1980

Stress-Strain and Strength Properties of Marine Sediments from the North Pacific

Stephen A. Akers
University of Rhode Island

Follow this and additional works at: <https://digitalcommons.uri.edu/theses>

Recommended Citation

Akers, Stephen A., "Stress-Strain and Strength Properties of Marine Sediments from the North Pacific" (1980). *Open Access Master's Theses*. Paper 1002.
<https://digitalcommons.uri.edu/theses/1002>

This Thesis is brought to you for free and open access by DigitalCommons@URI. It has been accepted for inclusion in Open Access Master's Theses by an authorized administrator of DigitalCommons@URI. For more information, please contact digitalcommons-group@uri.edu.

STRESS-STRAIN AND STRENGTH PROPERTIES OF MARINE
SEDIMENTS FROM THE NORTH PACIFIC

BY

STEPHEN A. AKERS

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF THE
REQUIREMENTS FOR THE DEGREE OF
MASTER OF SCIENCE
IN
OCEAN ENGINEERING

UNIVERSITY OF RHODE ISLAND

1980

MASTER OF SCIENCE THESIS

OF

STEPHEN A. AKERS

Approved:

Thesis Committee

Major Professor

Armand J. Silva
V.G. Napp
E.E. McNamee
A. A. Michel

Dean of the Graduate School

UNIVERSITY OF RHODE ISLAND

1980

ABSTRACT

The objective of this investigation was to study the strength and stress-strain properties of "undisturbed" and reconstituted-reconsolidated (remolded) samples of two Pacific (MPG-1) deep-sea sediments; illite and smectite. Isotropically consolidated undrained (CIU) triaxial tests were conducted on eleven series of samples, from which the basic soil parameters of effective cohesion ($\bar{c}$) and effective angle of friction ($\bar{\phi}$) were determined.

An undisturbed smectite series (LL-44 GPC-3 1800 cm) had a friction angle of 35.5° and two undisturbed illite series (MA-02 GC-04 100 and 200 cm) had friction angles of 33.0° and 34.8° . Friction angles for the smectite sediments were apparently not significantly affected by the type of remolding-reconsolidation process used in the test program, while illite samples exhibited a 30% reduction in friction angle due to remolding. Compared under the same conditions, undisturbed samples had greater drained and undrained strengths than the remolded samples, and smectite samples had greater drained and undrained strengths than the illite samples.

Using a hyperbolic stress-strain theory, the tangent Young's moduli and hyperbolic deviator stresses were determined for the samples. With the exclusion of samples which exhibited strain softening behavior, there was an excellent correlation between the experimental and hyperbolic

values.

A new lateral strain gauge was built to measure lateral deformations of the samples during uniaxial compression. From these measurements, values of Poisson's ratio were calculated assuming a parabolic deformational shape. For a majority of samples, the standard area correction underestimated the true area of the samples.

CHAPTER 1	INTRODUCTION	1
1.1	Project Description	1
1.2	Task Selection	1
1.3	Efficient Selection	1
1.4	Role Closure Program	1
1.5	Related Subscribed Programs	1
1.6	OSL/OSL Queue-Static Variables Program and Objectives	1
CHAPTER 2	SAMPLING AND TESTING PROCEDURES	11
2.1	Introduction	11
2.2	Study Site MFC-1, CORE	11
2.3	General Specific	11
2.4	Individual Display	11
2.5	DATA DISPLAY AND CONTROL	11
2.6	DATA-01 CODE	11
2.7	DATA-02 CODE	11
2.8	Importance of MFC-1 CODE	11
2.9	Role / Sample Testing	11
2.10	Data Presentation	11
CHAPTER 3	TECHNICAL EQUIPMENT AND PROCEDURES	11
3.1	Basic Procedures	11

TABLE OF CONTENTS

	<u>Page</u>
ACKNOWLEDGEMENTS.....	vii.
LIST OF TABLES.....	ix.
LIST OF FIGURES.....	x.
LIST OF FIGURES FOR APPENDIX A.....	xviii.
LIST OF SYMBOLS.....	xix.
CHAPTER 1 INTRODUCTION.....	1
1.1 Program Objectives.....	1
1.2 Site Selection.....	1
1.3 Sediment Selection.....	3
1.4 Hole Closure Program.....	5
1.5 Related Subseabed Programs.....	6
1.6 URI/MGL Quasi-static Triaxial Program and Objectives.....	9
CHAPTER 2 SAMPLING AND TESTING PROGRAM.....	13
2.1 Introduction.....	13
2.2 Study Site MPG-1, North Central Pacific.....	13
2.3 Undisturbed Samples.....	14
2.4 LONG LINES-44 Cruise.....	14
2.5 VEMA-32 Cruise.....	17
2.6 MARA-02 Cruise.....	17
2.7 Comparisons of MPG-1 Cores.....	19
2.8 Hole Closure Program.....	20
2.9 Data Presentation.....	22
CHAPTER 3 TRIAXIAL EQUIPMENT AND PROCEDURES.....	31
3.1 Basic Procedures.....	31

TABLE OF CONTENTS (cont'd)

	<u>Page</u>
3.2 Equipment.....	31
3.3 Sample Set-up and Test Procedures.....	33
CHAPTER 4 LATERAL STRAIN AND POISSON'S RATIO.....	39
4.1 Introduction.....	39
4.2 Disk-Ring Gauge.....	41
4.3 URI/MGL Lateral Strain Gauge.....	43
4.4 Poisson's Ratio.....	45
4.5 Calculated Poisson's Ratio.....	49
CHAPTER 5 STRESS-STRAIN MODELS.....	57
5.1 Elastic Models.....	57
5.2 Nonlinear Models.....	59
5.3 Mohr-Coloumb Failure Theory.....	60
5.4 Hyperbolic Model.....	61
5.5 Hyperbolic Data Analysis.....	65
CHAPTER 6 PRESENTATION OF RESULTS AND DATA ANALYSIS.....	71
6.1 Introduction.....	71
6.2 Discussion of Stress-Strain Data.....	72
6.3 Drained Strength Analysis.....	87
6.4 Undrained Strength Analysis.....	97
6.5 Hyperbolic Data Analysis.....	106
6.6 Lateral Strain and Poisson's Ratio.....	113
CHAPTER 7 DISCUSSION OF RESULTS.....	234
7.1 Introduction.....	234

TABLE OF CONTENTS (cont'd)

	<u>Page</u>
7.2 Mohr-Coulomb Parameters.....	235
7.3 Undisturbed versus Remolded Samples.....	237
7.4 Smectite versus Illite Samples.....	239
7.5 Vertical versus Horizontal Samples.....	240
7.6 Hyperbolic Theory.....	241
7.7 Poisson's Ratio.....	242
CHAPTER 8 CONCLUSIONS.....	246
APPENDIX A CIU TRIAXIAL TESTING MANUAL.....	248
A.1 Introduction.....	249
A.2 Equipment.....	249
A.3 Reference Literature.....	250
A.4 Sample Set-up.....	250
A.5 Cell Preparation.....	254
A.6 Line Pressures.....	256
A.7 Test Procedures.....	258
A.8 HP Triax Program Preparation.....	259
A.9 Final Test Preparations.....	262
A.10 Plotting Test Data.....	263
A.11 Summary.....	265
A.12 Data Sheets.....	265
A.13 Computer Listing.....	265
REFERENCES.....	282

ACKNOWLEDGEMENTS

The author wishes to express gratitude to Dr. Armand J. Silva, major advisor, for his critical review of the text, for his advice and support, and for the educational opportunities experienced as his research assistant. Professor V.A. Nacci is thanked for his review of the text, his advice, and for the inspiration that was felt as one of his students. Dr. Everett McEwen is thanked for his review of the text during the final writing.

Particular thanks go to David I. Calnan for his advice and assistance in the lab and to Greg Chiarello for his supervision of the classification testing. Mr. Wilson Lamb is acknowledged for his help with all those electrical gadgets. The author extends his uttermost appreciation to Kathy Stein for her typing of the final draft and for her friendship.

I wish to thank all the many friends I have met and worked with at URI, especially: Ken Baldwin, Jim Hetherman, Scott Bamford, Peter Lemmond, Stephan Criscenzo, Roger Morin, Sylvie Schwartz, Greg Walker, Osman Pekin and David Carchedi. Special and sincere thanks go to Kate Moran and Jan Wildman, whose friendship and assistance has helped me through many a troubled time.

Funding for this research was provided mainly by the Department of Energy - Contract Numbers 13-2561, 13-9927 and by Office of Naval Research, Contract Number N00014-

76-C-0226. This support is gratefully acknowledged.

<u>Table</u>	<u>Title</u>	<u>Page</u>
6.1	Material Properties.....	120
6.2	Triaxial Failure Parameters.....	122
6.3	Triaxial Test Results.....	123
6.4	Normalized Strengths.....	124
7.1	One-Dimensional Parameters.....	243
7.2	One-Dimensional Normalized and Normalized Strengths.....	245

LIST OF TABLES

<u>Table</u>	<u>Title</u>	<u>Page</u>
6.1	Material Properties.....	120 & 121
6.2	Triaxial Failure Parameters.....	122 & 123
6.3	Triaxial Test Results.....	124
6.4	Normalized Strengths.....	125 & 126
7.1	Mohr-Coloumb Parameters.....	244
7.2	Average Normalized Undrained and Drained Strengths.....	245

LIST OF FIGURES

<u>Figure Number</u>		<u>Page</u>
1.1	Triaxial Testing Program.....	12
2.1	Mid-Plate Gyre (MPG-1) Study Area.....	23
2.2	Mid-Plate Gyre Region 1 - Sediment Thickness.....	24
2.3	Core Locations in MPG-1.....	25
2.4	Water Content and Shear Strength versus Depth: LL-44 GPC-3.....	26
2.5	GPC-3 Visual Description.....	27
2.6	Water Content versus Depth: V-32 PC-115.....	28
2.7	Water Content and Shear Strength versus Depth: MA-02 GC-04.....	29
2.8	MPG-1 Core Comparison.....	30
3.1	Triaxial Pressure System.....	36
3.2	Cell Manifold and Base.....	37
3.3	Wykeham-Farrance Triaxial Cell.....	38
4.1	Disk Gauge or Rock Specimen.....	52
4.2	URI/MGL Lateral Strain Gauge.....	53
4.3	Machining Jig and Disk-Ring.....	54
4.4	Sample with Lateral Strain Gauge and Metal Tabs Attached.....	55
4.5	Sample Deformation.....	56
5.1	Hyperbolic Representation of a Stress-Strain Curve.....	67
5.2	Variation of Initial Tangent Modulus with Confining Pressure.....	68
5.3	Hyperbolic Deviator Stress vs. Strain.....	69

LIST OF FIGURES (Cont'd)

<u>Figure Number</u>		<u>Page</u>
5.4	Tangent Modulus vs. Deviator Stress.....	70
6.1	Deviator Stress vs. Strain: LL-44 GPC-3 1800 cm.....	129
6.2	Pore Pressure vs. Strain: LL-44 GPC-3 1800 cm.....	130
6.3	Stress Path (q vs. p): LL-44 GPC-3 1800 cm.....	131
6.4	A-Factor vs. Strain: LL-44 GPC-3 1800 cm.....	132
6.5	Vertical/Lateral ($\bar{\sigma}_1/\bar{\sigma}_3$) vs. Strain: LL-44 GPC-3 1800 cm.....	133
6.6	Lateral Strain vs. Vertical Strain: LL-44 GPC-3 1800 cm.....	134
6.7	Poisson's Ratio vs. Strain: LL-44 GPC-3 1800 cm.....	135
6.8	Deviator Stress vs. Strain: PS-9 Vertical.....	136
6.9	Pore Pressure vs. Strain: PS-9 Vertical.....	137
6.10	Stress Path (q vs. p): PS-9 Vertical.....	138
6.11	A-factor vs. Strain: PS-9 Vertical...	139
6.12	Vertical/Lateral ($\bar{\sigma}_1/\bar{\sigma}_3$) vs. Strain: PS-9 Vertical.....	140
6.13	Lateral Strain vs. Vertical Strain: PS-9 Vertical.....	141
6.14	Poisson's Ratio vs. Strain: PS-9 Vertical.....	142
6.15	Hyperbolic Deviator Stress vs. Strain: PS-9 Vertical.....	143
6.16	Tangent Modulus vs. Deviator Stress: PS-9 Vertical.....	144

LIST OF FIGURES (Cont'd)

<u>Figure Number</u>		<u>Page</u>
6.17	Deviator Stress vs. Strain: PS-9 Vertical.....	145
6.18	Pore Pressure vs. Strain: PS-18 Vertical.....	146
6.19	Stress Path (q vs. p): PS-18 Vertical.....	147
6.20	A-factor vs. Strain: PS-18 Vertical.....	148
6.21	Vertical/Lateral ($\bar{\sigma}_1/\bar{\sigma}_3$) vs. Strain: PS-18 Vertical.....	149
6.22	Lateral Strain vs. Vertical Strain: PS-18 Vertical.....	150
6.23	Poisson's Ratio vs. Strain: PS-18 Vertical.....	151
6.24	Hyperbolic Deviator Stress vs. Strain: PS-18 Vertical.....	152
6.25	Tangent Modulus vs. Deviator Stress: PS-18 Vertical.....	153
6.26	Deviator Stress vs. Strain: PS-18 Horizontal.....	154
6.27	Pore Pressure vs. Strain: PS-18 Horizontal.....	155
6.28	Stress Path (q vs. p): PS-18 Horizontal.....	156
6.29	A-factor vs. Strain: PS-18 Horizontal.....	157
6.30	Vertical/Lateral ($\bar{\sigma}_1/\bar{\sigma}_3$) vs. Strain: PS-18 Horizontal.....	158
6.31	Lateral Strain vs. Vertical Strain: PS-18 Horizontal.....	159
6.32	Poisson's Ratio vs. Strain: PS-18 Horizontal.....	160

LIST OF FIGURES (Cont'd)

<u>Figure Number</u>		<u>Page</u>
6.33	Hyperbolic Deviator Stress vs. Strain: PS-18 Horizontal.....	161
6.34	Tangent Modulus vs. Deviator Stress: PS-18 Horizontal.....	162
6.35	Deviator Stress vs. Strain: MA-02 GC-04 200 cm.....	163
6.36	Pore Pressure vs. Strain: MA-02 GC-04 200 cm.....	164
6.37	Stress Path (q vs. p): MA-02 GC-04 200 cm.....	165
6.38	A-factor vs. Strain: MA-02 GC-04 200 cm.....	166
6.39	Vertical/Lateral ($\bar{\sigma}_1/\bar{\sigma}_3$) vs. Strain: MA-02 GC-04 200 cm.....	167
6.40	Lateral Strain vs. Vertical Strain: MA-02 GC-04 200 cm.....	168
6.41	Poisson's Ratio vs. Strain: MA-02 GC-04 200 cm.....	169
6.42	Hyperbolic Deviator Stress vs. Strain: MA-02 GC-04 200 cm.....	170
6.43	Tangent Modulus vs. Deviator Stress: MA-02 GC-04 200 cm.....	171
6.44	Deviator Stress vs. Strain: MA-02 GC-04 100 cm.....	172
6.45	Pore Pressure vs. Strain: MA-02 GC-04 100 cm.....	173
6.46	Stress Path (q vs. p): MA-02 GC-04 100 cm.....	174
6.47	A-factor vs. Strain: MA-02 GC-04 100 cm.....	175
6.48	Vertical/Lateral ($\bar{\sigma}_1/\bar{\sigma}_3$) vs. Strain: MA-02 GC-04 100 cm.....	176

LIST OF FIGURES (Cont'd)

<u>Figure Number</u>		<u>Page</u>
6.49	Lateral Strain vs. Vertical Strain: MA-02 GC-04 100 cm.....	177
6.50	Poisson's Ratio vs. Strain: MA-02 GC-04 100 cm.....	178
6.51	Hyperbolic Deviator Stress vs. Strain: MA-02 GC-04 100 cm.....	179
6.52	Tangent Modulus vs. Deviator Stress: MA-02 GC-04 100 cm.....	180
6.53	Deviator Stress vs. Strain: V-32 PC-115 200 cm.....	181
6.54	Pore Pressure vs. Strain: V-32 PC-115 200 cm.....	182
6.55	Stress Path (q vs. p): V-32 PC-115 200 cm.....	183
6.56	A-factor vs. Strain: V-32 PC-115 200 cm.....	184
6.57	Vertical/Lateral ($\bar{\sigma}_1/\bar{\sigma}_3$) vs. Strain: V-32 PC-115 200 cm.....	185
6.58	Hyperbolic Deviator Stress vs. Strain: V-32 PC-115 200 cm.....	186
6.59	Tangent Modulus vs. Deviator Stress: V-32 PC-115 200 cm.....	187
6.60	Deviator Stress vs. Strain: V-32 PC-115 400 cm.....	188
6.61	Pore Pressure vs. Strain: V-32 PC-115 400 cm.....	189
6.62	Stress Path (q vs. p): V-32 PC-115 400 cm.....	190
6.63	A-factor vs. Strain: V-32 PC-115 400 cm.....	191
6.64	Vertical/Lateral ($\bar{\sigma}_1/\bar{\sigma}_3$) vs. Strain: V-32 PC-115 400 cm.....	192

LIST OF FIGURES (Cont'd)

