \cdot \gamma$ as an index to combine the effects of shear strain amplitude and shear strain rate. In Figure 6.16, the data are plotted as $f \cdot \gamma$ vs. post-cyclic strength, with the actual strains labeled beside each data point. The data suggest that at equal strain rates, higher strain amplitudes are accompanied by a reduction in post-cyclic strength, although the data show some scatter.

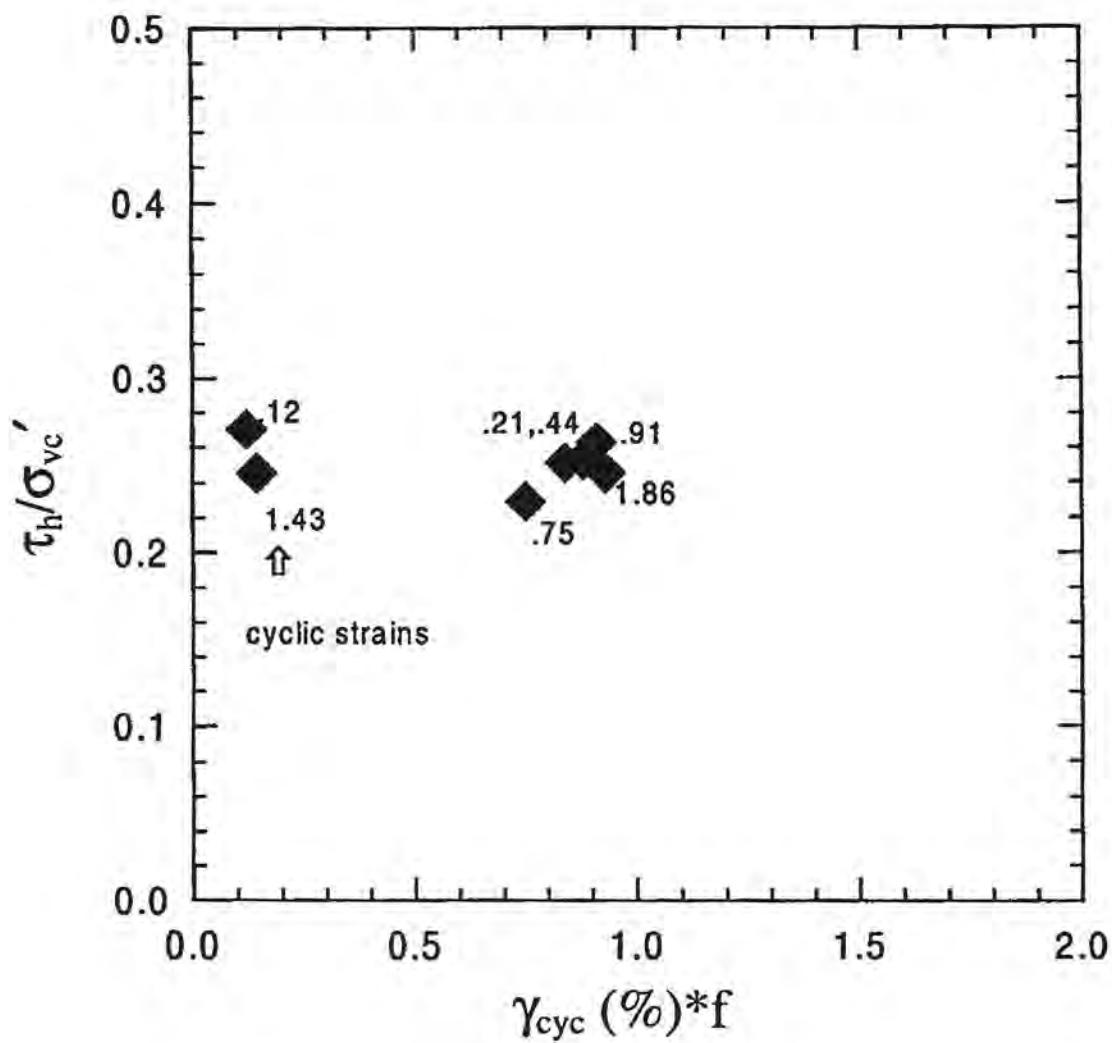


Figure 6.16 Cyclic frequency effects on post-cyclic strength.

6.4.2 Effect of Pore Pressures on Post-Cyclic Strength

Figure 6.17 shows the relationship between strain and pore pressure response in all of the tests where pore pressures were not allowed to dissipate between cyclic and post-cyclic shearing. During the initial stages of monotonic shearing, the pore pressures in all specimens decreased slightly. This initial dilatant response is contrary to what has been observed in similar studies showing initial increasing pore pressures regardless of cyclic strain levels (Andersen et al., 1980; Matsui et al., 1992). The large-strain pore pressure response is fairly consistent between specimens in terms of the final stress state. To achieve the final state, specimens with the higher pore pressure ratios at the beginning of monotonic shear showed increasingly dilatant response, which is consistent with the model of specimens becoming “apparently overconsolidated” with increasing cyclic pore pressure generation, because mechanically overconsolidated specimens are observed to generate increasingly negative pore pressures with increasing OCR under monotonic shear.

Figure 6.14 shows all ten of the post-cyclic monotonic tests performed. In three of these tests, the excess pore pressures were allowed to dissipate prior to shearing the soil in post-cyclic monotonic shear. The average post-cyclic strength for the tests performed without pore pressure dissipation is 0.25, while the average for the three tests where pore pressures were allowed to dissipate is 0.27, an 8% increase in strength. This result is similar to results presented by Yasuhara (1994) for Ariake clay and Fukushima clay, which have similar plasticity indexes to Holocene bay mud.

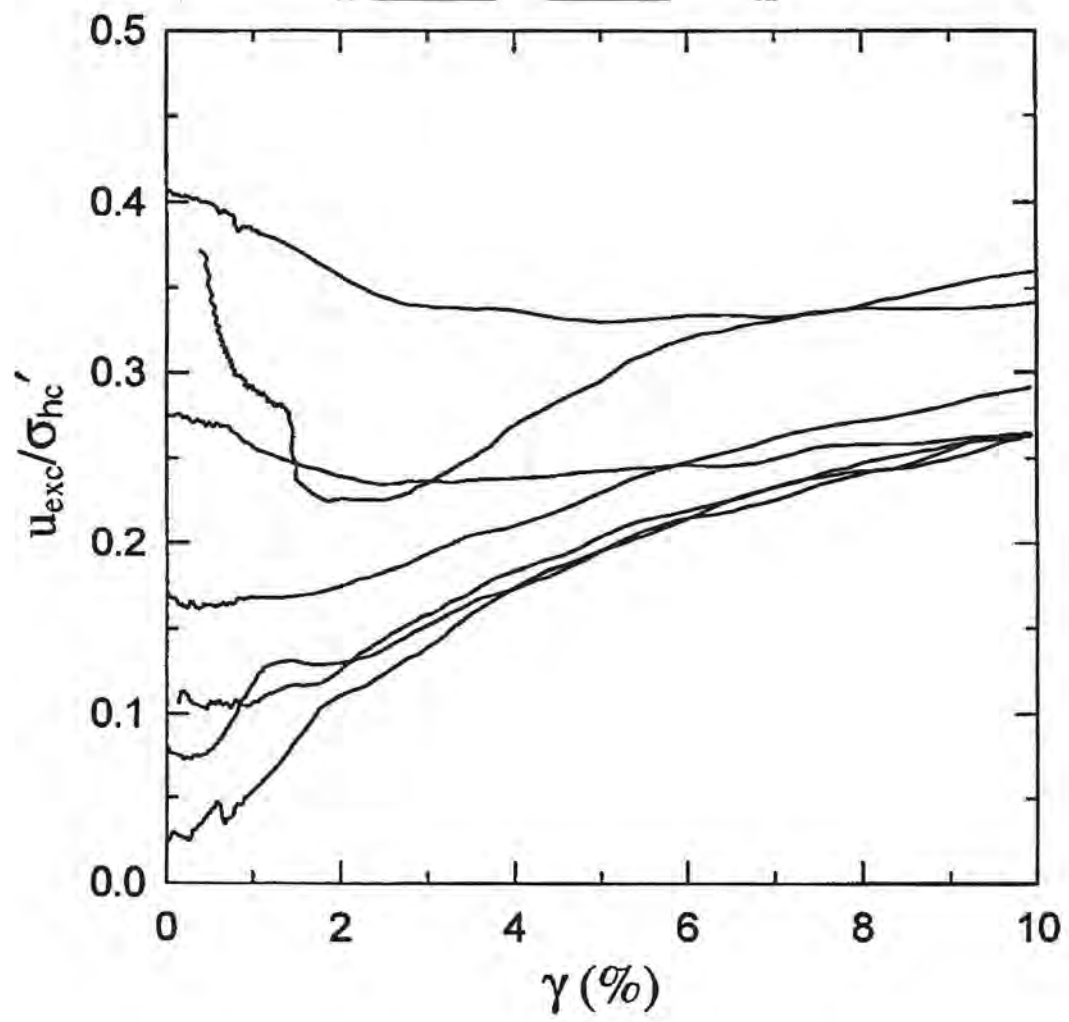


Figure 6.17 Pore pressure development in post-cyclic monotonic tests.

Matsui et al. (1980) showed that there is a linear relationship between the excess pore pressure developed during cyclic loading, and the logarithm of the cyclic strain obtained in stress-controlled, cyclic triaxial testing of Senri clay. This relationship held true even through their tests were performed at different frequencies, confining pressures, cyclic shear stress levels, and numbers of cycles. Matsui et al. (1980) also plotted data from Koutsoftas (1978) and showed that the relationship between excess pore pressure and logarithm of cyclic strain can be drawn for any type of clay at a particular level of OCR.