<u>Figure Number</u>		<u>Page</u>
6.65	Hyperbolic Deviator Stress vs. Strain: V-32 PC-115 400 cm.....	193
6.66	Tangent Modulus vs. Deviator Stress: V-32 PC-115 400 cm.....	194
6.67	Deviator Stress vs. Strain: PI-18 Vertical.....	195
6.68	Pore Pressure vs. Strain: PI-18 Vertical.....	196
6.69	Stress Path (q vs. p): PI-18 Vertical.....	197
6.70	A-factor vs. Strain: PI-18 Vertical.....	198
6.71	Vertical/Lateral ($\bar{\sigma}_1/\bar{\sigma}_3$) vs. Strain: PI-18 Vertical.....	199
6.72	Hyperbolic Deviator Stress vs. Strain: PI-18 Vertical.....	200
6.73	Tangent Modulus vs. Deviator Stress: PI-18 Vertical.....	201
6.74	Deviator Stress vs. Strain: PI-2 Vertical.....	202
6.75	Pore Pressure vs. Strain: PI-2 Vertical.....	203
6.76	Stress Path (q vs. $\bar{p}$): PI-2 Vertical.....	204
6.77	A-factor vs. Strain: PI-2 Vertical.....	205
6.78	Vertical/Lateral ($\bar{\sigma}_1/\bar{\sigma}_3$) vs. Strain: PI-2 Vertical.....	206
6.79	Lateral Strain vs. Vertical Strain: PI-2 Vertical.....	207
6.80	Poisson's Ratio vs. Strain: PI-2 Vertical.....	208

LIST OF FIGURES (Cont'd)

<u>Figure Number</u>		<u>Page</u>
6.81	Hyperbolic Deviator Stress vs. Strain: PI-2 Vertical.....	209
6.82	Tangent Modulus vs. Deviator Stress: PI-2 Vertical.....	210
6.83	Deviator Stress vs. Strain: PI-2 Horizontal.....	211
6.84	Pore Pressure vs. Strain: PI-2 Horizontal.....	212
6.85	Stress Path (q vs. p): PI-2 Horizontal.....	213
6.86	A-factor vs. Strain: PI-2 Horizontal.....	214
6.87	Vertical/Lateral ($\bar{\sigma}_1/\bar{\sigma}_3$) vs. Strain: PI-2 Horizontal.....	215
6.88	Lateral Strain vs. Vertical Strain: PI-2 Horizontal.....	216
6.89	Poisson's Ratio vs. Strain: PI-2 Horizontal.....	217
6.90	Hyperbolic Deviator Stress vs. Strain: PI-2 Horizontal.....	218
6.91	Tangent Modulus vs. Deviator Stress: PI-2 Horizontal.....	219
6.92	Normalized Undrained Strength vs. Plasticity Index.....	220
6.93	Shear Strength vs. Depth: LL-44 GPC-3.....	221
6.94	Area vs. Strain: LL-44 GPC-3 1800 cm.....	222
6.95	Area vs. Strain: PS-18 Vertical.....	223
6.96	Area vs. Strain: PS-18 Horizontal.....	224
6.97	Area vs. Strain: MA-02 GC-04 200 cm..	225

LIST OF FIGURES (Cont'd)

<u>Figure Number</u>		<u>Page</u>
6.98	Deviator Stress vs. Strain: LL-44 GPC-3 1800 cm.....	226
6.99	Deviator Stress vs. Strain: PS-9 Vertical.....	227
6.100	Deviator Stress vs. Strain: PS-18 Vertical.....	228
6.101	Deviator Stress vs. Strain: PS-18 Horizontal.....	229
6.102	Deviator Stress vs. Strain: MA-02 GC-04 200 cm.....	230
6.103	Deviator Stress vs. Strain: MA-02 GC-04 100 cm.....	231
6.104	Deviator Stress vs. Strain: PI-2 Vertical.....	232
6.105	Deviator Stress vs. Strain: PI-2 Horizontal.....	233

LIST OF FIGURES FOR APPENDIX A

- A.1 Cell Manifold and Base
- A.2 Geonor Sample Stand
- A.3 Sample with Top Drainage
- A.4 Sample with Lateral Strain Gauge and Metal Tabs Attached
- A.5 Cell Pressure Line Attachment: Absolute Pressure Transducer
- A.6 Cell Pressure Line Attachment: Differential Pressure Transducer
- A.7 Triaxial Pressure System
- A.8 Flowchart

LIST OF SYMBOLS

- $\bar{a}$ = stress path intercept
- A_f = A-Factor
- B = Skempton's B parameter
- C = one-half disk thickness
- C_c = Compression Index
- C'_c = slope of water content vs. log consolidation pressure
- $\bar{c}$ = effective cohesion intercept
- c = undrained strength
- D' = distributed sample diameter
- D_m = mid-height sample diameter
- D_o = original sample diameter
- E = Young's modulus
- E_i = initial Young's modulus
- E_t = tangent Young's modulus
- E_u = undrained Young's modulus
- e = void ratio
- G_s = specific gravity of solids
- h = height of adjusting screw above disk
- I_L = liquidity index
- K = modulus number
- K_d = fractionation coefficient
- K = bulk modulus
- K_o = at-rest lateral earth pressure coefficient
- M = tangent modulus
- m = modulus number

ϵ_f = vertical strain of failure
 ϵ_L = lateral strain
 ϵ_r = radial strain
 ϵ_v = vertical strain
 ϵ_{vol} = volumetric strain
 $\epsilon_1^e, \epsilon_2^e, \epsilon_3^e$ = principal elastic strains
 ν = Poisson's ratio
 ν_u = undrained Poisson's ratio
 σ = stress
 σ_a = atmospheric pressure
 $\bar{\sigma}_c$ = consolidation stress
 $\bar{\sigma}_n$ = effective normal stress
 σ_o = original consolidation stress
 $\bar{\sigma}_{vo}$ = effective overburden pressure
 σ_3 = confining stress
 $\sigma_1, \sigma_2, \sigma_3$ = principal stresses
 $(\sigma_1 - \sigma_3)_f$ = deviator stress at failure
 $(\sigma - \sigma_3)_{ult}$ = ultimate dev. stress
 $\bar{\sigma}_1 / \bar{\sigma}_3$ = principal stress ratio
 $\bar{\phi}$ = effective friction angle

n = exponent number
 P = point load
 P_a = atmospheric pressure
 p = consolidation pressure
 $\bar{p} = 1/2(\bar{\sigma}_1 + \bar{\sigma}_3)$
 $\bar{p}_f = \bar{p}$ at failure
 $q = 1/2(\sigma_1 - \sigma_3)$
 $q_f = q$ at failure
 q_{fd} = drained strength at failure
 q_{fu} = undrained strength at failure
 R_f = deviator stress ratio
 S_u = undrained strength
 U_f = pore pressure at failure
 V_r = rod wave velocity
 V_s = shear wave velocity
 v^e = elastic volumetric strain
 W = width of disk
 w' = water content desired
 w_f = final water content
 w_i = initial water content
 w_o = original water content
 Z = depth
 $\bar{\alpha}$ = slope of q vs. $\bar{p}$
 δ = change in sample diameter
 ϵ = strain
 ϵ_a = axial strain
 ϵ_D = strain on top surface of disk

CHAPTER 1

INTRODUCTION

1.1 Program Objectives

The work presented in this report is part of the Sub-seabed Disposal Program (SDP) sponsored by the Department of Energy (DOE) through Sandia National Laboratories (SNL). The primary objective of this program (Anderson, 1979a), is to assess the technical, environmental, and engineering feasibility of disposing processed high level radioactive waste, in geologic formations under the world's oceans. The containment concept relies on the use of multiple barriers between the radioactive waste material and the biosphere. Man-made barriers utilized in this scheme include the waste form itself which would be solidified in a ceramic-like material and the canister used for containment and emplacement. The natural barriers include the sediments, the basement rock, the benthic boundary layer and the water column.

1.2 Site Selection

The desired geologic formation would provide a medium of containment which would isolate the waste for a sufficient period of time such that the decay rate of waste constituents is higher than the rate of nuclide migration through the containment medium. The primary site locations beneath the oceans have been characterized as mid-plate/mid-gyre (MPG) regions. A mid-plate location would isolate

the wastes from the edges of the tectonic plate, where seismic and volcanic activity is predominant. A mid-gyre location would place the wastes near the center of a circular water mass, this area having low biological activity.

The criteria for the selection of generic study sites are summarized below (from report, Seabed Disposal Program, Anderson, 1979b):

1) Tectonic stability: low volcanic and earthquake potential and no apparent faulting. The site should exhibit slow, continuous depositional processes.

2) Climatic stability: the site should be unresponsive to changes in climate such as ice ages.

3) Absence of resources: low biological activity (past and present) with no mineral resources of use to man.

4) Remoteness to Man: the site should be inaccessible to man and thus provide high security for the waste material.

5) Predictability: the ability to anticipate changes in properties with time.

These criteria place a tight restriction on the acceptable sites for the Seabed Program. The first criteria limits sites to the center of tectonic plates where seismic activity is minimal. Most of these sites also exhibit slow continuous deposition unaffected by climatic conditions. The third and fourth restrictions eliminate the continental shelves and slopes because of biologic resources, possible hydrocarbon content and man's ability to

work in these depths. The crustal rock beneath ocean basins have been tentatively eliminated from consideration because studies indicate considerable fracturing with water flow. Therefore attention is focussed on the sediment deposits. Predictability includes the ability to extrapolate the chemical and physical properties of the sediments in the horizontal and vertical direction from a few selective cores and measurements. The deep ocean basins being the least active in terms of biological and physical activity, represent the most predictable areas on earth. Areas such as the continental slopes where massive mud slides and turbidity currents occur, would be eliminated by the criteria of predictability.

1.3 Sediment Selection

The sediments, because of certain inherent properties, have been judged to be the primary barrier to radionuclide migration, with the waste form, canister, benthic boundary layer, and the water column acting as secondary barriers in the containment system. The sediment medium must possess the necessary properties to contain the waste for the specified period of time without being adversely effected by canister emplacement, high temperatures of the waste, and possible leaching of waste material after canister corrosion.

The most desirable characteristics of a sediment medium (Hollister, 1978) would include:

1) Low permeability and high K_d , where K_d is the fractionation coefficient, a measure of the ion exchange capacity (Heath, et. al., 1978).

2) Plastic properties, the ability of the sediments to close the projectile cavity after penetration.

3) Non-convective (or at least predictable) behavior under the likely thermal conditions.

4) Low organic matter, well oxidized.

These sediment criteria exclude certain sediment types and narrows down the areas where acceptable sediments can be found. The exclusions include (Hollister, 1978):

1) Coarse grained deposits such as proximal portions of the abyssal plains, all fracture zones and ponded turbidite sands. Also eliminated are all submarine canyons, continental shelves, shallow exposed mid-ocean ridges less than 2000 meters deep and the landward two-thirds of all abyssal plains.

2) Deposits of high organic carbon content.

3) Deposits exhibiting variable or high thermal conductivity.

4) Sediments less than 50 meters thick.

The primary marine sediments being considered are soft, fine-grained, plastic sediments, composed primarily of inorganic clay which has high adsorption and permeability characteristics capable of chemically and physically containing the waste for the necessary time periods.

To a large extent, the abyssal hills have the characteristics necessary to be of primary interest in the Sub-seabed Disposal Program (Anderson, 1979b). These regimes exhibit low tectonic activity due to their distance from the plate edges and the characteristics discussed earlier are typical of abyssal hill areas of the world's oceans. The predominant sediment type is the red clay, which appears to be suitable for waste containment.

1.4 Hole Closure Program

Part of the work at the University of Rhode Island Marine Geomechanics Laboratory (URI/MGL) has involved the study of geotechnical properties of marine sediments from possible MPG disposal sites. Physical property data for several types of sediment is being compiled into an extensive data base. This data base supports several project groups within the Program. One project group is investigating methods of waste emplacement into the seabed and the effect of techniques on sediment response. Several possible emplacement concepts are being considered, ranging from, a free-fall penetrometer, a winch controlled emplacement, to drilled-hole emplacement. A testing program was initiated to determine the sediment response for the free-fall and slow penetration methods.

Small scale projectile penetration experiments (referred to in this report as Hole Closure Experiments) were undertaken to scope out the problems and provide qualitative and quantitative information for the computer model

penetration studies. The studies include the dynamic response of the sediment to projectile intrusion (loading) and behavior in the vicinity of the projectile cavity subsequent to quasi-static penetration. The Hole Closure Program has been described in detail in the URI/MGL Annual Progress Report No. 4 (Silva and Calnan, 1978). Static and dynamic projectile penetration tests were conducted on remolded samples of illite and smectite. Sediment tanks (see Section 2.8) were instrumented to record shock waves, pore pressures, and sediment acceleration and displacement during projectile emplacement. The projectiles were instrumented with accelerometers during the dynamic tests or load cells during static penetration tests. Cavity closure was also measured after static emplacement tests. After the completion of penetration tests in each tank, triaxial and consolidation samples were taken from the re-consolidated (see Section 2.8) sediment. Hereafter, the term remolded will be used to identify the samples taken from the Hole Closure Experiment tanks. The term undisturbed will be used to identify the samples which simulate as close as possible the behavior and properties of in-situ sediments. These samples have been taken from the various cores discussed in Section 2.4, 2.5, and 2.6.

1.5 Related Subseabed Programs

Due to the extremely slow natural processes occurring in the potential repository sites and the nature of the processes occurring after canister emplacement, these

processes will be simulated by computer models and extrapolated to long periods of time. Short term in-situ and laboratory experiments, such as the Hole Closure Experiment, are being conducted to validate the predictive models and to acquire new information for inclusion in the models.

Several computer models have been adapted and modified by personnel at Sandia Labs. Two models currently being used in the penetration studies, are the HONDO and TOODY codes. The HONDO code (Key, 1974) is a finite element model designed to compute the time-dependent displacements, velocities, accelerations, and stresses within elastic or inelastic, two dimensional or axisymmetric bodies of arbitrary shapes and materials.

The TOODY code (Chavez, et. al., 1979) is a two dimensional, explicit, finite difference code with large strain capabilities. The material parameters which must be entered into the codes include: Young's Modulus, Poisson's Ratio, a yield strength, and a hardening modulus. These parameters will be supplied by URI/MGL from the stress-strain-time test program. Other codes are under development.

The Waterways Experiment Station (Corps of Engineers, Vicksburg, Miss.) will conduct dynamic one dimensional compression and triaxial tests on samples of illite and smectite. WES is also working with Sandia Labs to develop constitutive equations for some of the modeling work.

The In-situ Heat Transfer Experiment (ISHTTE) is a

joint program involving personnel at URI/MGL, SNL, and the Univ. of Washington, Applied Physics Laboratory. In this experiment, an instrument package will be placed on the sediment surface for one year to monitor heat flow data from a simulated waste canister imbedded in the sediment. Cores will be taken at the site to establish the effect of high temperatures on sediment properties.

Another related activity in the SDP is the Long Coring Facility (LCF) Program. The goal of this project is to develop a corer to obtain a continuous fifty meter undisturbed sample of marine sediment in the deep sea. Knowledge of sediment properties and behavior is essential to the penetration analysis and core design.

One of the activities at URI/MGL is the study of stress-strain-time behavior of undisturbed and remolded samples of fine grained sediments. This study has been further broken down into short (quasi-static) and long term (creep) stress-strain behavior. The stress-strain-time data will be used primarily in four areas of study: computer modeling of the Hole Closure Experiment, modeling of the Long Coring Facility Program, modeling of the canister migration, and in design of the ISHTE experiment. In the first phase of creep testing, drained, constant stress, creep tests are being run at three different percentage values of the failure deviator stress, as determined in the quasi-static triaxial program. The second phase of testing, will be conducted under conditions of high tempera-

ture and pressure (up to 200°C and 70.3 kg/cm^2). The data generated in these tests is being used to model the projectile cavity closure and possible canister migration during the heating and cooling cycle.

1.6 URI/MGL Quasi-static Triaxial Program and Objectives

The principal goal of the URI/MGL program is to provide insight into material properties and behavior of marine sediments. This data can then be applied to several special problems being studied by the Subseabed Disposal Program. The objective of this investigation was to study the stress-strain properties of remolded and undisturbed samples of marine sediments (see Section 2). For this test program, a total of 37 CIU (consolidated isotropically undrained) triaxial tests were performed on both undisturbed and remolded samples of North Pacific illite and smectite. The goals of the testing program were to:

- 1) Provide Mohr-Coulomb parameters for two deep-sea sedimentary materials, illite and smectite.
- 2) Compare the properties of undisturbed and remolded samples.
- 3) Compare the properties of illite and smectite.
- 4) Compare the properties of remolded samples taken in vertical and horizontal orientations.
- 5) Determine values of Young's modulus (E) for the two materials using a hyperbolic stress-strain model.
- 6) Measure lateral strain and determine values of Poisson's ratio for the sediments.

To accomplish these goals, a testing schedule was planned to derive the maximum information from the least number of samples since the number of undisturbed samples was very limited. Figure 1.1 is a flow-chart of the testing schedule detailing the paths of comparison between the samples. The primary comparison in the test schedule is between the remolded (reconstituted-reconsolidated) and undisturbed samples.