Figure 6.18 shows the generation of pore pressures at the end of cyclic loading for all of the tests in the current study. The excess pore pressures have been normalized to the initial horizontal effective stress. Although there is a clear relationship between cyclic strain and pore pressure response, there is also some scatter to the data, most of which is probably due to the fact that pore pressures are the least reliable measurements made in this type of testing.

According to the apparent overconsolidation theory, the post-cyclic strength depends on the effective stress state after cyclic loading. Yasuhara (1994), following a theoretical development similar to that of Van Eekelen (1978), proposed equations that relate post-cyclic strength to the compressibility of the soil:

$$\frac{(S_u)_{post-cyclic}}{(S_u)_{static, NC}} = \exp \left[\left(\frac{\Lambda}{1 - C_s / C_e} \right) \ln(AOCR) \right] \quad 6.2$$

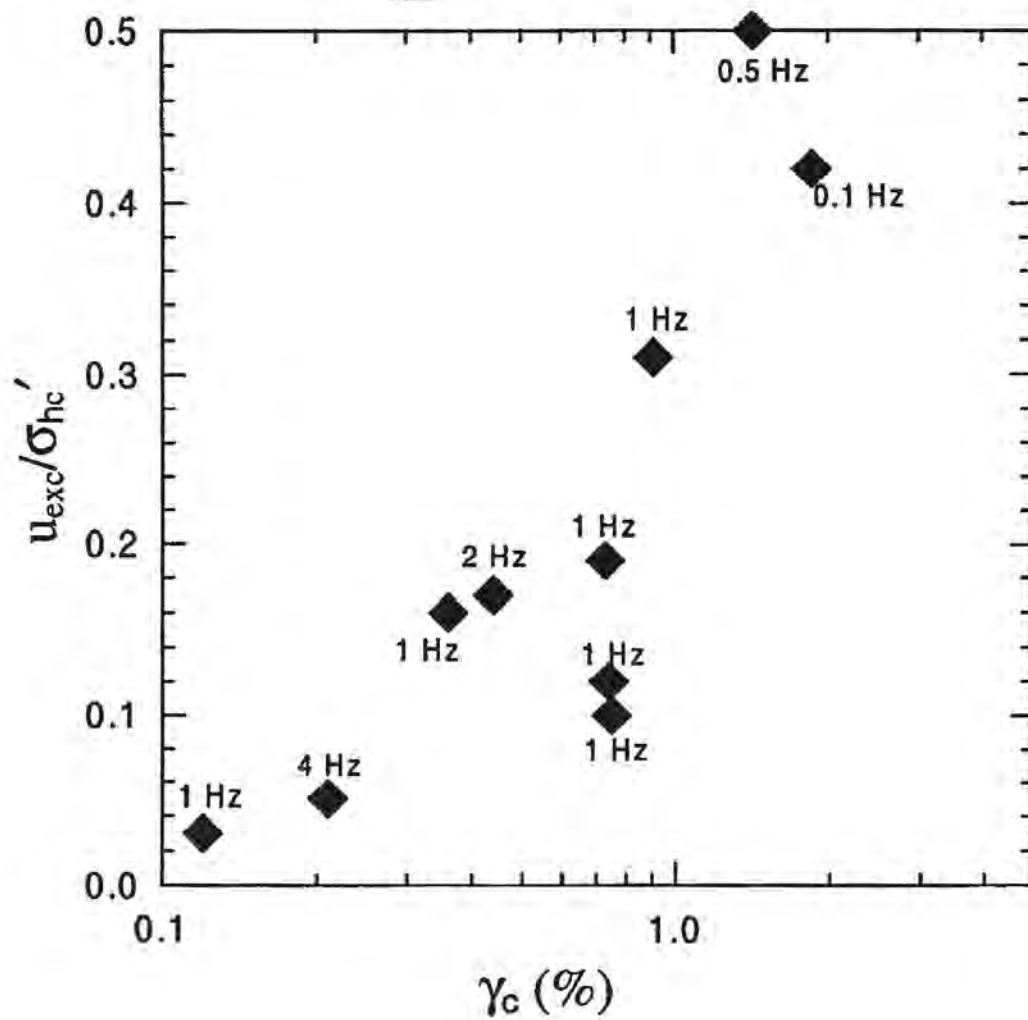


Figure 6.18 Pore pressures generated at the end of cyclic loading. The abscissa shows the cyclic shear strains during the test, and the ordinate shows the normalized pore pressures at the end of cyclic loading.

In the above equation, AOCR is the apparent overconsolidation ratio, defined as the ratio between the initial effective stress and the effective stress at the end of cyclic loading ($AOCR = (\sigma_{vc}')/(\sigma_v', \text{ end of cyclic loading})$). AOCR thus represents the magnitude of pore pressure that is generated during cyclic loading compared to the consolidation stress. C_c and C_s in the equation are the compression and swell indices, respectively. Yasuhara used empirical relationships to relate compressibility to the plasticity index of the soil:

$$C_s/C_c = 0.185 + 0.002PI \quad 6.3$$

$$\Lambda/(1-C_s/C_c) = 0.939 - 0.002PI \quad 6.4$$

Figure 6.19 shows the relationship between AOCR and post-cyclic strength degradation of Holocene bay mud. The contours for plasticity index are predicted by the model proposed by Yasuhara using Equations 6.2 and 6.4. The heavy solid and dashed lines are predicted for bay mud from Hamilton Air Force Base ($PI = 45$) or the Marina District ($PI = 25$) using Equation 6.2 and measured values of the compression and swell indices instead of the empirical correlation with plasticity index. The plotted data include results from the tests performed on Marina District specimens by Sitar and Salgado (1994). For these tests, Figure 6.19 shows that the actual compression and swell indices yield a much better prediction of strength degradation than the empirical relationships proposed by Yasuhara.

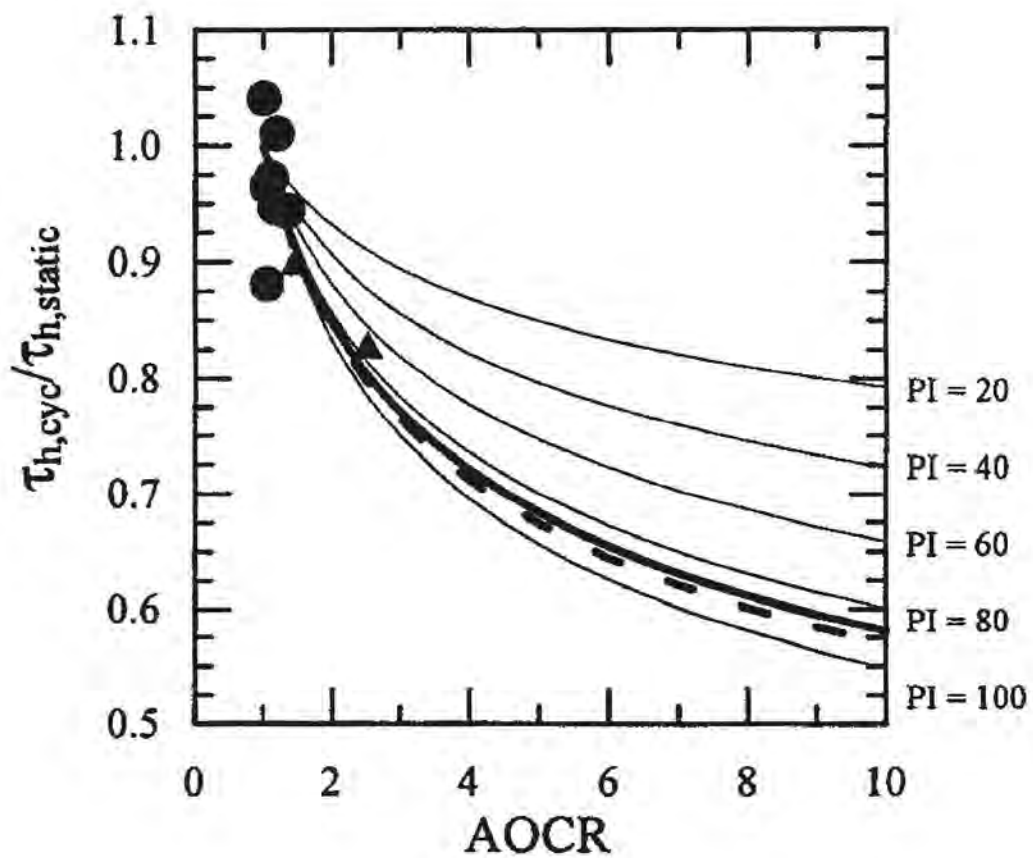


Figure 6.19 Comparison of post-cyclic strength data with apparent overconsolidation model. AOCR = apparent overconsolidation ratio, $(\sigma'_{vc})/(\sigma'_{v, \text{end of cyclic loading}})$. Contours for plasticity index are predicted by Yasuhara (1994), based on empirical relationships between PI and compressibility. Heavy solid and dashed lines are predicted for Holocene bay mud specimens from Hamilton Air Force Base (circles) and Marina District (triangles), based on actual compressibility.

At approximately the same level of cyclic strain, the specimens from the Marina District developed higher pore pressures than those from Hamilton Air Force Base, but the post-cyclic strengths follow roughly the same trend with AOCR. Thus the increased plasticity index of the HAFB specimens appears to correlate with lower pore pressure generation during cyclic loading, and higher post-cyclic strengths at the same levels of cyclic strain.

The observations of the post-cyclic response of HAFB and Marina District specimens follow the same trend of more severe degradation with lower plasticity index as the data taken from the cyclic phase of testing. In the cyclic phase, the Marina District specimens showed a significant degradation in mobilized strength at five or more cycles beyond one or two percent shear strain. In this range, the Marina District specimens experienced higher generation of pore pressures, leading to lower strength in post-cyclic testing.

This response to cyclic loading is consistent with the observations by Vucetic (1994) with respect to threshold strains, and to other authors (Sun et al., 1998; Vucetic and Dobry, 1991) with respect to the degradation of shear modulus. These studies have shown that plasticity index correlates strongly with the development of pore pressures and the degradation of the soil structure leading to a loss in stiffness. The comparison of the response of Holocene bay mud from HAFB and the Marina District thus extends this framework to encompass the entire range of cyclic behavior, from very small strains to strains approaching failure.

CHAPTER 7 SUMMARY AND CONCLUSIONS

7.1 Scope of Research

Understanding the effects of earthquakes on cohesive soils was a low priority for the geotechnical community for a long time, as early geotechnical research focused more attention on the dramatic and sometimes catastrophic liquefaction of loose, saturated sands. The 1985 Mexico City earthquake and the 1989 Loma Prieta earthquake accentuated the potential of clay soils to amplify ground motions and thus cause surprisingly strong shaking in areas far from an earthquake rupture zone. But the long-standing paradigm for clays is that relative to sands they are resistant to cyclic loading and do not exhibit strength degradation similar to liquefaction.

Whether or not this paradigm is correct has serious consequences for structures built on deposits of soft natural clays, as is the case around the San Francisco bay area and elsewhere worldwide. Examination of the case history record shows that seismically-induced slope failures in cohesive soils have occurred in the past, and re-analysis of some landslides first believed to have failed due to liquefaction showed a strong possibility that strength degradation in clay layers was a significant contributing factor.

Following the Loma Prieta earthquake, Salgado and Sitar (1994) performed tests on Holocene bay mud from the Marina District in San Francisco that showed a significant decrease in mobilized shear stress with a low number of cycles (on the order of five

cycles), and a post-cyclic strength that was as much as 17% lower than static strength. Given an engineering climate that included seismic retrofitting and reconstruction of major engineering structures around the San Francisco bay area following the Loma Prieta earthquake, Salgado and Sitar's test results raised important questions about the paradigm for the seismic response of clays, resulting in the current study.

The response of clays to cyclic loading is a balance between two opposing phenomena: an increase in stiffness and strength due to the faster strain rates applied during an earthquake (compared with the strain rates in a monotonic, "static" test), and progressive damage to the soil fabric with cyclic stress reversals, leading to an increase in pore pressures and a decrease in stiffness and strength. The purpose of this study was to observe the response of Holocene bay mud over the range of strains and numbers of cycles to be expected in a moderate to large earthquake, and to evaluate the magnitude of the stiffness and strength decreases as a function of these two parameters.

The study was motivated by the desire to predict the loss of strength leading to permanent deformations in embankments overlying clay deposits, a condition that is found in many places around the San Francisco bay area of California. The insights gained from this study can also be used to approach the analysis of seismic bearing capacity and soil-structure interaction for shallow and deep foundation systems, and is applicable to bay mud and similar cohesive deposits.

7.2 Research Findings

1. Examination of the geotechnical literature and data from related projects around the San Francisco Bay shows that Holocene bay mud in the San Francisco bay area is not a strictly uniform deposit, but varies from location to location and with depth, according to the geologic regime at the time and place of deposition. An example that has important consequences for cyclic resistance is the variability of the Atterberg Limits. The plasticity index for bay mud reported in the literature ranges from 14 to 64.
2. Holocene bay mud from Hamilton Air Force Base (HAFB), in Marin County, California, has an average liquid limit and plasticity index of approximately 90 and 45, respectively. The silty clay from this location has properties that are within the expected range for similar types of clays.
3. A simple numerical study of the response of clay deposits to a vertically propagating shear wave shows that the number of cycles above 1% shear strain in a moderately large earthquake ($M_L = 5.6$ to 7.6) is on the order of between zero and five. Obviously, the soil profile, the spectral shape, and the magnitude of the input accelerations have a significant effect on the distribution of shear strain amplitudes. Nevertheless, these results show that tests with cyclic strain amplitude in excess of 1% are essential to characterize the deformation potential of soft clay deposits.

4. The strength of Holocene bay mud from HAFB measured in monotonic tests is within the acceptable variability for the SHANSEP normalization procedure presented by Ladd and Foott (1974). However, a decreasing trend of normalized strength with consolidation stress suggests an effect of sensitivity on normalized strength parameters; therefore, care was taken in this study to perform tests within a tight range of consolidation stress (σ_{vc}'/σ_p'). The undrained strength ratio that best represents the monotonic strength of samples subjected to cyclic testing (with $K_c = 0.6$ and $\sigma_{vc}'/\sigma_p' = 2.3$) is $S_u/\sigma_{vc}' = 0.25$.
5. The effect of laboratory consolidation technique was shown to have a strong influence on the strength of overconsolidated specimens. Two specimens were taken from one tube, one of which was tested by reconsolidating to just past the in situ stresses (resulting in $OCR = 1.7$) and the other by creating $OCR = 2.0$ using the SHANSEP consolidation/rebound methodology. The SHANSEP-consolidated specimen ($OCR = 2.0$) had a lower undrained strength ratio than the specimen consolidated near the in situ stresses ($OCR = 1.7$), suggesting that some of the strength otherwise contributed by soil structure had been lost as a result of using the SHANSEP procedure on the moderately-sensitive bay mud.
6. Consideration of clay response within the framework of threshold strains proposed by Vucetic (1994) suggests that the higher the plasticity index, the higher the strains required to cause volume or pore pressure changes. This hypothesis is confirmed by numerous studies that show a correlation between plasticity index and the

degradation of shear modulus with strain (Sun et al., 1988; Vucetic and Dobry, 1991).

7. A volumetric cyclic threshold strain of 0.02% to 0.04% shear strain was determined for Holocene bay mud at HAFB based on an examination of pore pressure development in the cyclic tests. This approximately corresponds to a 35 percent reduction in shear modulus, as proposed by Vucetic (1994). The volumetric cyclic threshold shear strain determined in this study is consistent with results by Isenhower and Stokoe (1981) on HAFB soils.
8. The degradation of shear modulus with strain amplitude determined in this study is consistent with data presented by Stokoe and Lodde (1978), who tested Holocene bay mud from HAFB in a resonant column apparatus. Both studies suggest a somewhat more severe modulus degradation of bay mud than estimated from empirical relationships of $G/G_{\max}$ vs. γ to plasticity index developed by Sun et al. (1988) or Vucetic and Dobry (1991). The present study provides some data on the shear modulus of bay mud at higher strains (greater than 0.5% shear strain), which has not been published in earlier studies. The effect of cyclic stress reversal (number of cycles) on shear modulus was much less pronounced than the effect of strain amplitude, across all strain ranges.
9. The damping ratios calculated for the first cycle of loading, while scattered, were consistent with the limits suggested for cohesive soils by Seed and Idriss (1970).

10. The mobilized stress in the first cycle of the cyclic tests was up to 40-50% higher than at equivalent strains in the monotonic tests. This corresponds to a 13-16% increase in mobilized stress per order of magnitude increase strain rate. Lacerda (1976) previously reported a 9% increase for specimens from HAFB tested in ICU triaxial compression. In comparison, Kulhawy and Mayne (1990) and Ishihara (1981) have reported a range of 10 to 15% for all clays.
11. The mobilized stress decreased from 40-50 % of the equivalent stress in a monotonic test in the first cycle, to approximately the monotonic level by the fifteenth cycle. The greatest decrease in mobilized stress occurred between the first and fifth cycles. The numerical study of site response showed that from zero to five or more cycles above 1% shear strain can be expected in a moderately large earthquake ($M_L = 5.6$ to 7.6). This suggests that it is not appropriate to characterize the dynamic strength of bay mud as uniformly higher than the monotonic strength over the duration of an earthquake event. A mechanism to account for cyclic strength degradation could be incorporated into analyses performed in the time domain, or else an appropriate strength could be chosen between the monotonic strength and the strength mobilized in the first cycle.
12. The post-cyclic monotonic strengths were within $\pm 10\%$ of the monotonic strengths of specimens that had not undergone cyclic loading. There were no clear effects of strain amplitude, frequency of loading, or strain rate on the post-cyclic strengths.

However, dissipation of pore pressures between the end of cyclic loading and the start of monotonic testing resulted in an increase of post-cyclic monotonic strength, presumably through the decrease of void ratio through consolidation. This is consistent with the results of Yasuharu (1994) and Matsui et al. (1992) on Japanese clays.

7.3 Recommendations for Future Research

The results of the extensive literature review and laboratory testing of Holocene bay mud suggest that there is a strong correlation between the plasticity index of cohesive soils and cyclic resistance. This is due to the higher cyclic volumetric threshold strain (Vucetic, 1994) of high-plasticity soils. High-plasticity soils require relatively high strains to disturb the electrochemical attractions between particle assemblages and generate excess pore pressures and strain softening behavior. In contrast, cohesionless soils are acted on by body forces only and the frictional contact between particles is easily disturbed, resulting in liquefaction and related phenomena.

The present study provides a fairly comprehensive characterization of the dynamic properties of Holocene bay mud from Hamilton Air Force Base in Novato, California. As such it is a complement to the compilation of static properties of bay mud from HAFB presented in Bonaparte and Mitchell (1979). The soil properties and initial

testing conditions were purposely held constant so that the response of bay mud to strain amplitude and number of cycles could be measured. However, as shown in this study, the plasticity index of bay mud varies across the region from 14 to 64, and as such, one would expect the cyclic resistance of bay mud to vary significantly. In fact, the cyclic and post-cyclic degradation of bay mud samples from the Marina District ($PI = 25$), in comparison with the test results on bay mud from Hamilton Air Force Base ($PI = 45$), is more severe. Accordingly, the next logical step in this line of inquiry is to sample and test bay mud from other locations around the bay area to see how degradation changes with plasticity index.