To differentiate between properties of illite and smectite, three remolded sample series of each material were tested and compared to the undisturbed core samples from MARA-02 (illite), VEMA-32 (illite) and LL-44 (smectite) (see Figure 1.1). Also, two remolded sample series, in vertical and horizontal orientations, were tested for each material to determine the degree of sample anisotropy.

Lateral strains were measured on most of the samples at mid-height, using a specially designed version of the "Karl Schuler" disk-ring gauge (Schuler, 1978). From these measurements, values of Poisson's ratio were calculated assuming a deformational shape (see Section 4.5).

Values of initial and tangent Young's modulus determined using a hyperbolic stress-strain theory have also been reported. A detailed presentation of the hyperbolic theory appears in Section 5.

From the analysis of the test data, an attempt has been made to produce a qualitative and quantitative description of the stress-strain properties of the sediment-

ary material. An attempt has been made to ascertain whether or not remolded sediment properties can be used to predict or model the behavior of undisturbed marine sediments. Using Mohr-Coulomb theory and effective stress principles the basic soil parameters of effective cohesion ($\bar{c}$) and effective angle of friction ($\bar{\phi}$) have been determined. Hyperbolic stress-strain parameters based on Mohr-Coulomb theory have also been used to model the stress-strain behavior of the sediments. As part of the analysis, the elastic parameters, Young's Modulus and Poisson's Ratio have been determined from experimental data and used in the comparison of remolded and undisturbed samples.

Figure 4.1 - Generalized Stress-Strain

Figure 1.1 Triaxial Testing Program

CHAPTER 2

SAMPLING AND TESTING PROGRAM

2.1 Introduction

The undisturbed samples used in this testing program were from cores taken in MPG-1, a site in the middle of the central North Pacific approximately 600 miles north of Hawaii. Background information on the MPG-1 study site and core data are presented in this Chapter. Also included is a description of the Hole Closure Experiment testing procedures.

2.2 Study Site MPG-1, North Central Pacific

The MPG-1 study site encompassing an area of approximately 40,000 square kilometers is shown in Figure 2.1. This site has been selected as a reference system for study purposes (Anderson, 1979a). Although the site possesses most of the required characteristics discussed in Section 1.2 and 1.3, the sediment thickness in MPG-1 is considered too thin to be used as an actual disposal site. The region has low abyssal hills averaging a few hundred meters in relief (Hollister, 1978), with sediment thicknesses of 20-50 meters (see Figure 2.2) and water depths greater than 5000 meters. The sedimentation rates are very low, 2.5 mm/1000 yrs. for the last two million years and approximately 0.2-0.3 mm/1000 yrs. from 5-65 million years ago (Doyle and Riedel, 1979). The near-bottom currents have low velocities (1-3 cm/sec mean flow) and the

gradients of temperature and salinity are small (Hollister, 1978).

2.3 Undisturbed Samples

All the undisturbed triaxial samples were taken from cores using a URI/MGL designed undisturbed sampling device. A stainless steel sampling tube is placed onto a fixed piston which is positioned at the surface of the sediment (usually against a freshly trimmed section, perpendicular to the core axis). The tube is then pushed into the core with one continuous motion using a lever system. This method provides for excellent sample quality and has been designed to accommodate sample diameters of 3.57, 5.08 and 6.35 centimeters. After the sampling ring has been pushed into the core, it is carefully cut out, capped, labeled, and sealed with wax. The undisturbed samples are stored in seawater at 4 degrees centigrade until tested in the lab. Geotechnical tests performed directly on the core include vane shear and acoustic velocimetry. Other sampling includes water content-bulk density samples and bulk samples for classification, grain size, and specific gravity.

2.4 LONG LINES-44 Cruise

The AT & T cable laying ship C.S. LONG LINES was specially outfitted for handling the Giant Piston Coring (GPC) system and a coring expedition was held in October, 1976. During the cruise, a core (GPC-3) of 24.4 meters

length (undisturbed) was obtained. The device used to obtain the 24.4 meters of undisturbed sediment was a Giant piston Corer (Silva and Hollister, 1973; Silva, et. al., 1976; Silva and Hollister, 1979). The large core diameter of 11.4 cm provides a high quality sediment sample with little or no shear deformation along the walls. Since there is no liner, the sediment is extruded onboard ship and processed. This procedure greatly reduces disturbance to the sediment which normally occurs during handling and shipping. Extensive processing of GPC-3 was performed on ship including undisturbed sampling, vane shear, velocimetry measurements and many other measurements for other scientific studies. Figure 2.3 gives the location of GPC-3 and other cores taken in MPG-1.

The profile of water content (corrected for 35 ppt salt content) versus depth in GPC-3 is shown in Figure 2.4. The water content in the upper 6 meters is nearly constant (110%) with no major fluctuations, this depth being composed primarily of illite rich sediments. A dramatic increase in water content between 6 and 9 meters depth (to a value of 180% at 9 meters) corresponds to a change from illite to smectite-rich sediments. Between 9 and 15 meters, there is a gradual increase in water content with depth (to a maximum of 254%) but with a greater number of fluctuations than in the upper core. This variability is attributed to the presence of ash layers (see Figure 2.5). Below 15 meters, a gradual decrease in water content is

observed with a value of 160% at 24.4 meters. In this deeper section of core, extreme changes in water content take place over small changes in depth (e.g. at 20.0 to 20.5 meters, a drop of approximately 100%). The sediment below 24.4 meters to the bottom of the core was flow-in (which is sediment that is sucked into the bottom of the core when the piston is retracted up to the core weight stand during pullout).

The plot of undrained strength (vane shear strength) versus depth in core (Figure 2.4) shows a general increase in strength with depth. Vane shear measurements were made with a Wykeham-Farrance motorized miniature vane with a vane size of 1.27 x 1.27 centimeters, rotated at 24 degrees per minute. Variations at approximately 11 meters can be attributed to the volcanic ash layers at that depth, while extreme variations in strength between 22 meters and the bottom of the undisturbed sediment can be attributed to layering in that depth range. Section 6.4 presents two methods developed to predict undrained strength versus depth in GPC-3. The results are then compared to the measured vane strengths.

From the visual description in Figure 2.5, it can be observed that GPC-3 contained altered ash layers, one at 10 meters having a hard manganese coating (Hollister, 1978). This layer correlated well with an acoustic reflector observed in the 3.5 kHz records. The sediments at the bottom of the undisturbed core were found to be 65

million years old as dated by fish debris (Doyle and Riedel, 1979).

2.5 VEMA-32 Cruise

During August of 1975, the R/V VEMA took several piston cores in the MPG-1 study site. The cores were standard piston cores having an inside diameter of 6.4 cm, which results in a more disturbed sediment core than the larger diameter Giant Piston Core. Only PC-115 was sampled for undisturbed triaxial and consolidation samples and no vane shear measurements were obtained.

As indicated in Figure 2.6, the water content profile for PC-115 is very similar to the upper 10 meters of LL-44 GPC-3. The water content is approximately constant at 110% to a depth of 5 meters. Below this depth, there is an increase in water content to a value of 234% at 9.7 meters. The same changes in mineralogy are evident in this core, with the upper few meters being illite followed by a transition from illite to smectite-rich sediments.

2.6 MARA-02 Cruise

Five (5) gravity cores were obtained during a cruise in August of 1978 on the R/V WASHINGTON. Three (3) meter lengths of PVC pipe (inside diameter of 10.2 cm) were used as core barrels. This allows the sediment to be stored and if necessary transported in the pipe, thus eliminating the need for immediate extrusion and processing of the cores. These cores were cut in half, sealed, carefully

crated as to minimize disturbance, and shipped to URI for processing. When opened, the cores appeared in good condition with no apparent free standing water. As with previously described piston cores, the general suite of geotechnical tests were performed on all cores.

Undisturbed triaxial samples used in this testing program were taken from MA-02, GC-04. Figure 2.7 is a plot of water content and vane shear strength versus depth in the core. Figure 2.8 is a plot of water content versus depth of the upper few meters for the three cores used in this study. The three cores exhibit nearly identical water contents within this zone. The gravity core did not penetrate the transition zone observed in the piston cores and therefore consists entirely of illite sediments. Two distinct anomalies are observed in the shear strength plot of this core (MA-02 GC-04), one at 105 centimeters and the second at 160 centimeters (Figure 2.7). The water content, grain size and Atterberg limit data in the depth range of 95-105 cms is similar to the upper and lower core data. The specific gravity increases dramatically to 2.97 as compared to approximately 2.7 above and below this depth. The lower shear strength is thus a result of the higher void ratio in this layer (95-105 cm). A change in both the water content and shear strength is observed at 160 cm. The core in this depth range was visually homogeneous with similar values of physical property data (grain size, Atterberg limit and specific gravity) as the upper and lower

core. The increase in shear strength is the result of the decrease in water content at this depth.

2.7 Comparisons of MPG-1 Cores

Sediment thickness in the MPG-1 study site varies between 20 and 50 meters in depth. Two sub-bottom acoustic reflectors are observed in the 3.5 kHz records which were found to correlate with the volcanic ash layers at 10 and 20 meters in GPC-3 (Hollister, 1978). The top 4-5 meters of sediment is predominantly illite, a common clay mineral having a structure similar to that of muscovite mica but with less potassium (Mitchell, 1976). Illite has a moderate potential for absorbing ions (Heath, 1978), less in comparison to smectite due to the non-exchangable potassium ions which balance the charge deficiency (Mitchell, 1976). In addition to illite, the following minerals were found between 0 and 10 meters, quartz, smectite, chlorite, cristobalite, kaolinite, feldspar, and mica (Hollister, 1978).

Sediments between 10 and 24.4 meters are predominantly iron-rich smectite (Heath, 1978). Smectite clays are characterized as being plate shaped, extremely fine grained, very plastic, with high potential for sorbing ions (Heath, 1978). Mitchell (1976) reports that smectite minerals exhibit high cation exchange capacity due to the large amount of unbalanced substitution within the lattice. Other minerals found between 10 and 24.4 meters include smectite, phillipsite, feldspar, clinopillotite, and small

amounts of quartz (Hollister, 1978).

The change from smectite to illite occurred due to the movement of the MPG-1 region through two different types of sedimentation regimes (Hollister, 1978). From the several piston cores taken in the MPG-1 study area, the sedimentation rate for the last 30 to 40 million years was found to be constant, despite traumatic climate changes, such as the ice age (Anderson, 1979b). This finding has been substantiated by several dating techniques.

Lateral coherency, based on geotechnical data alone, can be observed in all the cores (e.g. see Figure 2.8). It should be noted that there is a consistent relationship between sediment type, water content and liquid limit. For the illite rich sediments, the liquid limit is always less than the natural water content. For the transitional material the liquid limits are approximately equal to the natural water content, while the liquid limits for the smectite are always greater than the natural water content (see Figures 2.4, 2.6, and 2.7). At the lower depths, the smectite layer has undergone consolidation due to the overburden, resulting in decreasing water contents with depth below approximately 15 meters.

2.8 Hole Closure Program

As part of the LL-44 cruise, a dredge was lowered to the bottom to obtain large volumes (one-third cubic meter) of illite material, which was placed in 55 gallon drums and transported to URI. The sediment was sieved to remove

all particles larger than a #20 sieve, since there were large numbers of manganese nodules present in the surface sediments. The illite material was used in the Hole Closure Program discussed earlier in this report.

A somewhat smaller volume of smectite material was obtained when the core tripping mechanism of GPC-2 failed and resulted in the core being filled with flow-in material of essentially smectite. This smectite was also used in the Hole Closure Program. It should be emphasized that these bulk samples were completely remolded at the time of sampling.

The Hole Closure Program has been described in detail in the URI/MGL Annual Progress Report No. 4 (Silva and Calnan, 1978) and will only be briefly reviewed here. As discussed in Section 1, static and dynamic projectile penetration tests were conducted on remolded samples of illite and smectite. Tanks 45.7 cm in diameter were used to consolidate the remolded material to simulate predetermined in-situ depths. Before the material was placed in the tanks, it was homogenized in a large clay mixer at a water content high enough to provide a workable consistency. After reconstitution, the material was placed in the tanks in a manner which minimized air entrapment within the sediment. As the penetration tests were completed, tri-axial and consolidation samples were taken at various levels within each tank. Vane shear and water content tests were also conducted as part of the analysis.

2.9 Data Presentation

Each sample series is identified by either the cruise number, core number, and depth (e.g. LL-44 GPC-3 1800 cm) or by the type of material, simulated depth in meters and orientation (e.g. PS-9 Vert.), which identifies the sample as a Pacific smectite with a 9 meter simulated depth and vertical orientation. Each series is represented by three or four samples tested at different consolidation stresses ($\bar{\sigma}_c$), such that the shear strength envelope for that series can be determined. For most test series, one or two samples were consolidated to stresses less than the preconsolidation stress, resulting in an overconsolidated sample. Data presentation will be explained further in Section 6.

FIGURE 5.1. HAWAIIAN ISLANDS LOCATION MAP

Figure 2.1 Mid-Plate Gyre (MPG)-1 Study Area

Figure 2.2 MID-PLATE-GYRE REGION I
 SEDIMENT THICKNESS:
 3.5 KHz PENETRATION TO DEEPEST OBSERVED REFLECTOR

CORE LOCATIONS IN MPG-1
FIGURE 2.3

LL-44 GPC-3

Figure 2.4 Water Content and Shear Strength Versus Depth:
LL-44 GPC-3

GPC-3 VISUAL DESCRIPTION

Figure 2.5

VEMA-32 PC-115

Figure 2.6 Water Content Versus Depth: V-32 PC-115

MA-02 GC-04

Figure 2.7 Water Content and Shear Strength Versus Depth:
MA-02 GC-04

LL-44 GPC-3

HEMA-32 PC-115

MA-02 GC-04

Figure 2.8 MPG-1 Core Comparison

CHAPTER 3

TRIAXIAL EQUIPMENT AND PROCEDURES

3.1 Basic Procedures

The basic procedures used in this testing program were very similar to those proposed by Bishop and Henkel (1962). A few changes have been made due to technological improvements in equipment and instrumentation, such as the replacement of dial gauges with DCDT's and pressure gauges with pressure transducers. To conduct a CIU triaxial test, the sample must first be prepared, secondly it is consolidated and back pressured under a predetermined hydrostatic confining stress and finally tested under the desired loading rate.

To aid in subsequent triaxial testing using URI/MGL equipment, a user's manual has been prepared and is presented in Appendix A. This manual gives detailed descriptions on all test procedures as well as instructions in using the HP data acquisition system to monitor and output test data. The programs used with the HP system are also presented in Appendix A. A brief summary of the equipment and procedures is presented in this chapter.

3.2 Equipment

An air pressure system was substituted for the usual mercury pot system to obtain the necessary confining and back pressures. The maximum pressures are limited to approximately 6.3 kg/cm^2 (90 psi) due to the compressor sys-

tem. The air is controlled by two sensitive Anteus back pressure regulators (range: 0 to 17.6 kg/cm²) in each of three pressure cabinets. Figure 3.1 is a schematic of the pressure system indicating the position and function of each valve and fitting. The left regulator (5 on Figure 3.1) controls the consolidation or confining stress while the right regulator (6 on Figure 3.1) controls the pressure applied directly to the sample. In the back pressure mode, an increase in pressure from the back pressure regulator will result in an equal increase in pressure in the cell pressure lines. Confining pressures less than 0.14 kg/cm² may be generated with a head differential between two reservoirs, using the back pressure regulator to control both sides of the system to avoid fluctuations in confining stress.

The triaxial cells employed in the test program are manufactured by Wykeham-Farrance (see Figure 3.3). The piston seal relies on the fine machine fit between the piston and housing to prevent extreme leakage of cell fluid. A manifold (see Figure 3.2) has been provided to the cell to increase the possible variations of drainage and pressure application ports. All lines attached to the cell and used in the pressure cabinet are either stainless steel or nylon tubing.

Pore pressures were measured using strain gauge type pressure transducers. The particular gauge selected for use in a test was dependent upon the consolidation pres-

sure applied to the sample and the working range of the transducer. For all tests with consolidation pressures less than 0.14 kg/cm^2 , a 0.21 kg/cm^2 differential transducer was selected. For confining stresses greater than 0.42 kg/cm^2 an absolute transducer was selected, while intermediate stresses required either an absolute or differential transducer.

Displacements of the proving ring and displacements of the cell relative to the proving ring were made using HP DCDT's. These transducers are similar to LVDT's, but do not require signal conditioning instrumentation.

3.3 Sample Set-up and Test Procedures

For setting the sample up in the cell, a Geonor sample stand was adapted to the Wykeham-Farrance cells to minimize sample disturbance. This stand provides a secure platform from which the rubber membranes and o-rings may be applied to the sample. Water content specimens were trimmed from both ends of the sample and compared to the water content data in the core profile. If possible, all samples were trimmed to a length of three inches, providing a length to diameter ratio of two to one. The sample weight was also obtained to determine the bulk density of the sample.

Quarter inch strips of Whatman's No. 54 filter paper were applied to the sides of the sample to accelerate consolidation and pore pressure equalization and porous stones were used at both ends of the sample. Two rubber membranes

with an interlayer of silicone stopcock grease were used to isolate the sample from the confining fluid. Three o-rings at top and bottom insured an adequate seal of membrane to pedestal and top cap. Initially salt water was used as a confining fluid to reduce osmotic reactions between the sample and the cell fluid. However, problems encountered with the lateral strain gauges (see Chapter 4) necessitated the substitution of silicon oil.