The other direction suggested by this study is to incorporate the findings into analysis methods for predicting slope deformations. For example, simplified sliding block analysis methods are indiscriminately applied to slopes of all soil types, even though they were developed for very specific applications. Very little is currently understood about the mechanisms of slope deformations in cohesive soils, including the development of localized deformations and the shape of failure surfaces.

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APPENDIX A
Soil Profiles Used in Numerical Response Studies

Profile I: 15 m fill - 20 m bay mud - 50 m old bay clay

Description of layer	Layer #	Soil Type	Thickness ft	Damping %	Unit Weight kips/ft ²	Vs ft/sec
Compacted fill	1	9	5	5	0.120	700
	2	9	5	5	0.120	750
	3	9	5	5	0.120	800
Holocene bay mud light gray silty clay	4	1	3	5	0.095	249
	5	1	5	5	0.095	266
	6	1	5	5	0.095	287
	7	1	7	5	0.095	312
Old Bay Clay (Yerba Buena Mud)	8	6	5	5	0.115	594
	9	6	5	5	0.115	606
	10	6	10	5	0.115	625
	11	6	10	5	0.115	650
	12	6	10	5	0.115	675
	13	6	10	5	0.115	700
Weathered Bedrock	14	3	5	1	0.140	2500
	15	3	5	1	0.140	2500
	16	3	5	1	0.140	2500
	17	3	5	1	0.140	2500
	18	3	5	1	0.140	2500
	19	3	5	1	0.140	2500
Bedrock (Sandstone)	20				0.145	3500

Profile II: 15 m fill - 40 m bay mud - 400 m old bay clay

Description of layer	Layer #	Soil Type	Thickness ft	Damping %	Unit Weight kips/ft ²	Vs ft/sec
Compacted fill	1	9	5	5	0.120	700
	2	9	5	5	0.120	750
	3	9	5	5	0.120	800
Holocene bay mud light gray silty clay	4	1	8	5	0.095	260
	5	1	10	5	0.095	298
	6	1	10	5	0.095	340
	7	1	12	5	0.095	386
Old Bay Clay (Yerba Buena Mud)	8	6	15	5	0.115	656
	9	6	20	5	0.115	700
	10	6	25	5	0.115	756
	11	6	30	5	0.115	825
	12	6	35	5	0.115	906
	13	6	35	5	0.115	1069
	14	6	40	5	0.115	1088
	15	6	45	5	0.115	1194
	16	6	45	5	0.115	1306
	17	6	50	5	0.115	1425
	18	6	60	5	0.115	1563
Weathered Bedrock (Sandstone)	19	3	30	1	0.140	3500
Bedrock (Sandstone)	20				0.145	5000

Profile III: Hamilton Air Force Base

Description of layer	Layer #	Soil Type	Thickness ft	Damping %	Unit Weight kips/ft ²	Vs ft/sec
Holocene bay mud	1	1	3	5	0.095	410
oxidation zone	2	1	5	5	0.095	410
light stiff gray silty clay						
Holocene bay mud	3	1	4	5	0.095	225
light gray silty clay	4	1	4	5	0.095	225
	5	1	4	5	0.095	225
	6	1	5	5	0.095	280
	7	1	5	5	0.095	300
	8	1	6	5	0.095	325
	9	1	6	5	0.095	345
	10	1	8	5	0.095	375
Holocene bay mud	11	12	8	5	0.11	400
"semi-consolidated"						
dark greenish gray clay						
Alluvial Clay	12	6	9	5	0.1	800
Posey Formation	13	6	10	5	0.1	800
light olive brown to light yellowish brown clay						
Old Bay Clay	14	6	13	5	0.1	800
dark greenish gray clay (Yerba Buena Mud)						
Bedrock (Sandstone)	15				0.145	3000

Soil types used in SHAKE analyses

Soil Type

1: Holocene Bay Mud

$\gamma_c, \%$	$G/G_{\max}$	$\lambda, \%$
0.0001	1	1.56
0.001	1	1.56
0.003	0.991	1.56
0.01	0.9416	1.87
0.03	0.8398	2.64
0.1	0.6472	5.44
0.3	0.4512	10.3
1	0.303	17.73
3	0.135	24
10	0.085	27

3: Rock

$\gamma_c, \%$	$G/G_{\max}$	$\gamma_c, \%$	$\lambda, \%$
0.0001	1	0.0001	0.4
0.0003	1	0.001	0.8
0.001	0.9875	0.01	1.5
0.003	0.9525	0.1	3
0.01	0.9	1	4.6
0.03	0.81		
0.1	0.725		
1	0.55		

6: Old Bay Clay
(Clay, PI = 20-40)

$\gamma_c, \%$	$G/G_{\max}$	$\lambda, \%$
0.0001	1	1.55
0.001	0.999	1.55
0.00316	0.98	2.2
0.01	0.92	3.7
0.0316	0.78	6
0.1	0.532	8.7
0.316	0.293	12.5
1	0.137	17
3.16	0.075	21
10	0.025	22.75

9: Compacted Fill
(Sand, $C_p < 1$ ksc)

$\gamma_c, \%$	$G/G_{\max}$	$\gamma_c, \%$	$\lambda, \%$
0.0001	1	0.0001	1
0.000316	0.978	0.001	1.3
0.001	0.934	0.003	3.12
0.00316	0.838	0.01	5.8
0.01	0.672	0.03	9.5
0.0316	0.463	0.1	15.4
0.1	0.253	0.3	20.9
0.316	0.14	1	25
1	0.057	10	25.5

APPENDIX B
Undrained Monotonic Simple Shear Test Results

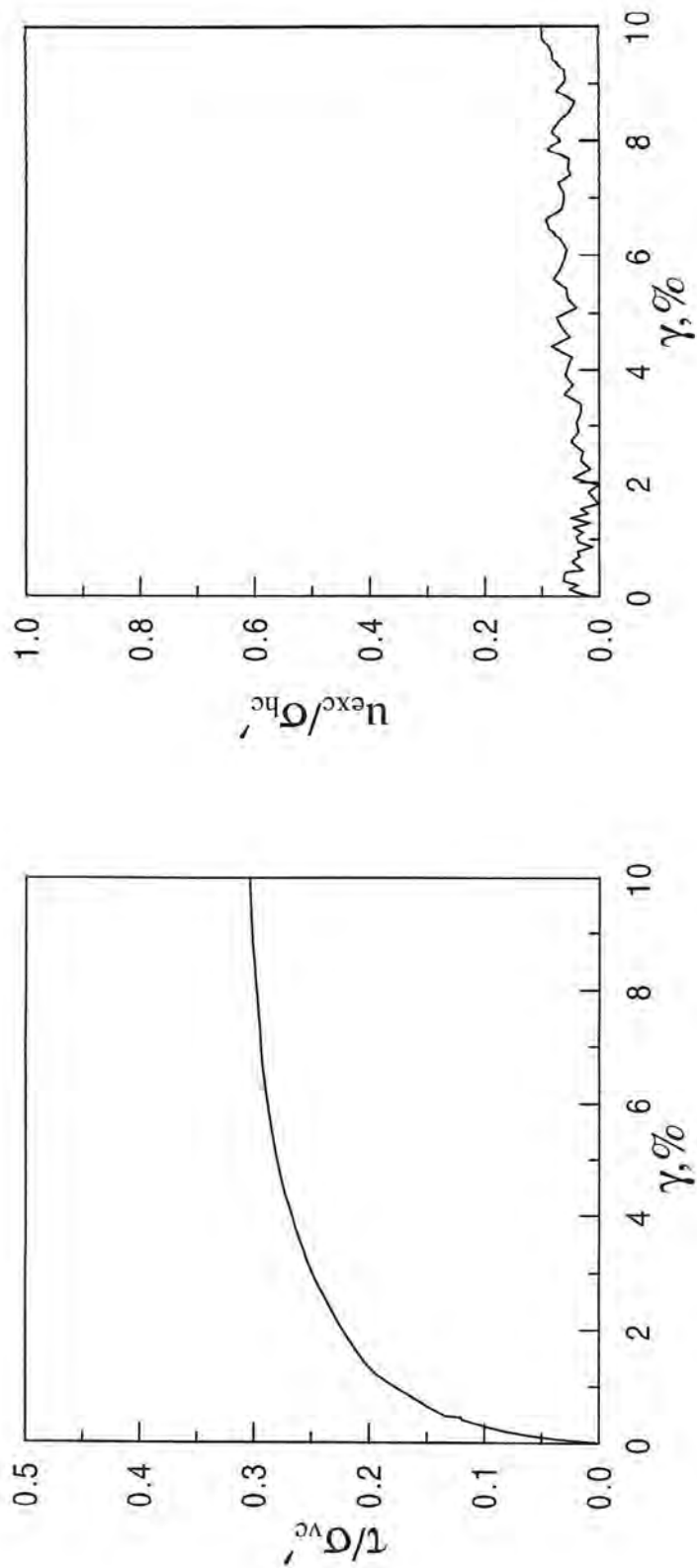


Figure B-1 Monotonic Test MONO-1

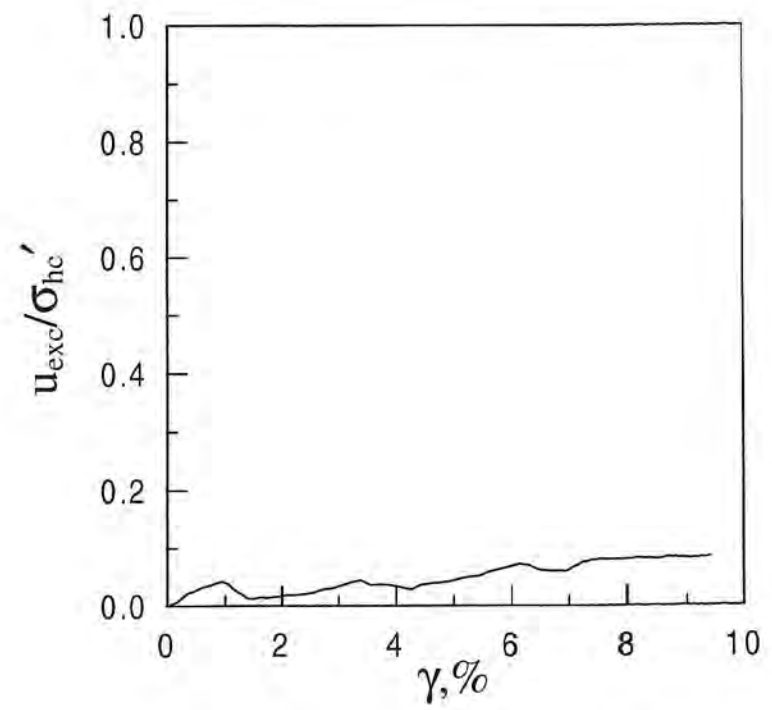
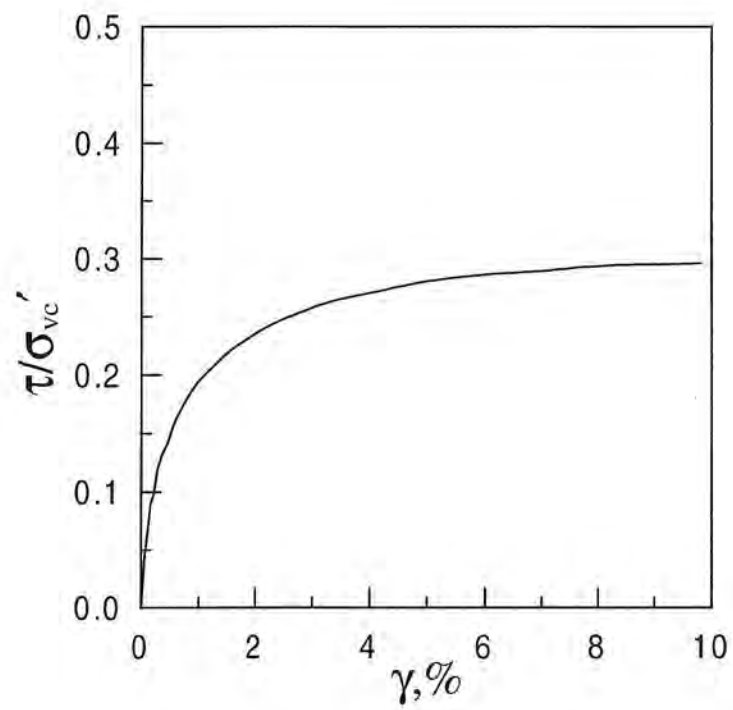


Figure B-2 Monotonic Test MONO-2

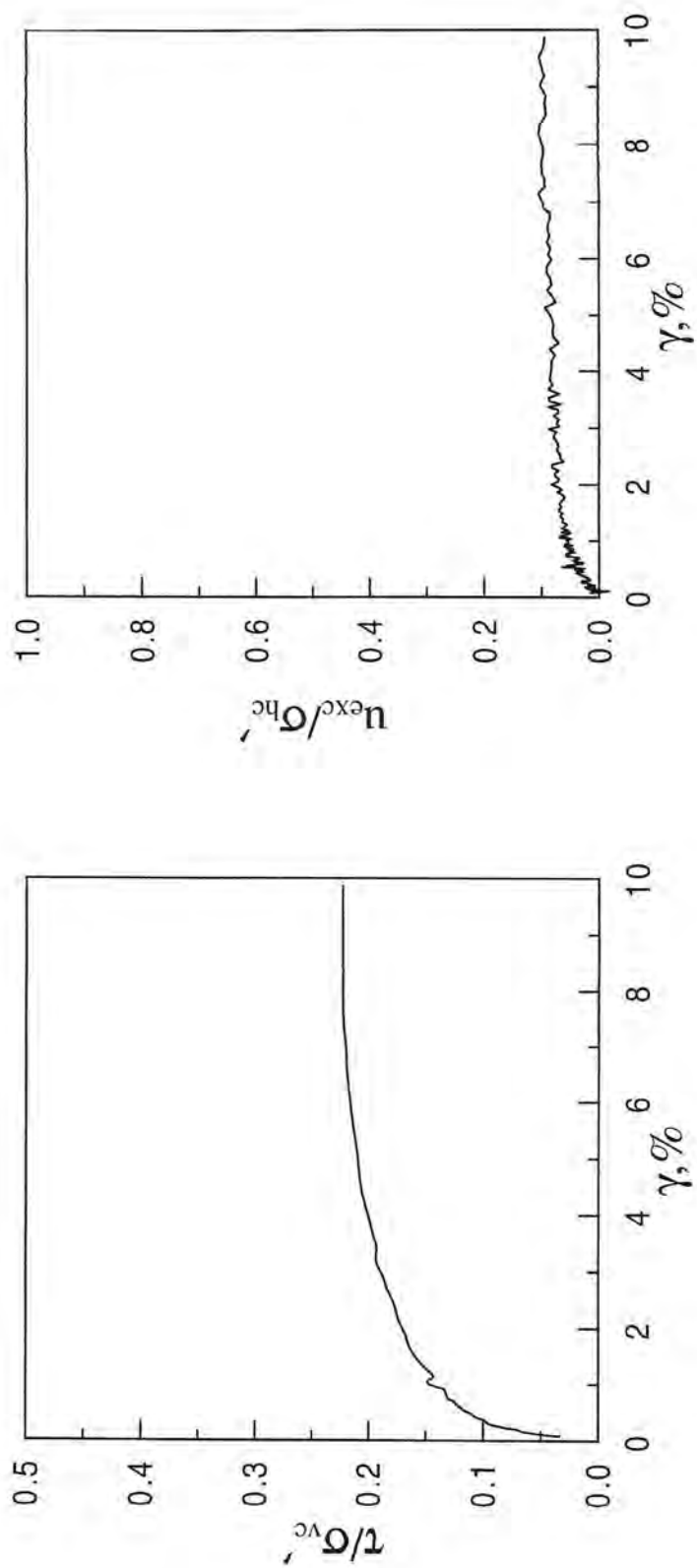


Figure B-3 Monotonic Test MONO-3

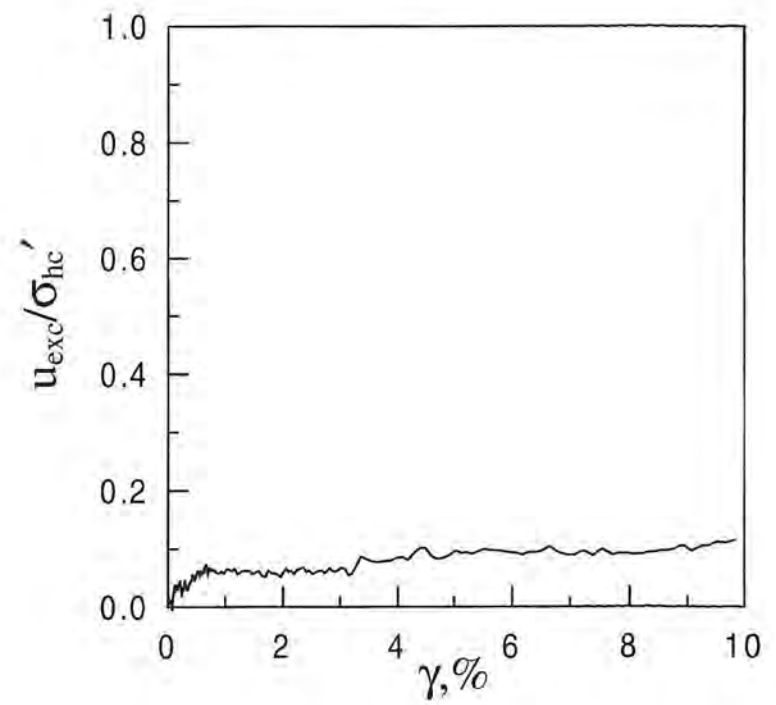
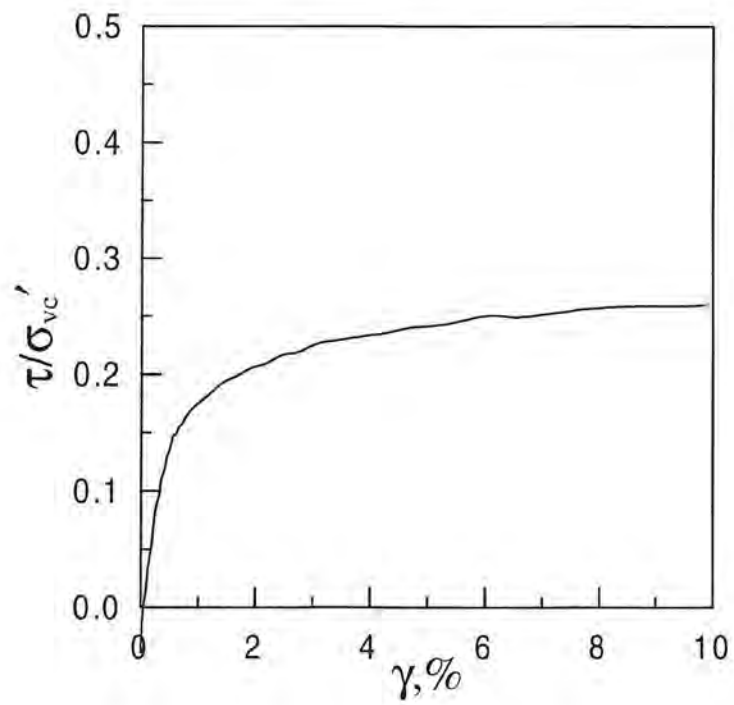