After the sample set-up procedures were completed, the sample was consolidated isotropically to the desired stress. Volume change versus time measurements were taken to determine the time for 90 percent consolidation. After consolidation, a back pressure of 2.81 kg/cm^2 was applied to the sample in increments of 0.70 kg/cm^2 . If a response of 95 percent was not obtained in two minutes an additional 0.70 kg/cm^2 back pressure was applied. Skempton's B parameter (Skempton, 1954) is considered equal to unity for a 95% response in two minutes. This parameter is an indicator of the degree of saturation of the sample, B equals unity for a 100 percent saturated sample.

For the duration of the test program, a deformation rate of 0.030 millimeters per minute was selected, resulting in 20 percent strain in approximately 8 hours for the three inch samples. It should be noted that the strain rate is not constant during the initial stages of compression due to the deformation of the proving ring used to measure axial load. As the change in axial load decreases,

the strain rate will approach a constant. Measurements of vertical deformation, load, pore pressure and lateral deformation were recorded and processed with the HP data acquisition system. This system was used to calculate test results, store data and plot results.

- (1) Main Panel Air Shut Off Valve
- (2) Cell Pressure Air Supply Regulating Valve
- (3) Back Pressure Air Supply Regulating Valve
- (4) Cross-over Valve
- (5) Cell Pressure Regulator
- (6) Back Pressure Regulator
- (7) Regulating Valve to Cell Pressure Quick-Disconnect
- (8) Regulating Valve to Back Pressure Quick-Disconnect
- (9)&(10) Quick-Disconnect
- (11) Cell Pressure Regulating Valve
- (12) Back Pressure Regulating Valve
- (13) Switching Valve
- (14) Burrettes
- (15) Cell Pressure Reservoir
- (16) Back Pressure Reservoir

Figure 3.1 Triaxial Pressure System

Figure 3.2
Cell Manifold and Base

Figure 3.3 Wykeham-Farrance Triaxial Cell

CHAPTER 4

LATERAL STRAIN AND POISSON'S RATIO

4.1 Introduction

There are four general techniques used to measure lateral strain as follows:

1) Volume change: gives an average value of lateral strain for entire sample. Requires a high conformance system, a small pressure vessel, and an accurate method to measure change in volume.

2) Circumference: measure circumference with a wire wrapped around sample. The change in circumference is determined by measuring the displacement of the end of the wire.

3) Localized gauges: strain gauges applied directly to sample or sample jacket. Both methods have inherent problems and no pre-calibration of the gauges is possible.

4) Diameter gauges: gauge applied to the sample to measure change in sample diameter. Usually some type of metal or plastic caliber upon which strain gauges or LVDT's have been placed.

The gauge of Bishop and Henkel (1962) is the first reported sensor used in soils work to monitor lateral strains of a sample. This gauge, which used a mercury vernier to measure lateral strain, was a manual system requiring constant monitoring by a technician. It was primarily used to conduct K_0 tests, in which the lateral strains were kept equal to zero by increasing the cell pressure.

Measurements of lateral strain have been made with a cantilever beam type sensor. This sensor uses spring steel arms as the cantilever beam with adjusting screws which contact the sample at the point of lateral strain determination. The cantilever is supported by a rigid ring, usually having a large mass. Strain gauges are placed on the cantilever close to the supporting ring. Exact machining tolerances are required to get matching deflection to output response for each cantilever. The large mass used to support the cantilever arms does not allow the gauge to be suspended on the sample. Due to the geometry of the gauge itself, only a small linear output range is possible.

Gauges similar to that of Bishop and Henkel's have been used by Menzies (1976) and Brown and Snaith (1974) to automatically measure lateral strains of the samples. With both gauges, the authors report using a circular caliber which surrounds the sample, upon which an LVDT has been placed to measure the expansion of the caliber. A band type lateral strain gauge has been used by El-Ruwayih (1976) and Boyce and Brown (1976) to measure volume changes and direct measurements of strain on the sample. These gauges consist of metal or acrylic bands which are formed into shapes capable of maintaining contact with the sample during deformation. Foil strain gauges are placed on the bands to measure the strain in the bands and thus the strain of the sample. All of these gauges are operated in some type of non-conducting oil. The majority have been

designed to allow more than one gauge to be placed on the sample at a time.

4.2 Disk-Ring Gauge

As part of the experimental program at Sandia Labs, a lateral deformation gauge was designed by Karl Schuler (1978) for use on rock specimens. The gauge measured the change in diameter of rock samples under uniaxial compression to determine the onset and extent of dilatancy. The gauge also served as a control parameter to obtain the entire stress-strain curve of the rock sample.

The required gauge characteristics (Schuler, 1978) include: a large linear range, to monitor lateral strains as high as 30 percent; a small size so that the pressure vessel diameter could be minimized; the ability to measure absolute specimen diameters; and a size which would allow more than one gauge to monitor one sample diameter. The final design sought to exclude the inherent problems of strain measurement techniques discussed in Section 4.1.

The gauge as constructed is illustrated in Figure 4.1. The gauge consists of a thin metal disk which fits around the sample. Two posts diametrically opposite and perpendicular to the disk each have an adjusting screw at equal heights above the disk. These adjusting screws contact the sides of the sample and as lateral deformation occurs, the sensing disk is flexed. Foil strain gauges placed on the top and bottom surfaces of the disk, midway between the posts, are wired in a full bridge configuration. De-

formation of the sensing disk produces positive strains in the foil strain gauges on the top surface of the disk and negative strains in the bottom foil gauges, resulting in a wide-ranged linear output.

As explained by Schuler (1978), from the geometry of the gauge and the manner of loading, the disk is deformed into a cylindrical surface. The deformed shape can then be considered as a beam in pure bending and analyzed as such. Choosing an optimum adjusting screw length results in a final equation for the analysis:

$$\delta = 3/4 \frac{h D_o}{C} \epsilon_D \quad (4-1)$$

δ = change in sample diameter

D_o = original diameter of sample

C = one-half the disk thickness

h = height of adjusting screw above disk

ϵ_D = strain on top surface of disk

By changing the height of the adjusting screws above the disk, the sensitivity of the gauge and its linear range can be changed.

Point loads were determined from the pure loading theory used to determine the strains. The force exerted by the adjusting screw tips on the sample is:

$$P = 4/9 \frac{WC^3 E}{D_o h^2} \quad (4-2)$$

where W is $1/2$ (OD-ID) and P is the point load.

The final design for rock resulted in a gauge with a large linear output unaffected by the high confining pressures used in rock mechanics (a pressure of 200 mPa resulted in a 0.5% change in calibration). Also, the design allowed two gauges to be positioned on the sample to measure two diameter changes at the mid-height of the sample.

4.3 URI/MGL Lateral Strain Gauge

As outlined in Section 1.6, one of the goals of the test program was to determine values of Poisson's ratio for the two sedimentary materials from lateral deformation measurements. It was decided to modify the gauge designed by Schuler. A major consideration is working with the soft marine sediments was the point loads which would be imposed upon the samples by the adjusting screws. In preference to other methods, two small metal tabs were chosen to distribute the point loads of the adjusting screws. A bearing capacity analysis was performed using an undrained strength of 25 gm/cm^2 , assuming that the membranes and filter strips do not support the metal tab. Using the maximum allowable load determined in the bearing capacity analysis and Equation 4-2, the factors of C, W, and h were varied to produce a gauge with an acceptable point load and a workable design.

In the final design of the URI lateral strain gauge (see Figure 4.2) the adjusting screws were positioned 2.54 centimeters above the disk, the disk itself being 0.64 cm (1/4 inches) wide and 0.25 mm (0.010 inches) thick. The

metal disk was machined from spring steel stock (Figure 4.3). The fabrication jig (Figure 4.3) allowed three to four disks to be machined at once. After machining, the disks were heat treated to a spring temper and hardened. This procedure produced a flat disk and improved the elastic quality of the metal. Four foil strain gauges (BLH gauges: FAE-25-50) of equal resistance were epoxied onto the disk after proper surface treatment using procedures outlined in the Micro-Measurement Catalog (1979). The gauges were wired in a full bridge configuration with leads long enough to feed out through the cell pressure port of the triaxial cell. The leads were epoxied into a feedthru connector to seal the cell.

The gauges were calibrated by separating the points of the adjusting screws with a vernier caliber and recording the output and displacement. A least squares line was fit to the data points to calculate a calibration factor for the gauges. The plots of displacement versus output were linear for the entire range of predicted sample deformation.

In placing the gauge on a sample, the entire sample set-up procedure (see Chapter 3) was first completed. The metal tabs were epoxied onto the rubber membrane at the mid-height of the sample using Eastman 910, a quick setting epoxy. The gauge was positioned on the sample and deflected (using the adjusting screws) enough to compensate for predicted changes in volume during consolidation

(Figure 4.4). The initial output of the gauge was recorded before and after consolidation. During uniaxial compression, the output of the gauge was recorded as one of the four channels of data (see Appendix A).

In the initial phase of testing, only one major problem was encountered, the inability to protect the gauge from the salt water used as a confining fluid. Several different types of coatings were tried unsuccessfully. Under pressure the water would permeate the coatings. Finally, a nonconductive silicon oil was used as a cell fluid. No further problems were encountered after switching to the silicon oil. No problems were encountered with the metal tabs punching into the sample, even on the soft PI-2 series samples. The distribution of the load by the tabs and the support of the membranes is believed to be the reason for the success of this procedure.

4.4 Poisson's Ratio

The analysis method employed in this report assumed Poisson's ratio was equal to the ratio of the two principal strains multiplied by a correction factor to account for the deformational shape of the sample. A complete presentation of the analysis method appears in Section 4.5. Alternative methods of determining Poisson's ratio and a review of pertinent literature are discussed here.

For a triaxial compression test with stress increments: $d\bar{\sigma}_1 > 0$, $d\bar{\sigma}_2 = d\bar{\sigma}_3 = 0$, the following elastic strains result:

$$\begin{aligned}\epsilon_1^e &= \epsilon & \epsilon_3^e &= -v\epsilon \\ \epsilon_2^e &= -v\epsilon & v^e &= (1-2v)\epsilon\end{aligned}\quad (4-3)$$

where ϵ_1^e , ϵ_2^e and ϵ_3^e are the principal elastic strains and v^e is the elastic volumetric strain. For an undrained test (change in volume = 0):

$$-\epsilon_3^e / \epsilon_1^e = 1/2 \quad (4-4)$$

If the two principal strains are measured, the values of Poisson's ratio can be calculated.

By measuring volumetric change during a drained tri-axial test in addition to axial strain, Poisson's ratio can be calculated. Defining radial strain as:

$$\epsilon_r = 1/2 (\epsilon_{vol} - \epsilon_a) \quad (4-5)$$

where ϵ_{vol} equals the volumetric strain and ϵ_a the axial strain, Poisson's ratio is calculated from the formula:

$$v = -\epsilon_r / \epsilon_a \quad (4-6)$$

This method appears to be the most commonly used, having been reported in papers by Wong and Duncan (1974), Hardin (1978), and Lambe and Whitman (1969). Drained tests performed on a well graded calcareous sand from Libya (Lambe and Whitman, 1969) exhibited Poisson's ratios greater than 0.65. Lambe and Whitman (1969) state, "Poisson's ratio is less than 0.5 only during the early stages of such a test". Elastic theory and its associated parameters of Young's modulus and Poisson's ratio is only applicable in the lin-

ear range of the stress-strain curve. Wong and Duncan (1974) determined Poisson's ratio from drained tests on a silty sandy gravel. They found Poisson's ratio to increase with increasing vertical strain and decrease with increasing consolidation pressure. Lambe and Whitman (1969) also report that values of Poisson's ratio vary with axial strain. Krizek (1977) tested eight samples of different oriented soil fabric and found Poisson's ratio to be strongly strain dependent but apparently independent of soil fabric. Kirkpatrick and Belshaw (1968) present data on axial and radial strains measured on sand samples using X-ray techniques. In four samples of different porosities, Poisson's ratios in excess of 0.5 were measured. Poisson's ratios greater than 0.5 indicate the soil is dilatant, i.e. increasing volume during shear. Wong and Mitchell (1975) using drained triaxial and plane strain tests on sensitive Canadian clay found values of Poisson's ratio between 0.1 and 0.15 for the first few readings of deviator stress.

Poisson's ratio can be determined from velocity measurements of rod and shear waves using the formula (Hardin, 1978):

$$\frac{v_r}{v_s}^2 = 2(1+\nu) \quad (4-7)$$

According to Hardin (1978), the rod wave and therefore Poisson's ratio is dependent upon the coupling between the

soil skeleton and the pore water. Poisson's ratios close to 0.5 in saturated clays indicate coupling between the soil particles and the pore fluid and is not the effective stress Poisson's ratio for clay. Hamilton (1971) determines Poisson's ratio from bulk density, compressional wave velocity measurements and a calculated bulk modulus. Hamilton found Poisson's ratio to decrease with increasing rigidity and increasing density. Typical values of Poisson's ratio given by Hamilton (1971) are: sands (all grades) 0.470; continental terrace silt, 0.478; deep-sea clay, 0.487. Bishop and Hight (1977) performed a theoretical analysis of Poisson's ratio of saturated rocks and soils. They found the undrained Young's modulus (Equation 4-8) and undrained Poisson's ratio (Equation 4-9) to be dependent upon Skempton's B parameter and the effective stress Poisson's ratio, as shown:

$$E_u = \frac{3E}{3-(1-2v)B} \quad (4-8)$$

and

$$v_u = \frac{3 + (1-2v)B}{3-(1-2v)B} \quad (4-9)$$

where E is the effective Young's modulus and v is the effective Poisson's ratio. If the compressibility of the soil skeleton is large in comparison to the compressibility of the water and the soil solids and if B is approximately equal to unity then the undrained Poisson's ratio is equal to 0.5. Bishop and Hight (1977) state the direct

measurement of Poisson's ratio is difficult due to the accuracy required in the measurements.

The parameters of Young's modulus and Poisson's ratio are both constants from the theory of generalized Hooke's Law, which is based upon the assumption of small strains. Using the equations of unit strain, assuming conditions of triaxial compression and no volume change (incompressible material), it can be shown that Poisson's ratio is equal to 0.5. However, if a cylindrical sample is deformed with no volume change and if all the second order terms are included in the analysis, Poisson's ratio is greater than 0.5. Calculated values of Poisson's ratio from this analysis are listed below for several axial strains:

Axial strain	Poisson's ratio
5%	0.52
10%	0.54
15%	0.56
20%	0.59

This analysis indicates the inaccuracies in small strain theory when applied to large strain conditions. It suggests some degree of flexibility is possible when measuring lateral strain and calculating Poisson's ratio.

4.5 Calculated Poisson's Ratio

Lateral strains were calculated as the change in diameter divided by the original diameter (diameter after consolidation D_0). The diameter change, monitored by the

lateral strain gauge, was determined at the mid-height of the sample. Plots of lateral versus vertical strain are presented and discussed in Section 6.6.

Having measured the principal strains, Poisson's ratio was calculated as:

$$\nu = -\epsilon_L / \epsilon_V \quad (4-10)$$

where ϵ_L is the lateral strain and ϵ_V is the vertical strain. For no volume change conditions, i.e. an undrained test and a right circular cylinder shape, the calculated Poisson's ratio using Equation 4-10 should be approximately one-half. However, the majority of samples tested deformed in a barrel-like shape, by bulging at the center.

To correct Poisson's ratio, it was assumed that the samples deformed in a parabolic shape. The volume of the parabola was then distributed evenly over the entire length of the sample (Figure 4.5a). The new diameter, after the volume redistribution is approximately $2/3$ the diameter at the mid-height of the sample ($D' = 2/3 D_M$). Therefore, the ratio of lateral to vertical strain was multiplied by $2/3$ to determine Poisson's ratio.

The assumed parabolic deformational shape neglects two important factors. One, the deformation was not symmetric over the length of the sample. Second, the true deformation for a majority of the samples appeared as illustrated in Figure 4.5b.

Investigations by several authors indicate Poisson's

ratio is not a constant when measured in triaxial compression. An extensive literature search found only one report of direct measurement of lateral deformation to calculate Poisson's ratio. The more common technique was to measure volumetric and axial strains in a drained triaxial test. Assuming a correct sample deformational shape is a major problem associated with the direct measurement of lateral deformation and the calculation of Poisson's ratio.

Figure 4.1 Disk Gauge on Rock Specimen (from Schuler, 1978)

URI/MGL Lateral Strain Gauge

Figure 4.2

Figure 4.3 Machining Jig and Disk-ring

**Sample with Lateral Strain
Gauge and Metal Tabs
Attached**

Figure 4.4

Figure 4.5 Sample Deformation

CHAPTER 5

STRESS-STRAIN MODELS

5.1 Elastic Models

A constitutive equation or relationship can be defined as a numerical description of physical quantities such as stress, strain, and time (Christian and Desai, 1977). A constitutive law used extensively in geotechnical work to relate stress to strain is Hooke's law. Hooke's law expresses the relationship that for small deformations stress is directly proportional to strain, it is a linear elastic behavior. The generalized Hooke's law equations are due to Navier-Stokes (Sechler, 1952) and can be mathematically represented by

$$\{\sigma\} = [C]\{\epsilon\} \quad (5-1)$$

where $\{\sigma\}$ represents the six components of stress in an elastic body, $\{\epsilon\}$ the six components of strain, and $[C]$ the stress-strain matrix composed of 36 constants. This condition is the most general (anisotropic triaxial) case with six components of stress and strain. Due to the symmetry of the material matrix, $C_{ij} = C_{ji}$, only 21 coefficients are independent.

If the material is assumed homogeneous and isotropic, the following conditions must follow: 1) uniform normal stresses can not produce shear with respect to the same coordinate system; 2) pure shearing stress can not produce extension or compression with respect to the same coordi-

nate system (Sechler, 1952). Also, if isotropy is assumed, the material matrix $[C]$ has only two independent terms: the two material parameters Young's modulus, E , and Poisson's ratio, ν . Hence, for a homogeneous, isotropic material the stress-strain matrix reduces to:

$$C = \frac{E}{(1+\nu)(1-2\nu)} \begin{bmatrix} 1-\nu & \nu & \nu & 0 & 0 & 0 \\ \nu & 1-\nu & \nu & 0 & 0 & 0 \\ \nu & \nu & \nu & 0 & 0 & 0 \\ 0 & 0 & 0 & 1-2\nu & 0 & 0 \\ 0 & 0 & 0 & 0 & 1-2\nu & 0 \\ 0 & 0 & 0 & 0 & 0 & 1-2\nu \end{bmatrix} \quad (5-2)$$

For conditions of plane strain, ϵ_{zz} , ϵ_{xz} and ϵ_{yz} are all zero, further reducing the stress-strain matrix to:

$$C = \frac{E}{(1+\nu)(1-2\nu)} \begin{bmatrix} 1-\nu & \nu & 0 \\ \nu & 1-\nu & 0 \\ 0 & 0 & 1-2\nu \end{bmatrix} \quad (5-3)$$

The two elastic material parameters Young's modulus and Poisson's ratio can both be determined experimentally.

Young's modulus, E , relates axial stress to axial strain in a uniaxial compression test (Cartesian coordinate system, axial direction represented by x-axis) $\sigma_{xx} = E \epsilon_{xx}$ with all other stresses equal to zero. Poisson's ratio, ν , relates axial strain to transverse strain $\epsilon_{yy} = \epsilon_{zz} = -\nu \epsilon_{xx}$ where all other σ 's = 0, except σ_{xx} . The numerical value

of Poisson's ratio has an upper bound of 0.5 corresponding to an incompressible material and a lower bound of zero. The numerical restrictions on Poisson's ratio can be observed by considering the compressibility modulus, the reciprocal of the bulk modulus.

$$\frac{1}{K} = \frac{3(1-2\nu)}{E} \quad (5-4)$$

If $\nu > 0.5$ there would be increasing volume with increasing confining pressure or if $\nu < 0.0$ negative transverse strains would occur under axial tension.

5.2 Nonlinear Models

According to Hardin (1978) it is impossible to apply an effective stress increment to a soil without some slippage at particle contacts and thus plastic deformation. Therefore, a purely elastic analysis of soil deformation will be inaccurate after even small strains. Therefore, if a constitutive equation is to realistically model soil response the nonlinearity of the stress-strain curve must be included. Methods of predicting the nonlinear behavior of soils include the representation of a given stress-strain curve by curve fitting methods, interpolation or mathematical functions such as spline functions, nonlinear elasticity theories, and plasticity theories (Christian and Desai, 1977). Specific models discussed below are the hypoelastic model, hyperbolic model, and the elastoplastic model.

Hypoelastic models have been developed by finding the

necessary soil parameters from curve fitting of the experimental stress-strain curves (Vagneron, et. al, 1976). A grade one hypoelastic model requires seven soil parameters and the soil stress increments are a function of the strain increments.

The hyperbolic model is based upon the work of Konder (1963), who noted that a hyperbola had the approximate shape of a typical stress-strain curve. The hyperbolic model requires eight soil parameters to predict the stress-strain response of the soil. This model will be described in detail in Section 5.5.

An elasto-plastic model (simple plastic) incorporates a yield criterion and an associative or non-associative flow rule. The theory can model non-elasticity, the influence of the intermediate stress, stress-path dependency, and shear dilatancy effects (Vagneron, et. al, 1976) and requires nine soil parameters.

5.3 Mohr-Coulomb Failure Theory

Sediment strength can be expressed by the Mohr-Coulomb failure law as:

$$\tau = \bar{c} + \bar{\sigma}_n \tan \bar{\phi} \quad (5-6)$$

where $\bar{c}$ is the effective cohesion or cohesion intercept, $\bar{\phi}$ is the effective angle of friction, $\bar{\sigma}_n$ is the effective normal stress and τ is the shear strength. Further descriptions of Mohr-Coulomb failure theory can be found in Lambe and Whitman (1969) and Terzaghi and Peck (1967). The

failure state is more commonly expressed in terms of deviator stress, $(\sigma_1 - \sigma_3)$. For drained conditions the deviator stress at failure can be written:

$$(\sigma_1 - \sigma_3)_f = \frac{2\bar{c} \cos \bar{\phi} + 2\sigma_3 \sin \bar{\phi}}{1 - \sin \bar{\phi}} \quad (5-7)$$

and for undrained conditions the deviator stress at failure is:

$$(\sigma_1 - \sigma_3)_f = \frac{2\bar{c} \cos \bar{\phi} + 2\sigma_3 \sin \bar{\phi}}{1 - \sin \bar{\phi} + 2A_f \sin \bar{\phi}} \quad (5-8)$$

where $\bar{c}$ = the cohesion intercept, $\bar{\phi}$ = the angle of internal friction, and A_f = A-Factor at failure. Thus, knowing the three parameters $\bar{c}$, $\bar{\phi}$, and A_f the deviator stress at failure can be determined for any confining stress, σ_3 , for either drained or undrained conditions.

5.4 Hyperbolic Model

Konder (1963) first developed the method of using hyperbolic curves to model the response of soils in triaxial compression. Hyperbolic stress-strain relationships were further developed by Daniel and Olson (1974) and Wong and Duncan (1974). Wong and Duncan (1974) used the hyperbolic relationships in finite element analysis to model soil deformation due to applied loading. In their analysis, an incremental process was used in which the stress-strain response of the soil was assumed linear for each increment and the relationship between stress and strain was assumed to be governed by the generalized Hooke's law of elastic

deformation.

In its hyperbolic form the deviator stress can be written:

$$(\sigma_1 - \sigma_3) = \frac{\epsilon}{a + b\epsilon} \quad (5-9)$$

where (ϵ) is strain; a and b are constants. The ultimate value of the deviator stress can be obtained by taking the limit of Equation 5-9 as strains become larger:

$$(\sigma_1 - \sigma_3)_{ult.} = \lim_{\epsilon \rightarrow \infty} (\sigma_1 - \sigma_3) = 1/b \quad (5-10)$$

Equation 5-9 can also be differentiated with respect to strain and evaluated at $\epsilon = 0$.

$$\frac{d\sigma}{d\epsilon} (\epsilon = 0) = 0 \quad (5-11)$$

If the expression, $\epsilon/(\sigma_1 - \sigma_3)$, is plotted versus strain a linear relationship exists with intercept a and slope b (see Figure 5.1a). The initial Young's modulus, E_i , is equal to $1/a$, the reciprocal of the intercept, and the ultimate deviator stress $(\sigma_1 - \sigma_3)_{ult}$ is equal to $1/b$, the reciprocal of the slope. The hyperbolic stress-strain curve can then be expressed as:

$$(\sigma_1 - \sigma_3) = \frac{\epsilon}{\frac{1}{E_i} + \frac{\epsilon}{(\sigma_1 - \sigma_3)_{ult.}}} \quad (5-12)$$

and is plotted in Figure 5.1b.

For increasing values of confining stress, σ_3 ; the values of E_i , the initial Young's modulus and $(\sigma_1 - \sigma_3)_{ult.}$,

the ultimate deviator stress increase. After extensively studying a large number of stress-strain curves for a variety of materials, Janbu (1963) showed that the tangent modulus varied with effective stress. The tangent modulus could be represented by the formula:

$$M = m \sigma_a \left[\frac{\bar{\sigma}}{\sigma_a} \right]^{1-a} \quad (5-13)$$

M = tangent modulus

m = modulus number

$\bar{\sigma}$ = the effective stress

σ_a = atmospheric pressure

a = exponent number between 0 and 1

For a purely elastic material a equals unity and m equals zero for a normally consolidated clay (Janbu, 1963).

Janbu's formula may be modified to develop an equation for E_i as a function of confining stress, resulting in the formula:

$$E_i = K P_a \left(\frac{\sigma_3}{P_a} \right)^n \quad (5-14)$$

where K is the modulus number, n is the exponent number and P_a is the atmospheric pressure. The values of K and n may be determined by plotting values of $\log (E_i/P_a)$ versus $\log (\sigma_3/P_a)$ (see Figure 5.2).

The deviator stress at failure, $(\sigma_1 - \sigma_3)_f$, can be expressed in terms of the ultimate deviator stress as:

$$(\sigma_1 - \sigma_3)_f = R_f (\sigma_1 - \sigma_3)_{ult.} \quad (5-15)$$

From the theory, the values of $(\sigma_1 - \sigma_3)_f$ cannot exceed $(\sigma_1 - \sigma_3)_{ult.}$. Thus the extreme value of R_f is unity.

By differentiating Equation 5-12 with respect to strain and substituting Equations 5-14 and 5-15, an expression for the tangent Young's modulus can be obtained.

$$E_t = \left(1 - \frac{R_f (\sigma_1 - \sigma_3)}{(\sigma_1 - \sigma_3)_f} \right)^2 K P_a \left(\frac{\sigma_3}{P_a} \right)^n \quad (5-16)$$

For drained conditions Equation 5-7 would be substituted into Equation 5-16 for the term $(\sigma_1 - \sigma_3)_f$, whereas Equation 5-8 would be substituted into Equation 5-16 for undrained conditions. Thus, for undrained conditions the following expression for E_t results:

$$E_t = \left(1 - \frac{R_f (1 - \sin \bar{\phi} + 2A_f \sin \bar{\phi}) (\sigma_1 - \sigma_3)}{2 \bar{c} \cos \bar{\phi} + 2\sigma_3 \sin \bar{\phi}} \right)^2 K P_a \left(\frac{\sigma_3}{P_a} \right)^n$$

This equation can then be used to calculate the value of tangent Young's modulus for any stress condition σ_3 and $(\sigma_1 - \sigma_3)$ if the value of K , n , $\bar{c}$, $\bar{\phi}$, R_f , and A_f are known. The expression for tangent modulus applies only to stresses up to and including the peak deviator stress and cannot predict strain softening.

The major advantage of the hyperbolic method is that the required soil parameters may be obtained from standard triaxial tests. However, the above advantage is also a weakness of the hyperbolic model (Vagneron, et. al, 1976). The model is not general, it applies only to the type of

loading condition used in defining the parameters. Therefore, the laboratory tests used to determine the soil parameters should approximate the loading conditions which will exist in the field.

The model is only accurate for predicting displacements under stable soil conditions, a requirement imposed upon the model by the generalized Hooke's law. Hooke's law is only applicable for the analysis of stress and strain conditions prior to failure (Wong and Duncan, 1974). Another limitation of the model was discussed earlier in Section 5.2. Elastic theory requires an upper bound on the numerical value of Poisson's ratio of 0.5. With this restriction, the hyperbolic model cannot predict volume changes in soils subject to shear dilatancy (Wong and Duncan, 1974).

5.5 Hyperbolic Data Analysis

To determine the hyperbolic parameters for each sample, the axial strain is divided by the deviator stress and plotted versus strain. A best fit line through all the points is used to determine the values of E_i , the initial Young's modulus and $(\sigma_1 - \sigma_3)_{ult.}$, the ultimate deviator stress. Using Equation 5-12 with the values of E_i and $(\sigma_1 - \sigma_3)_{ult.}$, the hyperbolic stress-strain curve can be generated and plotted with the experimental stress-strain curve (for example, see Figure 5.3). The solid line on the plot represents the hyperbolic curve and the asterisks (*) the experimental points. It should be emphasized that in

this plot the smooth curve is not a best-fit line through the experimental points, but represents a hyperbolic curve that has been fit to the experimental data points using the assumptions discussed in Section 5.5.

The values of K and n are determined, as outlined in Section 5.5, for each sample series. The change in tangent modulus E_t , with changing deviator stress $(\sigma_1 - \sigma_3)$ is determined using Equation 5.17. The tangent modulus for the experimental points is determined using the formula:

$$E_t = \frac{(\sigma_1 - \sigma_3)_{i+1} - (\sigma_1 - \sigma_3)_{i-1}}{\epsilon_{i+1} - \epsilon_{i-1}} \quad (5-18)$$

The two sets of values can be plotted together (for example, see Figure 5.4), the hyperbolic E_t represented by the solid line and the experimental E_t values with asterisks (*). Chapter 6 presents a detailed analysis of the hyperbolic data generated in the test program.

(a)

(b)

Hyperbolic Representation of a Stress-Strain Curve (from Wong and Duncan, 1974)

Figure 5.1 (a & b)

**VARIATION OF INITIAL TANGENT MODULUS
WITH CONFINING PRESSURE**

Figure 5.2

(from Wong and Duncan, 1974)

STRAIN %
DEV. STRESS vs. STRAIN
P.S.-18 HORIZ.

Figure 5.3

DEV. STRESS (KG/CM**2)
TANG. MOD. vs. DEV. STRESS
P.S.-18 HORIZ.

Figure 5.4

CHAPTER 6

PRESENTATION OF RESULTS AND DATA ANALYSIS

6.1 Introduction

This chapter presents: 1) all the test data compiled by this investigation and 2) a discussion of the results. Section 6.2 presents all the stress-strain data followed by a drained strength analysis (Section 6.3) and an undrained strength analysis (Section 6.4). A detailed analysis of the hyperbolic parameters appears in Section 6.5 and a discussion of the lateral strain measurements and calculated Poisson's ratio are presented in Section 6.6.

The test data has been analyzed using effective stress principles and Mohr-Coulomb failure theory. The peak deviator stress was adopted as the failure criteria. If no peak stress was observed then other stress-strain curves: pore pressure, stress-path, A-factor, or $\bar{\sigma}_1/\bar{\sigma}_3$ were used to select a failure point. A least squares line was fit to the experimental failure points to determine the cohesion intercept and the angle of friction. Therefore, the experimental failure points will not necessarily fall exactly on the calculated failure envelope.

In general, four samples were tested in each sample series. Two samples were consolidated to stresses greater than any past maximum consolidation stress imposed upon the samples (the samples were normally consolidated). Two samples were consolidated to stresses less than the past maximum. The ratio of the past maximum to the existing consoli-

dation is defined as the overconsolidation ratio (OCR).

Results of grain size, Atterberg limits, and specific gravity tests are listed in Table 6.1 for all the samples tested. These samples are classified as either inorganic clays (CH) or inorganic silts (MH) by the Unified Soil Classification System, plotting slightly above or below the A-line.

All samples (excluding LL-44 GPC-3 1885 cm) contain more than 65% clay sized particles with a median diameter of one micron (Table 6.1). The liquid and plastic limits of the smectites are considerably higher than the illites. The activity of the smectite samples is also higher, indicating the degree to which water is attracted to the sediment. The liquidity index (I_L) in soil is greater than unity if the natural water content is greater than the liquid limit. The liquidity index is also an indicator of the soils susceptibility to disturbance. The undisturbed illite sediments exhibit liquidity indices greater than unity (Table 6.1). The values of specific gravity (G_s) are very high for the majority of samples, due to the high iron content of these sediments (Heath, 1978).

6.2 Discussion of Stress-Strain Data

In this section, the test data from each sample series (Figure 1.1) is presented and discussed in detail. Pertinent plots of Mohr-Coulomb parameters are included for each sample series. Also included are plots of lateral strain, Poisson's ratio and the hyperbolic curves of deviator stress

and tangent modulus.

Table 6.2 lists the failure parameters for all the samples tested. The parameters listed are: consolidation pressure σ_3 , vertical strain at failure ϵ_f , volumetric strain during consolidation ϵ_{vol} , deviator stress at failure $(\sigma_1 - \sigma_3)_f$, pore pressure at failure u_f , principal stress ratio $\bar{\sigma}_1 / \bar{\sigma}_3$, A-factor at failure A_f , q_f one-half the deviator stress at failure, $\bar{p}_f$ the average of the principal stresses, the initial water content w_i , the water content at failure w_f , the ultimate deviator stress from the hyperbolic analysis $(\sigma_1 - \sigma_3)_{ult}$, the initial tangent modulus E_i , and the ratio of the ultimate to experimental deviator stress R_f . Table 6.3 lists the Mohr-Coulomb failure parameters: $\bar{c}$, the effective cohesion intercept and $\bar{\phi}$, the angle of internal friction. These values are obtained from the following formula:

$$\begin{aligned} \sin \bar{\phi} &= \tan \bar{\alpha} \\ \bar{c} &= \frac{\bar{a}}{\cos \bar{\phi}} \end{aligned} \quad (6-1)$$

where the terms $\bar{a}$ and $\bar{\alpha}$ are determined from the analysis of the failure points on the q versus $\bar{p}$ plot. Also listed on Table 6.3 are values of K (the modulus number) and n (the exponent number) determined from the hyperbolic theory.