Figure B-4 Monotonic Test MONO-4

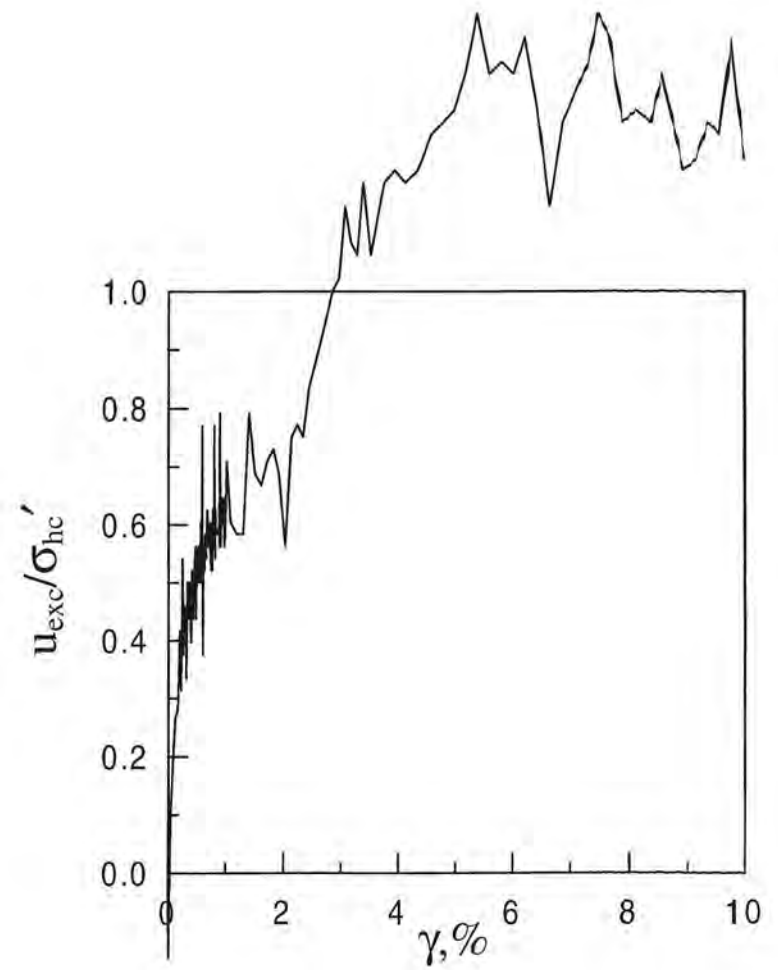
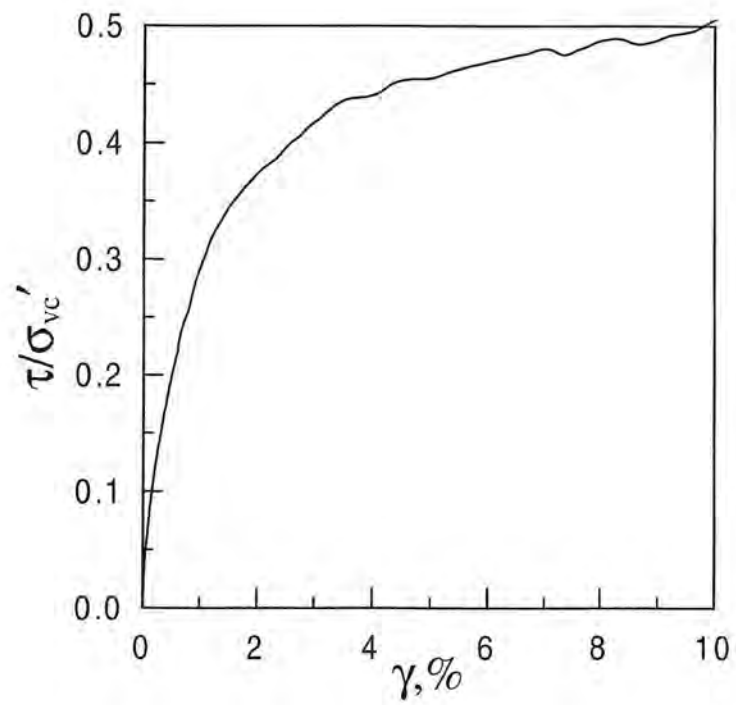


Figure B-5 Monotonic Test MONO-5

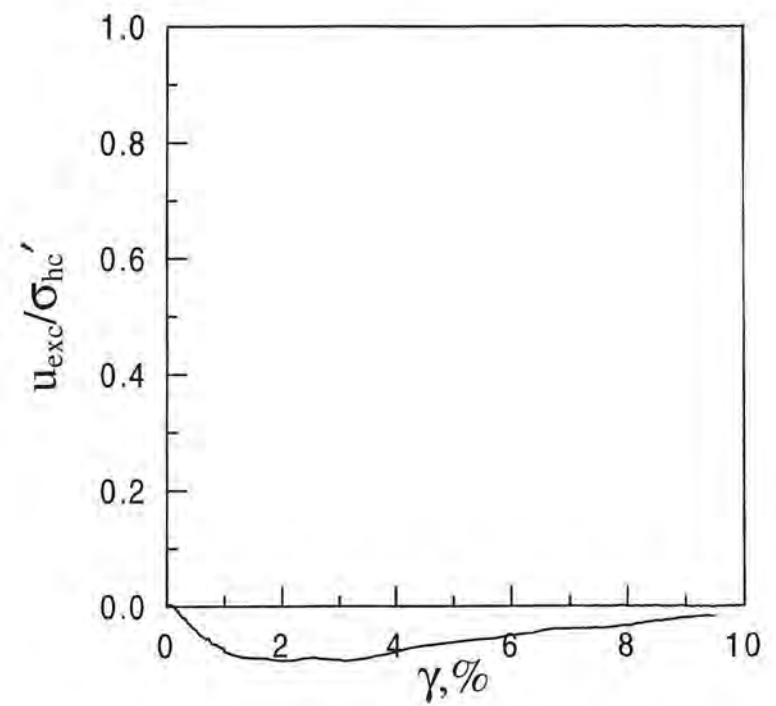
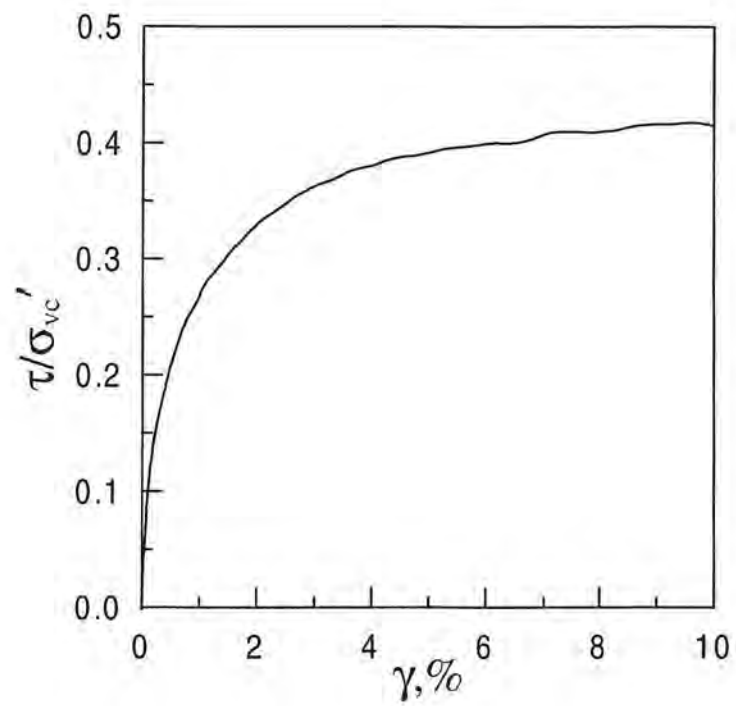


Figure B-6 Monotonic Test MONO-6

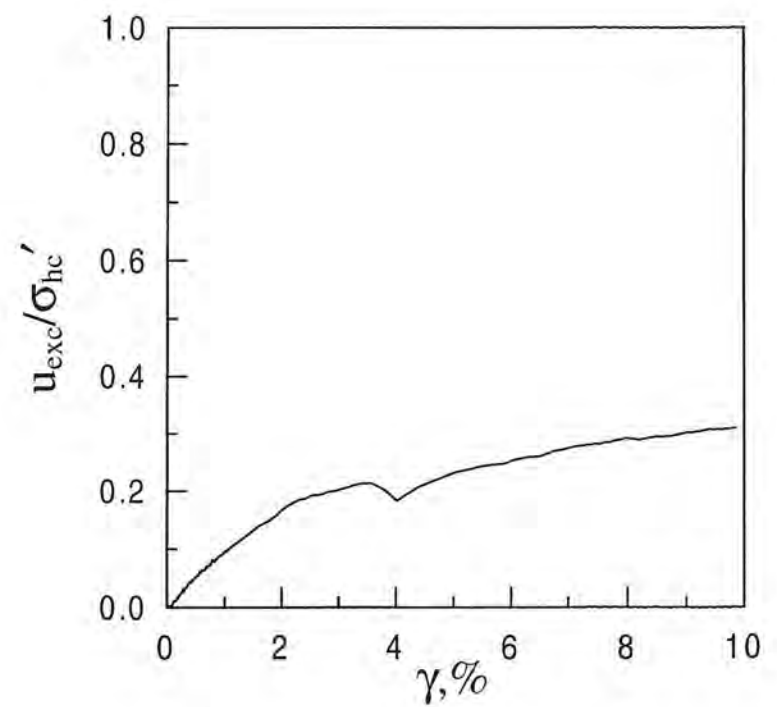
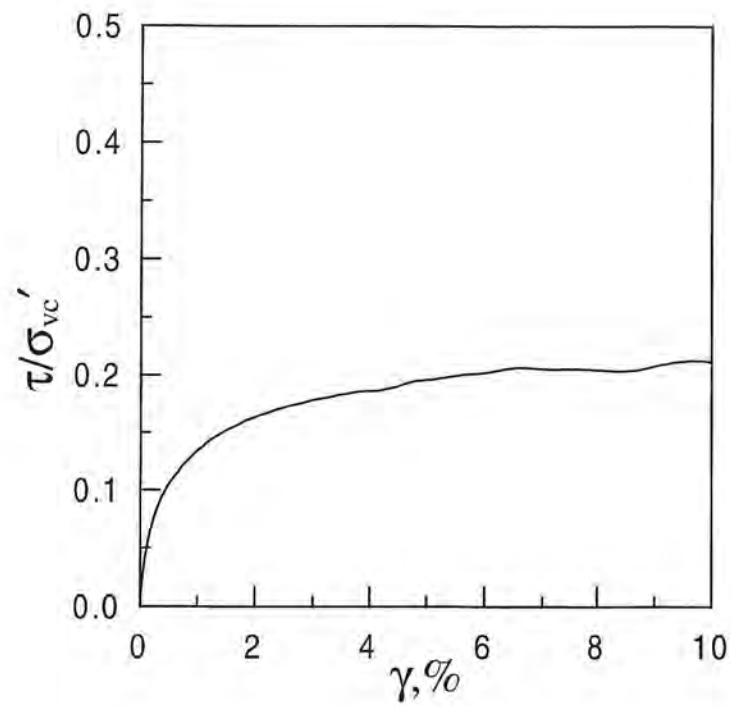


Figure B-7 Monotonic Test MONO-7

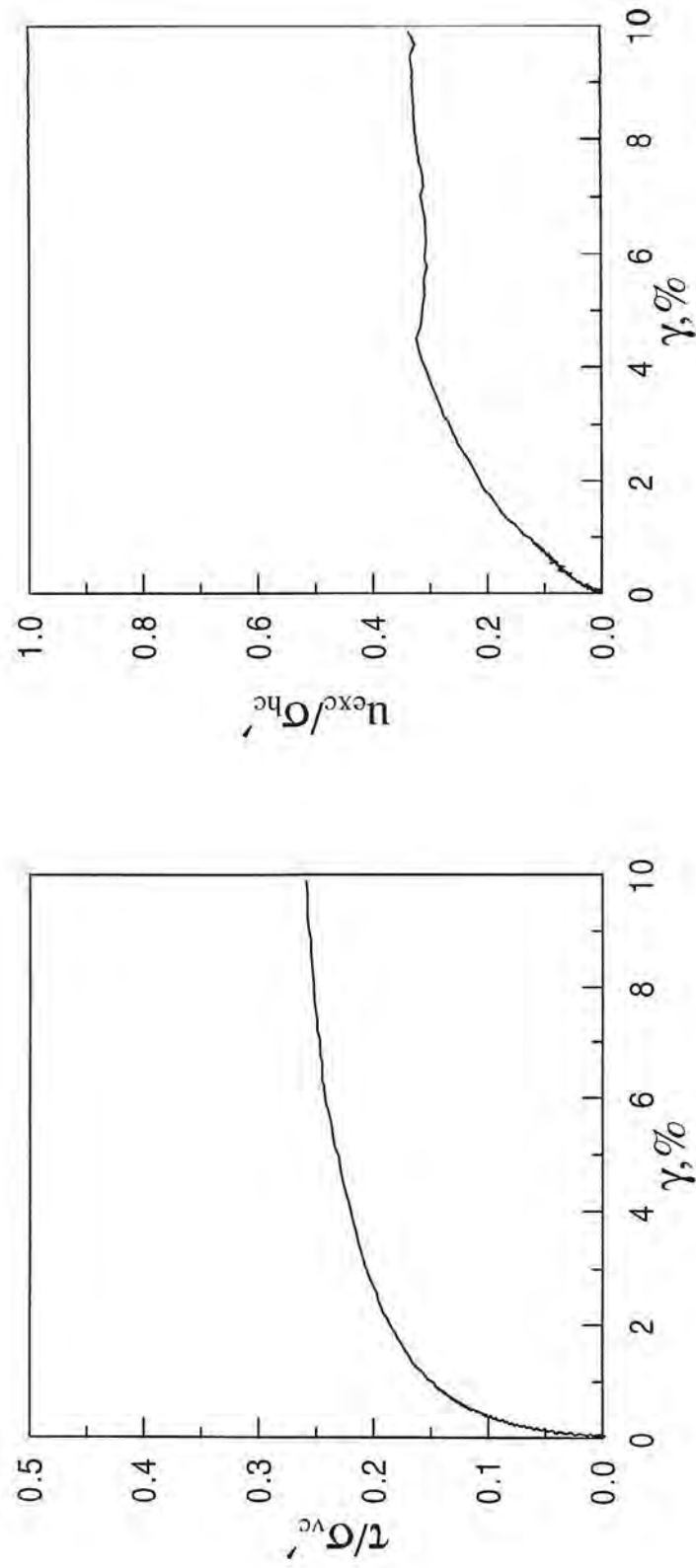


Figure B-8 Monotonic Test MONO-8

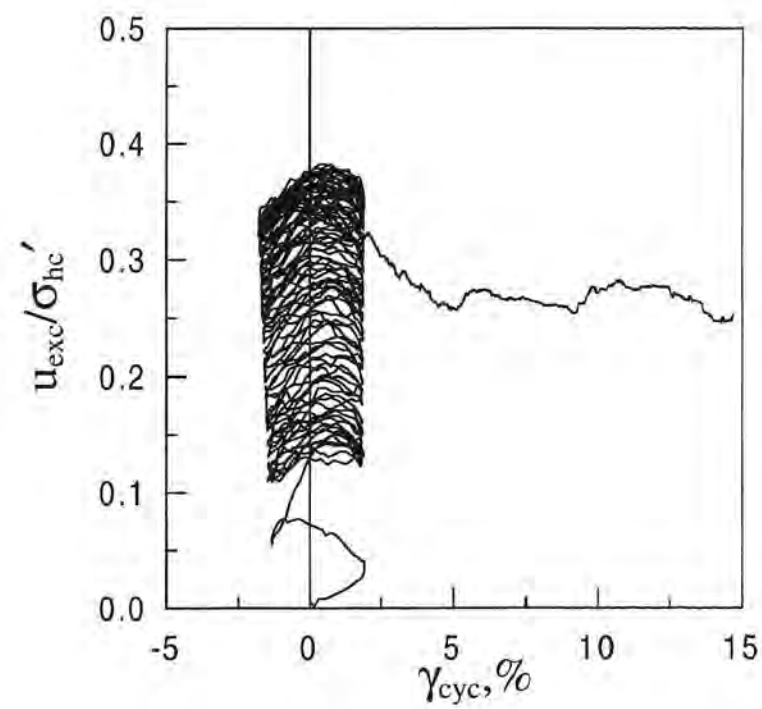
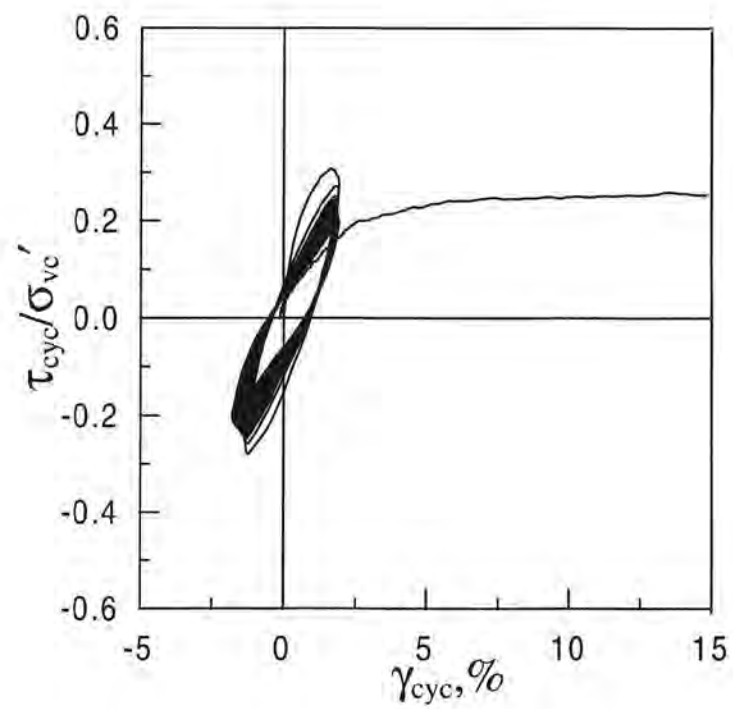