A) LL-44 GPC-3 1800 cm

These three samples are from a series of four samples, with the fourth sample being tested as part of the creep testing program. From observing core photographs, the

sample 1845-1857 cm was taken from an apparently homogenous section of core. The remaining samples in this series were taken in a zone of mottling (see Figure 2.5). When extruded the samples were stiff, with no noticeable layering or mottling. The initial water contents vary from 191 to 203% and are similar to those in the water content profile of GPC-3 (Figure 2.4), which vary from 196 to 226% between the depths of 1845 and 1890 centimeters.

In observing the plots of deviator stress and the stress paths (Figure 6.1 and 6.3), it appears that the behavior of the sample 1845-1857 cm is different than the other two samples, with a higher shear strength. The difference in behavior is again evident in the volumetric strains during consolidation which were 38.3%, 19.6%, and 22.8% respectively for the three samples 1845-1857 cm, 1859-1868 cm and 1873-1883 cm (see Table 6.2). This behavior would suggest that the sample 1845-1857 cm is of a different material. Due to the changes in composition observed in the photographs and the inconsistent behavior of this sample, it was discarded in the analysis of the values of cohesion intercept and angle of internal friction.

The two samples 1859-1868 cm and 1873-1883 cm exhibit stress-strain curves; deviator stress, pore pressure, stress path, A-factor, and $\bar{\sigma}_1/\bar{\sigma}_3$ (Figures 6.1 through 6.5) which are identical in shape. Both samples exhibit strain-softening with increasing pore pressures after failure (failure defined as peak deviator stress). All three sam-

ples developed distinct shear planes after failure. All the samples were tested as normally consolidated, the A-factors of 1873-1883 and 1859-1868 cm samples (Figure 6.4) approaching unity at 20% strain. Lateral strains were measured on two samples 1845-1857 and 1859-1868 cm. The gauge on sample 1873-1883 cm fell off the sample during preparation for shearing. The lateral strains measured are nearly identical up to 10% vertical strain (Figure 6.6). The lateral strain and values of Poisson's ratio will be discussed in a later section of this chapter. The hyperbolic analysis was performed on the test results from these samples, however negative values of E_1 were obtained and the results are therefore not included (see Section 6.5).

B) PS-9 Vertical Samples

The four samples in this series were obtained from a Hole Closure Experiment tank of smectite with a simulated depth of 9 meters (consolidation stress of 0.25 kg/cm^2). The remolded samples identified as "vertical" were taken with their axis parallel to the axis of the consolidation tank. The "horizontal" samples were taken at right angles to the axis of the tank. Two samples PS-9 #1 and #2 were tested as normally consolidated. Samples PS-9 #3 and #4 were tested as overconsolidated (OCR's of 1.7 and 5.2 respectively).

The spikes observed in the pore pressure versus strain plots (Figure 6.9) were caused by fluctuations in the air supply system. The regulators, although very sensitive,

were not able to completely compensate for the extreme fluctuations in the system. This problem was reduced later in the testing program by placing an additional regulator between the compressor and the control panel regulators. The absence of spikes in PS-9 #4 results from the use of a differential pore pressure transducer such that the change in cell pressure is automatically subtracted from the pore pressure readings. The spikes in the deviator stress curve for sample #4 (Figure 6.8) are attributed to changes in the piston load caused by the fluctuating line pressures. Although sample #4 was consolidated to a confining stress less than that of sample #3, the deviatoric stresses of sample #4 are greater than sample #3.

The fluctuations in pore pressure effect the other values which are a function of pore pressure. These fluctuations are amplified in the plot of A-factor versus strain (Figure 6.11). Cell pressure was not recorded during the shear process, therefore the variations in cell pressure are not known.

Lateral strains were measured in tests PS-9 #3 and #4. The two curves of lateral strain versus vertical strain (Figure 6.13) are considerably different. The hyperbolic analysis was performed on all four samples, the plot of deviator stress versus strain (Figure 6.15) shows the excellent fit of this analysis. A plot of tangent modulus versus deviator stress (Figure 6.16) compares the derived and experimental values. Values of K and n used for the

analysis of tangent Young's modulus are listed in Table 6.3.

C) PS-18 Vertical

These four samples were obtained from a Hole Closure Experiment tank of smectite having a simulated depth of 18 meters ($\bar{\sigma}_c = 0.50 \text{ kg/cm}^2$). Two samples were tested as normally consolidated (PS-18 Vertical #1 and #4) and two samples were overconsolidated (#3, OCR = 1.7 and #5, OCR = 5.2).

The spikes observed in the PS-9 samples were considerably reduced in this series, only samples #3 and #4 exhibit fluctuations in pore pressure (Figure 6.18). Samples #3 and #5 display characteristics typical of an overconsolidated cohesive soil. The pore pressure is initially positive and then approaches zero, resulting in a sharp peak in A-factor (Figure 6.20) gradually decreasing to zero. The stress paths (Figure 6.19) for these samples are typical of overconsolidated samples, moving from left to right. The #5 sample produced a well-defined shear plane after failure.

Stress-strain curves of PS-18 #1 and #4 peak at approximately 12% strain (Figure 6.17), while the overconsolidated samples peak at a smaller strain (less than 5% strain). The dissimilar behavior of these samples (normally and overconsolidated) can also be observed in the plots of pore pressure-strain, A-factor-strain, and $\bar{\sigma}_1/\bar{\sigma}_3$ - strain (Figures 6.18, 6.20, and 6.21).

Lateral strains were measured on samples #1 and #3 (Figure 6.22). As observed in the LL-44 samples, the lateral strains are nearly identical. The hyperbolic stress-strain curves again exhibit an excellent fit to the experimental points (Figure 6.24). In this series, only sample #3 had negative values of E_i . The tangent modulus values also compare well with the experimental values as illustrated in the plot of tangent modulus versus deviator stress (Figure 6.25).

D) PS-18 Horizontal

These samples were obtained from the same Hole Closure Experiment tank as the PS-18 Vertical samples. The consolidation stresses used to test these samples approximated the stresses used in the Vertical series. The two samples (#7 and #10) were tested as normally consolidated and (#8 and #9) were tested as overconsolidated ($OCR = 1.7$ and 5.2 respectively).

Sample #9 displays pore pressure, A-factor and stress path curves (Figure 6.27, 6.29, and 6.28) nearly identical in shape to PS-18 Vertical #5 (the same consolidation pressure was used). The A-factor curve for sample #9 peaks at a very low strain ($< 1\%$) and then gradually decreases to zero and is negative after 10% strain. The failure strains of the normally consolidated samples in the Vertical series were similar to sample #7 (Table 6.2). The stress path for sample #7 (Figure 6.28) is typical of a normally consolidated sample, initially moving to the right and then left

to the failure envelope. The difference in behavior of the normally and overconsolidated samples is observed in the plot of σ_1/σ_3 versus strain (Figure 6.30).

Lateral strain measurements are given for two samples (#8 and #10) (Figure 6.31). Again the lateral strains are nearly identical. The hyperbolic fit provides an excellent fit to the experimental values, especially #9 (Figure 6.33).

E) MARA-02 GC-04 200 cm

The three samples in this series were from a set of four but the fourth sample was disturbed during sample preparation and was not tested. The samples were taken from the bottom of the core where the sediment was visually homogeneous. The initial water contents of the samples vary from 106 to 108% (Table 6.2) which agree well with the profile water contents, 102 to 110% between the depths of 210 and 245 centimeters (Figure 2.7). Figure 2.7 illustrates that the natural water contents of the illite samples are greater than the liquid limits. The high values of specific gravity listed in Table 6.1 are probably the result of the high iron content (Heath, 1978) of these sediments. The stress-strain curves for this series, (Figure 6.35) although exhibiting small fluctuations, are typical of the shape obtained for plastic cohesive soils. As with the LL-44 samples, the strains at failure were between 0-10%. The spikes in the pore pressure versus strain plots (Figure 6.36) were also present in this series. The

pore pressures of sample 216-224 cm were measured with a differential transducer. This sample was tested as over-consolidated, the calculated preconsolidation stress equal to 0.19 kg/cm^2 , resulting in an OCR of 1.7. This sample behaved as a typical overconsolidated clay with noticeable differences from the two normally consolidated samples as observed in the plots of A-factor and $\bar{\sigma}_1/\bar{\sigma}_3$ versus strain (Figures 6.38 and 6.39).

The similar behavior of the two normally consolidated samples 228-235 cm and 238-245 cm indicates two important facts. First, the quality of the samples is indicated by the similar shapes of the stress paths, A-factor and $\bar{\sigma}_1/\bar{\sigma}_3$ curves. Second, the similar behavior suggests the test procedures and sample preparation are of a superior quality.

Lateral strain measurements were obtained for all the samples tested in this series. Similar strains are observed for samples 238-245 and 228-235 cm (Figure 6.40). The curves from the hyperbolic analysis compare well with the experimental values after 5% strain (Figure 6.42). The tangent modulus values from the hyperbolic analysis tend to overestimate the experimental values (Figure 6.43).

F) MARA-02 GC-04 100 cm

This series of four samples was taken from the upper meter of GC-04. The initial water contents of the samples vary from 105 to 121% as compared to 109 to 123% between 75 and 115 centimeters (Figure 2.7). A change in sediment

structure was detected in the depth range 95 to 105 cm (see Section 2.6). No differences are noted at this depth in the water content profile or the grain size and Atterberg limits. However, the specific gravity for the sample 95-105 cm (Table 6.1) is considerably higher than the other samples in this depth range. These factors indicate the sediment at this depth (95-105 cm) is of a higher void ratio (due to the higher specific gravity) than the sediment above or below this zone. The lower vane shear strength at this depth (see Figure 2.7) is attributed to the higher void ratio. The differences between the natural water contents and the liquid limits are greater in this depth range than in the lower core, an indicator of a potentially more sensitive material.

It is noted that equipment problems were encountered in this test series. The differential pore pressure transducer used with samples 86-94 and 95-105 cm was believed to be defective. This was not learned however until after the two samples were sheared. The transducer was believed to give pressure readings which were too high, explaining why the two tests do not reach the failure envelope.

Pore pressure fluctuations due to line fluctuations were not recorded in the tests of samples 76-84, 86-94, and 95-105 cm (Figure 6.45) because differential pore pressure transducers were used. However the large irregularities in the stress-strain curves (Figure 6.44) could be attributed to variations in cell pressure due to the line fluctuations.

Only the failure points from tests 78-84 and 105-115 cm were used to calculate $\bar{c}$ and $\bar{\phi}$ (Figure 6.46). Since the 76-84 cm sample was overconsolidated, the cohesion intercept is higher than if two normally consolidated tests were used in the analysis.

Lateral strain measurements were obtained for three of the four samples (Figure 6.49). The gauge on the fourth sample (95-105 cm) fell off the sample due to large volume changes during consolidation. The two samples 105-115 and 86-94 cm display near identical lateral strains. The hyperbolic stress-strain curves fit the experimental points very well despite the variations in the curves (Figure 6.51). The tangent modulus versus deviator stress curves for the samples 86-94 and 95-105 cm (Figure 6.52) do not approach values of zero (E_t) because their stress paths do not reach the failure envelope (see Section 5.5).

G) VEMA-32 PC-115 200 cm

These three samples were from a set of four samples taken in the upper few meters of PC-115. The 3.8 cm diameter samples were taken from a 6.4 cm inside diameter standard piston corer. This corer results in a somewhat more disturbed sample than the larger diameter (10.2 cm) gravity and Giant Piston Corer (11.4 cm). The initial water contents of the samples vary between 99 and 106% as compared to the natural water contents from the core profile of 102 to 110% between the depths of 210 and 250 cm (Figure 2.6). Classification data for these samples is

similar to that of the MARA samples.

The three samples were tested as normally consolidated, the plots of stress versus strain (Figure 6.53) having shapes similar to those of the MARA samples. These samples tend to strain harden, that is the deviator stress does not peak but rather continues to increase gradually beyond 5% strain. The similarity of all the samples in this series is observed in the plot of A-factor versus strain (Figure 6.56), all samples have A-factors approximately equal to unity at failure. Lateral strains were not measured on these samples. The hyperbolic analysis demonstrates its excellent fit to the experimental values (Figures 6.58 and 6.59).

H) VEMA-32 PC-115 400-500 cm

This series of samples was taken from a depth in the core approaching the transition zone of illite to smectite. The initial water content values of the triaxial samples ranging from 106 to 116% compare well with the natural water contents ranging from 104 to 117% between the depths of 490 and 540 centimeters (Figure 2.6). The liquid limits at this depth are 25 to 38% greater than the limits at the shallower depths of 235-245 cm. In observing Figure 2.6, in this depth range the natural water contents are approximately equal to the liquid limits, another indication of transition material.

The two samples 499-509 and 523-533 cm were tested as normally consolidated and the sample 483-493 cm was over-

consolidated with the calculated preconsolidation stress equal to 0.33 kg/cm^2 resulting in a OCR of 2.3. The stress-strain curves for these samples (Figure 6.60), although somewhat similar in shape to the shallower VEMA and MARA samples, show a tendency for strain-softening. The fluctuations in pore pressure are again evident in the plots of pore pressure versus strain (Figure 6.61). From the plots of pore pressure, A-factor, and $\bar{\sigma}_1/\bar{\sigma}_3$ versus strain (Figures 6.61, 6.63, and 6.64), the behavior of the overconsolidated sample 483-493 cm is observed.

Lateral strains were not measured on these samples. The hyperbolic analysis again provides an excellent fit for these samples (Figure 6.65). The fit of the derived tangent modulus values to the experimental values is also excellent (Figure 6.66).

I) PI-18 Vertical

This series of samples was removed from a Hole Closure Experiment tank of illite material with a simulated depth of 18 meters ($\bar{\sigma}_c = 0.82 \text{ kg/cm}^2$). The samples #1 and #2 were tested as normally consolidated and samples #3 and #4 were overconsolidated, having OCR's of 2.1 and 5.1 respectively.

Sample #2 exhibits a large amount of strain hardening (Figure 6.67) with no change in pore pressure (Figure 6.68). The combined effect of this behavior results in the stress path (Figure 6.69) turning back after failure, moving to the right. The failure point for this sample was selected

as the minimum value of $\bar{p}$ which occurs at the point of maximum A-factor. The curves of A-factor versus strain (Figure 6.70) for samples #3 and #4 are typical of over-consolidated samples, if the fluctuations due to line pressure variations are neglected. No lateral strains were measured on these samples. The hyperbolic stress-strain curves (Figure 6.72) display an excellent fit for samples #3 and #4. The computed tangent modulus versus deviator stress curves exhibit varying degrees of fit (Figure 6.73). The hyperbolic analysis cannot predict strain softening, resulting in a poor quality fit of the stress-strain and tangent modulus curves for sample #1.

J) PI-2 Vertical

Before discussing the next two series of samples, a comment on the quality of the samples is necessary. Small air voids were noticed in the sides of these samples during extrusion from the sampling tubes. Voids were not observed in any other samples, suggesting the voids were a result of the low effective stresses used in this tank. These samples have been analyzed and discussed in this report neglecting as much as possible the poor quality of the test results. Caution must be exercised when drawing conclusions from the test data of these samples.

The four samples in this series were obtained from a Hole Closure Experiment tank of illite with a simulated depth of two meters ($\bar{\sigma}_c = 0.10 \text{ kg/cm}^2$). Samples #1 and #2 were tested as normally consolidated and samples #4 and #5

were overconsolidated with OCR's of 1.9 and 3.9 respectively. The irregularities observed in the stress-strain curves (Figure 6.74) result from fluctuations in line pressure. They appear to be cyclic for sample #4, which would further support the contention they are caused by the air compressor fluctuations. Samples #1 and #2 exhibit unusual stress-strain curves, in that the deviator stress peaks at a low strain ($< 5\%$), then decreases for these samples (Figure 6.77) are also high when compared to other remolded samples. The difference in behavior of these samples (#4 and #5) is shown in the plot of A-factor versus strain (Figure 6.77). The effect of the small fluctuations in the stress-strain plot of samples #4 and #5 are observed in the last few points of the $\bar{\sigma}_1/\bar{\sigma}_3$ versus strain curves (Figure 6.78).

Lateral strain measurements were recorded for two samples (#4 and #5) (Figure 6.79). The lateral strains for sample #4 are untypical of the strains measured in other samples. Due to the variations in the stress-strain curves the hyperbolic curves give an average fit (Figures 6.81 and 6.82).

K) PI-2 Horizontal

The four samples of this series were obtained from the same tank as the PI-2 Vertical samples and thus the same problems relative to sample quality were evident. Consolidation pressures, approximately equal to those used to test the Vertical samples, were used to consolidate these samples.

samples #6 and #7 were tested as normally consolidated and samples #8 and #9 were overconsolidated, with OCR's of 1.9 and 3.9 respectively. Again the fluctuations observed in the stress-strain plot (Figure 6.83) are caused by variations in the cell pressure and/or the poorer quality samples. These fluctuations are amplified in the plots of A-factor and $\bar{\sigma}_1/\bar{\sigma}_3$ versus strain (Figure 6.86 and 6.87). The low failure strains determined for these samples are the result of the difficulty in determining a failure point. These samples are of better quality, in terms of the shape of the curves, than the PI-2 Vertical samples. The A-factors (Figure 6.86) for the normally consolidated samples (#6 and #7) are questionable.