Figure C-1 Cyclic Simple Shear Test CY-1

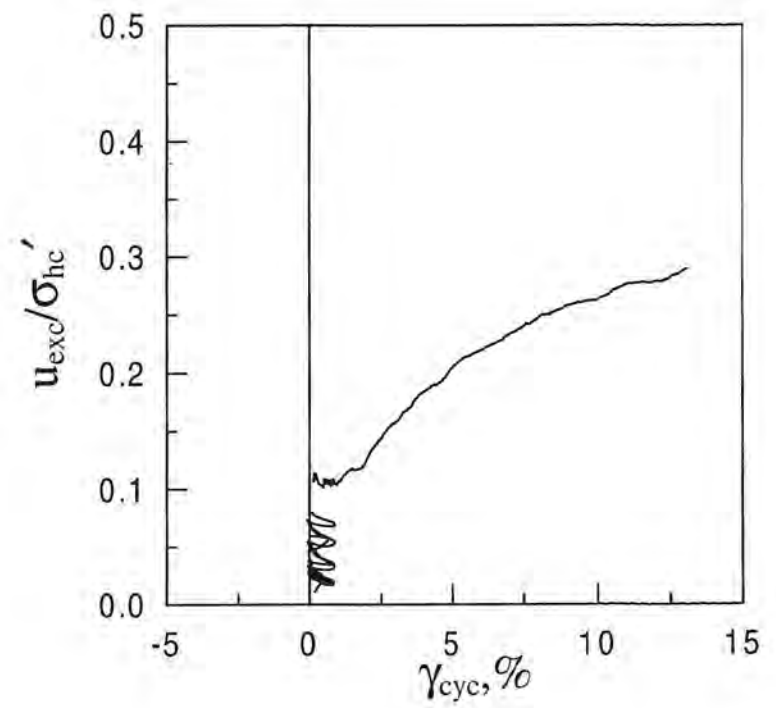
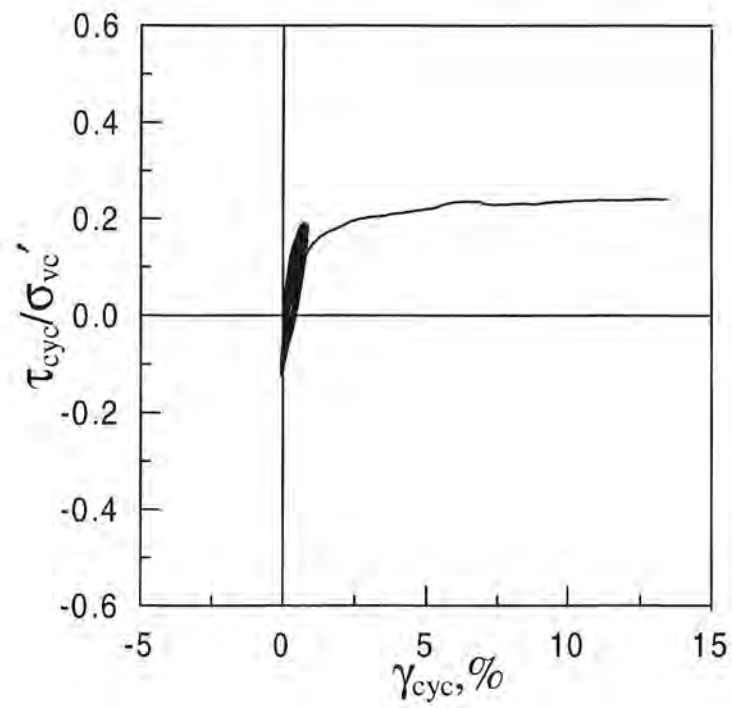


Figure C-3 Cyclic Simple Shear Test CY-3

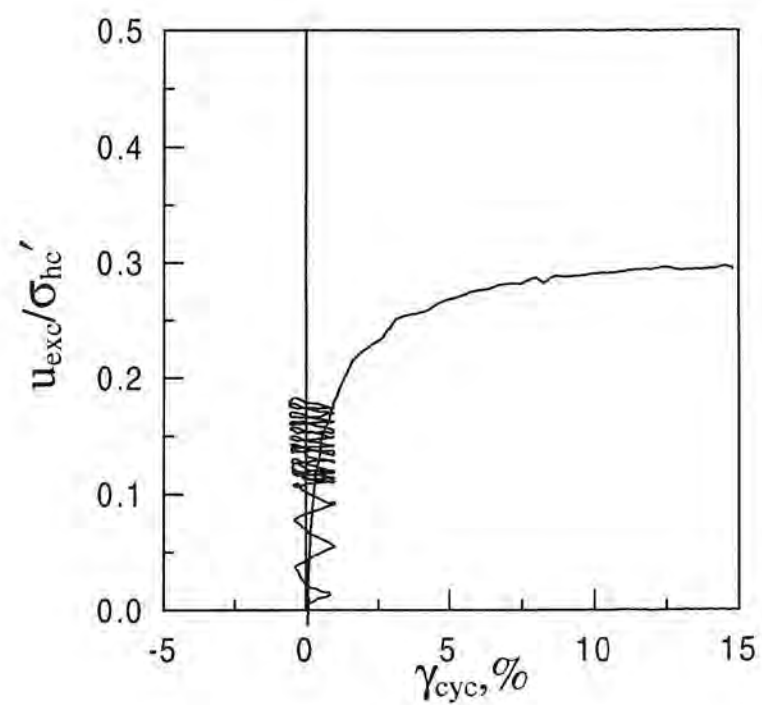
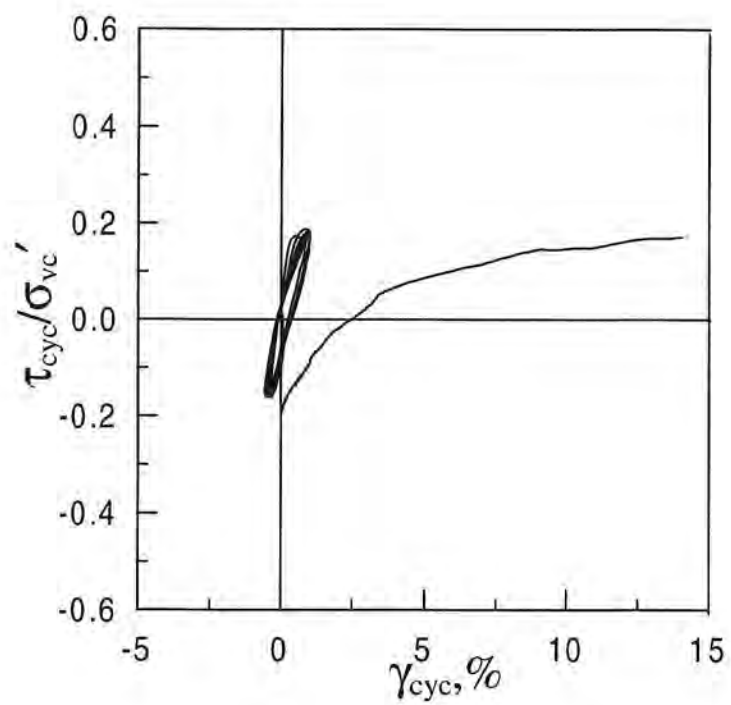


Figure C-5 Cyclic Simple Shear Test CY-5

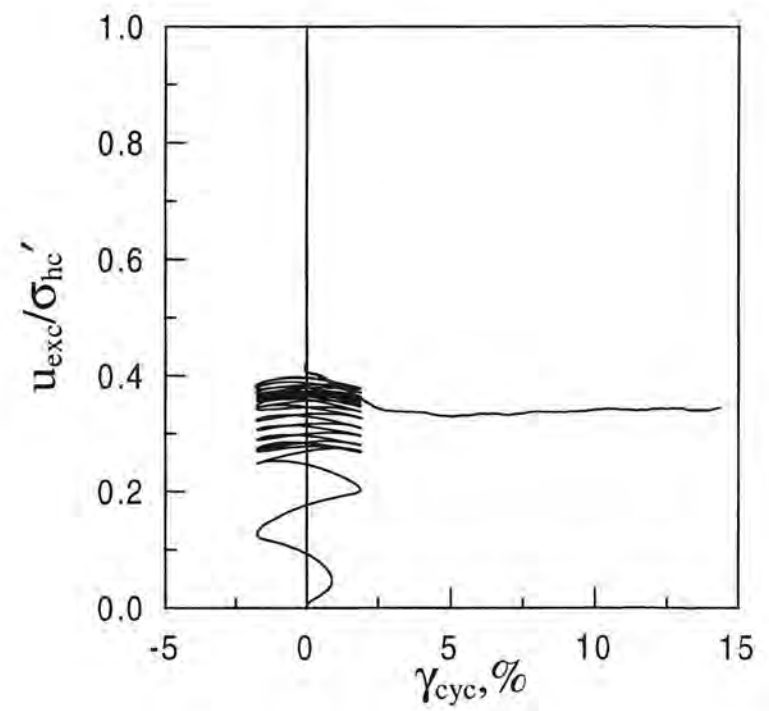
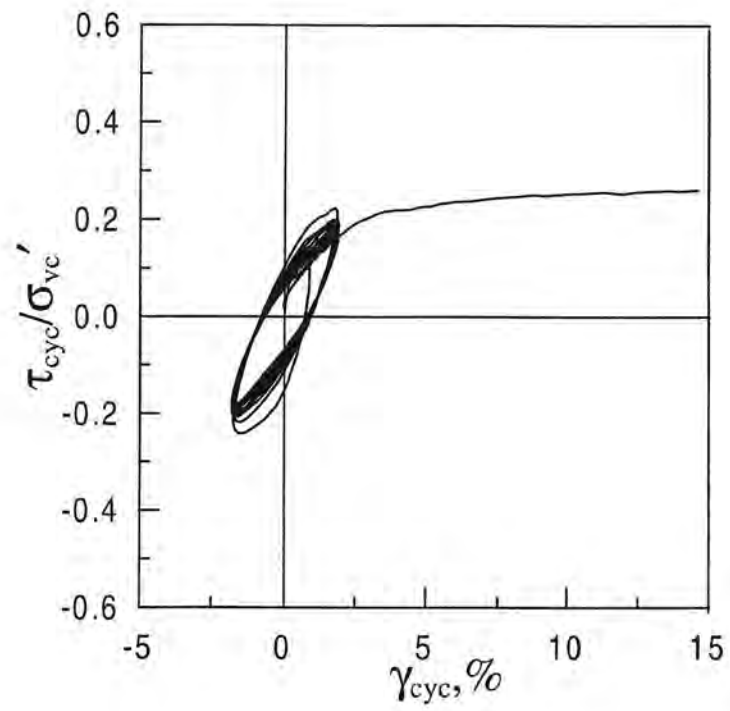


Figure C-10 Cyclic Simple Shear Test CY-10