The plot of lateral versus vertical strain displays similar strains for samples #6 and #7 (Figure 6.88) but the lateral strain measurements of samples #8 and #9 are questionable. The hyperbolic stress-strain curves (Figures 6.90) exhibit a good fit despite the irregular experimental values. The hyperbolic tangent modulus values underestimate the experimental values (Figure 6.91).

6.3 Drained Strength Analysis

A) Introduction

The effective strength parameters of effective cohesion $\bar{c}$ and effective angle of internal friction $\bar{\phi}$ can be used in a drained strength analysis. The direct comparisons of these parameters can be one basis of evaluating test data in terms of the goals of the testing program

(see Section 1.6). However it is emphasized that drained tests were not conducted in this program. Terzaghi's principle of effective stress $\sigma = \bar{\sigma} + u$ states that the total stress is equal to the effective stress plus the pore water pressure. Knowing the values of $\bar{c}$ and $\bar{\phi}$, the drained strength at failure can be calculated using the formula:

$$q_{fd}/\bar{\sigma}_c = \frac{\bar{c} \cos \bar{\phi} + \bar{\sigma}_c \sin \bar{\phi}}{(1 - \sin \bar{\phi}) \bar{\sigma}_c} \quad (6-2)$$

where q_{fd} is one-half the deviator stress (drained strength) at failure and $\bar{\sigma}_c$ is the consolidation stress (Lambe and Whitman, 1969). This formula assumes drained conditions, i.e. no pore pressure, and assumes K_0 is equal to unity.

Using Equation 6-2 and the values of $\bar{c}$ and $\bar{\phi}$ listed in Table 6.3 drained strengths at failure were calculated for the samples tested as normally consolidated. These values are listed in Table 6.4 under method three. The three parameters, $\bar{c}$, $\bar{\phi}$, and $q_{fd}/\bar{\sigma}_c$ are used to compare samples in terms of drained strength.

B) Undisturbed vs. Remolded Samples

Comparing the values of $\bar{c}$ and $\bar{\phi}$ for the undisturbed and remolded samples of smectite (Table 6.3), only small variations are observed. For sample series of equal vertical stress LL-44 GPC-3 1800 cm and PS-18 Vertical, the difference in friction angle (35.5° and 36.3° respectively) is

not considered significant. Also, the differences in cohesion intercept for the same series (0.016 and 0.009 kg/cm² respectively) are small since both values are close to zero. The effect of the combined difference in cohesion and friction angle are observed in the average $q_{fd}/\bar{\sigma}_c$ ratios (Table 6.4), 1.40 for LL-44 samples and 1.47 for PS-18 Vertical samples (Table 6.4). This ratio indicates that the remolded samples are slightly stronger than the undisturbed samples in terms of drained strength at different water contents. Although the undisturbed samples were consolidated to higher effective stresses than the PS-18 Vertical samples, the undisturbed samples exhibit larger water contents at failure. The differences in water content indicate dissimilar void ratios, due to the direct relationship between water content and void ratio. To compare the strengths on an equal basis, the void ratios, i.e. water contents, must be the same. A higher consolidation stress for the undisturbed samples would be required, resulting in an increased strength. The magnitude of this increase can not be determined with the available data. Overall the drained strength of these two sample series LL-44 and PS-18 Vertical should be considered nearly equal.

Comparing the friction angles for the undisturbed smectite samples (LL-44 GPC-3 1800 cm) and the other remolded sample series, a small variation in values is observed (37.3° for PS-9 Vertical and 33.4° for PS-18 Horizontal). These differences are considered more substantial

than those observed between the undisturbed samples and the PS-18 Vertical series. Based on friction angle alone, the PS-9 Vertical series is stronger than the undisturbed sample series. The low angle of friction 33.4° for the PS-18 Horizontal series could be attributed to the horizontal orientation of the samples. This will be discussed more extensively when comparing the vertical to horizontal oriented samples.

Values of cohesion intercept for all smectite series vary from 0.025 to 0.009 kg/cm², a difference of only 0.016 kg/cm². However, the $q_{fd}/\bar{\sigma}_c$ ratios (Table 6.4) range from 1.27 (average value) for PS-18 Horizontal to 1.57 (average value) for PS-9 Vertical. This difference is primarily caused by the variation in friction angle, since the cohesion intercept does not effect the $q_{fd}/\bar{\sigma}_c$ ratio significantly. Using the LL-44 sample series as an example, a 10% change in friction angle causes a 20% change in the $q_{fd}/\bar{\sigma}_c$ ratio, but a 40% change in cohesion intercept causes only a 0.5% change in the $q_{fd}/\bar{\sigma}_c$ ratio. Therefore, the differences in cohesion intercept must be considered insignificant when comparing undisturbed and remolded samples.

Comparing values of $q_{fd}/\bar{\sigma}_c$ ratio for the remolded series PS-9 Vertical, PS-18 Horizontal and the undisturbed series LL-44, indicate the PS-9 samples are stronger than the undisturbed samples and the PS-18 Horizontal samples are considerably weaker than the PS-9 samples. Again, the factor of water content at failure must be considered in

comparing the $q_{fd}/\bar{\sigma}_c$ ratios. The water content of the undisturbed sample 1859-1868 cm is approximately 50% greater ($w = 156.5$) than the water content at an equivalent consolidation pressure for PS-18 #4 Vertical ($w = 106.8$) and is 33% greater than the water content of sample PS-18 #7 ($w = 117.3$).

Comparing the remolded and undisturbed smectite sample series on the basis of friction angle alone, a remolded sample series (PS-18 Vertical) exhibits an equal value of $\bar{\phi}$ compared to the undisturbed series (LL-44 GPC-3 1800 cm), 36.3° versus 35.5° respectively (Table 6.3). However, a second remolded series, PS-9, exhibits a $\bar{\phi}$ value greater than the undisturbed series, 37.3° versus 35.5° respectively. The friction angle for the smectite sediments is apparently not significantly influenced by remolding or disturbance.

Comparisons of friction angle for the remolded and undisturbed illite sample series are complicated by the low friction angles observed in the undisturbed VEMA-32 sample series. In general, neglecting the VEMA samples, the undisturbed illite sample series have a higher friction angle than the remolded samples (Table 6.3). The lower friction angles of the VEMA samples can be attributed to the high sensitivity of the illite material and the tendency toward considerable disturbance with the standard piston corer. The average sensitivity from vane shear measurements of MA-02 GC-04 is 5.5 and for the upper 5 meters of GPC-3 the

sensitivity is 6.7. In GPC-3 the sensitivity decreases with depth in the core, thus suggesting a change in susceptibility to disturbance with the change in sediment type. The transition layer (5-10 meters) has a sensitivity of 4.8 and for the smectite, it decreases from 2.9 at 10 meters to 2.5 below 20 meters. On the basis of sensitivity, the illite material is more sensitive to disturbance and remolding than the smectite and transition material. This statement is supported by a comparison of the water contents and liquid limits for the three sediments. The natural water content of the illite was found to be greater than the liquid limit in the three cores LL-44, VEMA-32, and MARA-02 (see Section 2.7), an indication of potentially sensitive sediment. Thus, the lower friction angles observed in the VEMA samples are attributed to sample disturbance, which is a function of corer design. The VEMA samples were obtained with a standard piston corer having an inside diameter of 6.4 cm (2.5 inches) as opposed to the 10.2 cm (4.0 inch) inside diameter of the gravity corer used to obtain the MARA samples. Another factor is the piston oscillations inherent in the piston coring technique. The larger corer diameter provides a better quality sample and allows the triaxial samples to be taken away from the zones of disturbance along the walls of the core barrel. The VEMA samples which were taken at a depth of 400-500 cm are entering the transition layer (increasing smectite content) and therefore are less sensi-

tive to disturbance, resulting in a greater friction angle.

The influence of remolding on the angle of internal friction is also exhibited in a comparison of the remolded sample series. Comparing the $\bar{\phi}$ values of the MARA samples to the remolded sample series a difference of as much as 12 degrees is observed between the 200 cm series and the PI-2 Horizontal series (34.8° versus 22.8° respectively) (Table 6.3). Comparing samples of equal vertical stress for MA-02 GC-04 200 cm and PI-2 Vertical, the undisturbed samples are considerably stronger (34.8° and 24.6° respectively) than the remolded samples. The higher friction angles in the undisturbed samples can be attributed to differences in sediment structure and physio-chemical factors. It is generally accepted that undisturbed samples have an inherent structure which upon remolding is irreversibly altered. The loss of structure is indicated by the dissimilar water contents of the undisturbed and remolded samples, the undisturbed samples having a larger void ratio, i.e. water content. The effects of structure will be discussed in Section 6.4.

The cohesion intercepts for the illite samples vary considerably with a low value of zero for MA-02 GC-04 200 cm to a high value of 0.101 kg/cm^2 for PI-18 Vertical. The high values of cohesion observed in the PI-18 series can not be accounted for by problems with the stress-paths, both normally consolidated samples exhibit a distinct failure point. As was the case with the smectite samples,

the differences in cohesion intercept should be considered less significant when compared to the influence of friction angle upon the shear strengths.

Observing the normalized drained strengths in Table 6.4 (Method 3), the undisturbed samples are stronger than the remolded samples, neglecting the VEMA-32 samples. The only sample series having a $q_{fd}/\bar{\sigma}_c$ ratio similar to the MARA samples is PI-18 Vertical. However, in comparing water contents of these samples, those of the undisturbed samples are higher by 23%, ($w = 87\%$ versus $w = 64\%$) than the PI-18 series. The $q_{fd}/\bar{\sigma}_c$ ratios for the PI-2 Vertical samples are slightly greater than those of the V-32 200 cm series, (0.77 versus 0.70) at the same water content, 77%. The normalized strengths derived using method #3 (drained analysis) are a function of both $\bar{c}$ and $\bar{\phi}$ (the influence of $\bar{c}$ is minimal). Comparisons based on the normalized drained strengths will therefore lead to the same conclusions developed in the comparisons of the friction angles.

In summary, the friction angles for the "undisturbed" illite samples are greater than for the remolded samples. Remolding has a strong influence on the friction angle.

C) Smectite vs. Illite Samples

Comparing the vertically oriented smectite and illite samples, the drained strengths of the smectite samples are greater (Table 6.4). The cohesion intercepts of the smectite samples are in general lower than the illite samples, the exceptions include MA-02 GC-04 200 cm ($c = 0.0 \text{ kg/cm}^2$).

These low values should be considered less significant, as stated before, when compared to the change in strength resulting from variations in friction angle. The $q_{fd}/\bar{\sigma}_c$ ratios, neglecting the horizontal series, of the smectite samples are greater than the illite samples (Table 6.4). The smectite samples have water contents at failure much larger than the illite series (see Table 6.2, w_f). The smectite samples have greater strengths than the illite samples, based on a comparison of $\bar{\phi}$ values (smectite: 26% higher) and $q_{fd}/\bar{\sigma}_c$ ratios (smectite: 50% higher). The smectite samples are less sensitive to remolding as opposed to the illite samples which lose considerable strength after remolding.

D) Vertical vs. Horizontal Samples

Similar differences in behavior between vertical and horizontal samples are observed for both the smectite and illite series. In both cases the horizontally oriented samples exhibited lower friction angles when compared to the vertical samples of the same material:

	Vertical	Horizontal
Illite $\bar{\phi} =$	24.6°	22.8°
Smectite $\bar{\phi} =$	36.3°	33.4°

The change in friction angle is greater for the smectite than the illite, 2.9° (8.7%) versus 1.8° (7.9%). The cohesion intercepts increase for both, 0.009 to 0.025 kg/cm² for smectite and 0.013 to 0.025 kg/cm² for the illite. The

average $q_{fd}/\bar{\sigma}_c$ ratios for the horizontal smectite samples are 15% lower than the vertical samples (1.27 versus 1.47 respectively) (Table 6.4). For the illite samples the change in orientation decreases the average $q_{fd}/\bar{\sigma}_c$ from 0.77 to 0.73 (5.5%). As with the $\bar{c}$ and $\bar{\phi}$ values, the $q_{fd}/\bar{\sigma}_c$ ratio of the smectite samples were more sensitive to sample orientation.

The strength parameters of the remolded smectite samples are effected more by sample orientation than the remolded illite samples. Remolded horizontal samples when compared to remolded vertical samples of equivalent stress exhibit lower friction angles and $q_{fd}/\bar{\sigma}_c$ ratios and higher cohesion intercepts.

E) Comparisons with Other Data

The values of $\bar{c}$ and $\bar{\phi}$ obtained in the analysis of the undisturbed samples are similar to those published by H.J. Lee and E.L. Hamilton (1974) in a report from CEL and NUC. Samples of pelagic clay (illite) and amorphous iron oxide (smectite) were obtained from sites east of MPG-1 (Locations: 30-47 N Lat.; 135-155 W Long.) aboard the R/V KANA KEOKI. $\overline{CIU}$, $\overline{CAU}$, $\overline{UU}$, and creep tests were conducted on samples from four cores obtaining the values of $\bar{c}$ and $\bar{\phi}$ listed:

	$\bar{c}$ (kg/cm ²)	$\bar{\phi}$ (degrees)
Illite	0.018-0.034	35-36
Smectite	0.035-0.042	37-38

plots of water content and shear strength were comparable to those obtained from MPG-1, with pelagic clay (illite) underlaid by a transition material below which is amorphous iron oxide (smectite).

6.4 Undrained Strength Analysis

A) Introduction

To interpret the data in terms of the goals of the testing program, the undrained shear strength, q_{fu} , was normalized with respect to consolidation pressure ($q_{fu}/\bar{\sigma}_c$). This ratio, referred to in soil mechanic texts as the c/p ratio or the ratio of undrained strength to effective overburden pressure ($S_u/\bar{\sigma}_{vo}$), is a basic relationship used to compare strengths of different materials or the same material under different physical conditions.

As an example, Skempton and Bjerrum found a correlation of increasing c/p ratio with increasing plasticity index for normally consolidated marine clays. Figure 6.92 is a plot of this relationship for several marine clays (Meyerhof, 1979 and modified by Silva, 1979). Values of c/p for the sediments analyzed in this study are also plotted. As illustrated, the smectite clays plot below the line at high values of plasticity index while the illite clays plot above the line at considerable lower values of plasticity index.

The theoretical $q_{fu}/\bar{\sigma}_c$ ratio was calculated on the basis of four different assumptions and compared with the measured values. The five values of the $q_{fu}/\bar{\sigma}_c$ ratio for

each sample, values of consolidation stress and A-factor, and the equations used to calculate $q_{fu}/\bar{\sigma}_c$ are listed in Table 6.4 for the samples which were tested as normally consolidated. The first listing is the actual experimental value of $q_{fu}/\bar{\sigma}_c$ obtained for each individual sample. The first method of calculation (#1) derives the value of $q_{fu}/\bar{\sigma}_c$ from the formula:

$$q_{fu}/\bar{\sigma}_c = \frac{\sin \bar{\phi} + (\bar{c}/\bar{\sigma}_c) \cos \bar{\phi}}{1 + (2A_f - 1) \sin \bar{\phi}} \quad (6-3)$$

where $q_{fu}/\bar{\sigma}_c$ is a function of the cohesion intercept $\bar{c}$, the internal angle of friction $\bar{\phi}$, and the A-factor of the sample at failure A_f assuming K_0 is equal to unity. The ratios of the actual and calculated values (method #1) should be approximately the same. Any variation in the values is the result of the A-factor or the method used to calculate $\bar{c}$ and $\bar{\phi}$. The majority of the $q_{fu}/\bar{\sigma}_c$ values from these two methods are similar if not identical. The second method (#2) uses the above equation but assumes the cohesion intercept to be zero, which gives an indication of the degree to which the cohesion intercept contributes to the shear strength. The cohesion contributes to approximately 2% of the undrained strength for the smectite samples and 9% for the illite samples. The third method (#3) gives values of $q_{fu}/\bar{\sigma}_c$ for drained conditions i.e. no pore pressure. This indicates the combined effect of the two parameters $\bar{c}$ and $\bar{\phi}$ on the strength of the samples (see Section 6.3). From comparisons of the drained and undrained

strengths, the drained strengths of the smectite samples are 220% larger and 163% larger for the illite samples. The fourth method (#4) indicates the effect that anisotropy would have on the undrained strengths if the value of K_0 were equal to one-half. Assuming a K_0 of one-half as compared to unity reduces the undrained strengths of the smectite by 12% and the illite by 6%.

Henkel (1960) states that there is a unique relationship between deviator stress at failure and water content at failure for tests performed on remolded normally consolidated samples. Whitman (1960) indicates that shear strength is a function of many different variables including: stress history, soil structure, cementation, interparticle bonding, intermediate stress $\bar{\sigma}_2$, and temperature. It is probable that one or two variables will control changes in several others. The effect of structure on the shear strength of cohesive soils has been studied by many investigators. It is generally accepted that an undisturbed clay has a natural structure and interparticle bonds which upon disturbance and remolding are irreversibly destroyed. In summary it must be emphasized that the shear strength of any clay is not a function of consolidation stress alone. Jurgenson (1934) states that the $q_{fu}/\bar{\sigma}_c$ ratio for an undisturbed clay will be less than a remolded clay of the same material, due to the higher water contents observed in the undisturbed sample at failure. The difference in water content indicates a dissimilar struc-

ture between the two samples, the sample with the higher water content having a greater void ratio. If the undrained strength of undisturbed and remolded samples is to be compared, they must be compared under the same conditions, i.e. the same water content (void ratio). Comparing the undisturbed samples LL-44 GPC-3 and MARA-02 GC-04 to remolded samples of the same material and simulated depth, PS-18 and PI-2 (see Table 6.2), the difference in initial water content (w_i) indicates a change in structure due to remolding.

B) Undisturbed vs. Remolded Samples

Comparing the $q_{fu}/\bar{\sigma}_c$ ratios (see Table 6.4) of the remolded and undisturbed samples, the ratios (method one) are all similar excluding the samples PS-18 #1 and #2. However, the ratio of $q_{fu}/\bar{\sigma}_c$ for undisturbed samples should be less than that of the remolded samples if Jurgenson's hypothesis is correct since the water contents are considerably higher. The high $q_{fu}/\bar{\sigma}_c$ ratios of the undisturbed samples could be caused by physio-chemical factors in the samples which are destroyed after remolding. As a corollary to the statement by Jurgenson, if a remolded and an undisturbed sample have the same $q_{fu}/\bar{\sigma}_c$ ratio and the undisturbed sample has a higher water content (void ratio), then the undisturbed sample is of greater strength. Considering the higher water content of the undisturbed samples and the similar strength ratios, the undisturbed smectite samples have a greater strength. For example,

comparing the LL-44 sample series to PS-18 Vertical, the respective $q_{fu}/\bar{\sigma}_c$ ratios (method two) are 0.44 versus 0.47. The failure water contents for the same series are 154% for LL-44 versus 109% for PS-18. To reduce the water contents of the LL-44 samples to values similar to the PS-18 samples would require a considerably higher consolidation stress, resulting in a higher shear strength. The consolidation stress required to decrease the water content of the LL-44 samples can be approximated assuming a linear relationship between water content and the log of the consolidating stress. The change in consolidation stress required is expressed as:

$$\Delta\sigma = \sigma_o \left(10^{\frac{w_o - w'}{C_c}} - 1 \right) \quad (6-4)$$

where σ_o is the original consolidation stress, w_o is the original water content, w' is the water content desired and C_c' is the value of C_c determined in a one-dimensional consolidation test divided by the specific gravity of the sediment. The term C_c is the slope of the virgin consolidated portion of the e -log σ curve. Using Equation 6-4, the increased consolidation stress was 2.5 times the original stress. Therefore, the undisturbed samples are slightly less than 2.5 times stronger than the remolded samples at the same water content.

The undisturbed MARA illite samples have strengths greater than the remolded samples, neglecting the VEMA samples. The low strength values of the VEMA samples were

discussed in Section 6.3 and therefore these samples will be omitted from the undrained analysis. Comparing the undisturbed sample MA-02 GC-04 238-245 cm with the remolded sample PI-2 #1, both were consolidated to equal pressures, however the final water content of the undisturbed sample is 29% greater (88.7% versus 68.9%) (Table 6.2). The corresponding $q_{fu}/\bar{\sigma}_c$ ratios are 0.375 (undisturbed) versus 0.330 (remolded). Using Equation 6-4, the stress required to decrease the water content of MA-02 GC-04 238-245 to that of PI-2 #1 is 2.5 times the original consolidating stress. Comparing the MARA 200 cm samples ($q_{fu}/\bar{\sigma}_c = 0.37$) to the PI-18 samples ($q_{fu}/\bar{\sigma}_c = 0.45$), again the differences in water content must be considered, $w = 87\%$ (average value) for the MARA sample and $w = 64\%$ (average value) for PI-18 samples. Again, using Equation 6-4, the consolidation stress required to obtain equal water contents is approximately three times greater. This results in undrained strengths of 1.09 kg/cm^2 (MARA-02 200 cm) versus 0.45 kg/cm^2 (PI-18) when consolidated to 1.0 kg/cm^2 .

Undisturbed samples have similar and/or lower $q_{fu}/\bar{\sigma}_c$ ratios than the remolded samples when compared at the same consolidation stresses (with different water content). At equal water contents, the undisturbed samples would have much greater shear strengths.

C) Smectite vs. Illite Samples

In comparing the smectite and illite, an overall tendency for larger $q_{fu}/\bar{\sigma}_c$ ratios is exhibited by the

smectite samples (see Table 6.4). The only inconsistent values are for the PS-18 Vertical and the PI-18 Vertical samples. Again the influence of water content at failure must be considered in the analysis since the water contents at failure for the smectites are much larger than those of the illite samples. Taking both factors into consideration, the $q_{fu}/\bar{\sigma}_c$ ratio and the water content at failure, the smectite samples are judged to be stronger than the illite samples.

D) Vertical vs. Horizontal Samples

Comparisons of the vertical and horizontal oriented samples are difficult due to the inconsistencies of the PS-18 Vertical sample values. Using the average value of $q_{fu}/\bar{\sigma}_c$ for these samples and comparing them to the PS-18 Horizontal samples, the strengths are identical. Comparing the samples at the same water content, the horizontal samples are 50% stronger. The vertical illite samples exhibit a higher $q_{fu}/\bar{\sigma}_c$ ratio (21% higher) than the horizontal samples. The available data makes further comparisons impossible. It should be stated that the orientation has an effect upon the strength parameters.

E) Undrained Strength vs. Depth

To illustrate how triaxial parameters can be used to predict shear strength, the following analysis is provided. Lee (1974) outlined a method to determine undrained strength with depth in a core. The triaxial fail-

ure parameters $\bar{c}$, $\bar{\phi}$, and A_f obtained from surficial sedimentary material were used to extrapolate the undrained strength in the deeper sediments. The formula used (same as method four of Table 6-4):

$$q_{fu}/\bar{\sigma}_c = \frac{[K_o + A_f (1-K_o)] \sin \bar{\phi} + (\bar{c}/\bar{\sigma}_c \cos \bar{\phi})}{1 + (2A_f - 1) \sin \bar{\phi}} \quad (6-5)$$

takes into account the difference in horizontal and vertical in-situ stresses by including the term K_o , the at-rest earth pressure (K_o is the ratio of horizontal to vertical effective stress).

This method was used to determine the undrained strength versus depth using LL-44 GPC-3 material properties. The strength profile generated served as a check on the vane shear profile. Values of water content (476 data points) and specific gravity (70 data points) were entered into an interpolation program which calculated values of water content and specific gravity at every 10 centimeters to a depth of 24.4 meters. The submerged unit weight was then calculated and integrated with depth to produce a profile of effective overburden stress versus depth.

As stated in Section 2.7, three distinct materials were observed in the GPC-3 core, an upper layer of illite material, a transition layer, and the smectite. The ratio of $q_{fu}/\bar{\sigma}_c$ was determined for each of the three layers using the values indicated:

	$\bar{c}$	$\bar{\phi}$	A_f	K_O
Illite	0.00	34.0	1.00	0.55
Transition	0.050	33.0	0.80	0.55
Smectite	0.016	35.5	0.80	0.60

No K_O consolidation tests were run on these sediments, therefore only approximate values of K_O were available. However small changes in the value of K_O should not drastically change the ratio of $q_{fu}/\bar{\sigma}_c$.

Using the three ratios of $q_{fu}/\bar{\sigma}_c$ and the profile of $\bar{\sigma}_c$ (effective overburden stress) with depth, the undrained strength was calculated at every 10 centimeters to a depth of 24.4 meters. These values have been plotted with the values of vane shear strength (see Section 2.4) in Figure 6.93. The fit of the derived values to the measured values of vane shear strength is excellent. The comparison of strengths indicate several important points. One, the factor of K_O should be included into the strength calculations. Second, the comparison serves as a check on the vane shear measurements. Third, the quality (lack of disturbance) of the triaxial samples is indicated. Disturbed samples would produce lower strength values in comparison to the vane shear values. The vane shear tests were run aboard ship and therefore received less disturbance due to handling, shipping and sample preparation.

Also included in Figure 6.93 is a "best fit" plot of undrained strength versus depth. The equation of this plot is:

$$S_u(z) = 0.2 \ln (1+0.1149z) \quad (6-6)$$

where z is the depth in meters and $S_u(z)$ is the undrained strength in kg/cm^2 . This formula (Burchett, 1979) was obtained by fitting a curve to the vane shear values. The previous method uses the actual material failure parameters to obtain the strength profile and should be considered a more exact method of prediction. The log equation tends to underestimate the shear strength below 15 meters.

6.5 Hyperbolic Data Analysis

A) Introduction

The hyperbolic analysis discussed in Chapter 5 was applied to the test data from each series of samples and the results are discussed in this section. The parameters of initial tangent modulus E_i , ultimate deviator stress $(\sigma_1 - \sigma_3)_{ult}$ and deviator stress ratio R_f are listed in Table 6.2. Values of the modulus number K and modulus exponent n are listed in Table 6.3 for each series. The data is discussed in order of presentation of the Figures at the end of this chapter.

B) Data Analysis

When the stress-strain curves of the LL-44 GPC-3 samples (Figure 6.1) were transposed (plotted as $\epsilon/(\sigma_1 - \sigma_3)$ vs. ϵ), negative values of intercept were obtained. This resulted in a negative value of E_i and therefore an invalid hyperbolic stress-strain curve. These samples had strain softening effects, causing the values of $\epsilon/(\sigma_1 - \sigma_3)$,

after failure, to increase more than samples which showed constant stress after failure. This results in a steeper least squares fit line and thus a negative intercept. A possible correction could be made by using only two points before failure for the transposed curve. However, it was decided to use the same method of analysis throughout the test program. The slope of the transposed curve was still used to determine an ultimate deviator stress.

For the PS-9 Vertical samples, Figure 6.15 is a plot of the hyperbolic stress-strain curve (solid line) superimposed over the experimental points (+'s). The fit of the hyperbolic curve to the experimental points is excellent except for sample #4, which exhibited fluctuations in the experimental points. As samples #1 and #2 yielded, there was a tendency by the hyperbolic analysis to overestimate the experimental points. The initial tangent modulus (E_1) and the ultimate deviator stress $(\sigma_1 - \sigma_3)_{ult}$ should increase with increasing confining stress (Wong and Duncan, 1974). Thus, both E_1 and $(\sigma_1 - \sigma_3)_{ult}$ (Table 6.2) are in error for sample #4 (see Section 6.2).

Figure 6.16 is a plot of tangent Young's modulus versus deviator stress for the four samples of this series. The hyperbolic tangent moduli are represented by the solid line, while the experimental values are represented by the symbols. In the analysis of the tangent moduli the strength at failure is determined by a Mohr-Coulomb equation. If the stress-path of an overconsolidated sample and

its failure point lies above the failure envelope, the Mohr-Coulomb analysis will underestimate the true strength of the sample. Therefore, the hyperbolic tangent moduli for sample #4 are lower than the experimental values, due to inconsistent behavior of sample #4. For samples, #1, #2 and #3 the hyperbolic moduli match the experimental values very well. This indicates the ability of the hyperbolic model to simulate the stress-strain behavior of this sediment.

Figure 6.24 illustrates the fit of the hyperbolic curve to the PS-18 Vertical sample stress-strain curves. The strain softening shown by sample #3 resulted in a negative value of E_i . As observed in the PS-9 samples, the hyperbolic curve overestimates the experimental values during yielding for samples #1 and #4. The hyperbolic curve for sample #5 is indistinguishable from the experimental curve.

The plots of tangent modulus versus deviator stress (Figure 6.25) for the PS-18 Vertical samples show very little scatter in the experimental points. The hyperbolic model overestimates the moduli for samples #1 and #4 and compares satisfactorily with the points of samples #3 and #5.

The hyperbolic stress-strain curves for the PS-18 Horizontal samples (Figure 6.33) again exhibit a very good fit, except for sample #8. Compared to the other samples in the series, #8 has an E_i value too high and a $(\sigma_1 - \sigma_3)_{ult}$

value that is too low. The inconsistent values are caused by the strain softening of the sample, which decreases the intercept of the transposed curve resulting in a high initial modulus. This also causes a large discrepancy during yielding between the hyperbolic and experimental points. Sample #10 has a low value of E_i and a high ultimate deviator stress, which could be caused by the shallower slope of the elastic portion of the stress-strain curve.

The curves of tangent modulus versus deviator stress for the PS-18 Horizontal sample are shown in Figure 6.34. Samples #7 and #8 exhibit hyperbolic moduli less than the experimental values while the hyperbolic curve is slightly above the experimental values for samples #9 and #10. From observation of the stress-strain curves of samples #7 and #8, the hyperbolic moduli should actually be greater than the experimental values. The low values could be caused by the assumed values of parameters K and n , which determine a value of E_i in the formula for tangent Young's modulus.

The fit of the hyperbolic stress-strain curve to the experimental curve is fairly good for the 200 cm MARA samples (Figure 6.42). The only discrepancies occur during yielding of samples 228-235 cm and 238-245 cm, which are attributed to the steep elastic portion of the experimental curves. This factor also results in an erroneously high value of E_i for the sample 238-245 cm (Table 6.2).

Inconsistent values of E_i effect parameters of K and

n, which result in erroneous values of tangent modulus. To minimize this effect, the parameters of K and n were determined using values of E_i and σ_3 from both series of MARA samples. This allowed the elimination of inconsistent numbers and thus more representative values were obtained. The modulus exponent n (Table 6.3) when compared to typical values ($n < 1.0$) presented by Wong and Duncan (1974) is high. Figure 6.43 illustrates that the tangent moduli determined by the hyperbolic analysis were greater at low deviator stresses than the experimental moduli for samples 228-235 cm and 238-245 cm.

The variations in the experimental stress-strain curves of the 100 cm MARA samples makes comparisons of the hyperbolic and experimental points difficult. In Figure 6.51, it appears that the hyperbolic curve averages the variations and produces a "best fit" type plot (ex. MA-02 GC-04 76-84 cm). A negative initial modulus was determined for sample 86-94 cm and its hyperbolic curve is not presented. The stress paths of samples 86-94 cm and 95-105 cm did not reach the failure envelope due to equipment problems (Figure 6.46). This causes the hyperbolic analysis to overestimate the deviator stress at failure, resulting in tangent moduli which are too large. Sample 105-115 cm has an excellent fit of hyperbolic to experimental values (Figure 6.52). The experimental moduli plot well above the hyperbolic curve for sample 76-84 cm, due to the variations in the experimental stress-strain curve.

Figure 6.59 illustrates an excellent fit of the hyperbolic to experimental stress-strain values for the VEMA 200 cm series. The excellent fit is attributed to the strain hardening of these samples which is better approximated by a hyperbolic shape. As with the two MARA series, the values of E_i and σ_3 were combined for the two VEMA series to determine the parameters of K and n . The fit of experimental to hyperbolic tangent moduli (Figure 6.60) for this series is considered superior to all others.

The deeper VEMA samples (400 cm series) show a somewhat poorer agreement of experimental to hyperbolic stress-strain values (Figure 6.65) in comparison to the VEMA 200 cm series. The hyperbolic curve cannot model the strain softening observed in the sample 523-533 cm. The high E_i value of sample 499-509 cm causes the hyperbolic curve to overestimate the experimental values during yielding. Figure 6.66 shows an excellent correlation between hyperbolic and experimental tangent moduli, especially for sample 483-493 cm. The fit of the tangent moduli curves is much better than the fit of the stress-strain curves.

The hyperbolic stress-strain curves of the PI-18 Vertical samples (Figure 6.72) exhibit varying degrees of correlation with the experimental curves. Sample #1 has a high value of initial tangent modulus (Table 6.2), as a result of the strain softening. Sample #2 has a poor fit of hyperbolic to experimental points due to the low initial modulus, while samples #3 and #4 have excellent fits of

calculated to experimental values. Excluding sample #4, the hyperbolic tangent moduli exhibit a poor correspondence to the experimental points (Figure 6.73). Sample #3, which had an excellent fit of stress-strain curves, has experimental moduli above the hyperbolic curve. However, the stress path of this sample (Figure 6.69) plots above the failure envelope, resulting in the high experimental tangent moduli. The inferior match shown by sample #1 and #2 is due to the general shape of the experimental stress-strain curve.

The hyperbolic curves of the PI-2 Vertical (Figures 6.81 and 6.82) and Horizontal (Figures 6.90 and 6.91) samples are of poor quality due to the variations in the experimental points and therefore will not be discussed. In tests which have such variations, the hyperbolic analysis could be used to obtain an approximate stress-strain curve.

C) Summary

The majority of samples exhibit an excellent correlation between the hyperbolic and experimental stress-strain curves. The exceptions are the samples that exhibit strain softening, which the hyperbolic analysis cannot model. The hyperbolic tangent moduli, based on Mohr-Coulomb theory, are affected by several parameters, which are derived from the experimental data. The major advantage of the hyperbolic analysis is its simplicity. The parameters required to determine tangent Young's modulus can be obtained from unconfined compression, drained or undrained triaxial tests.

6.6 Lateral Strain and Poisson's Ratio

A) Introduction

The procedures used to measure lateral strain were discussed in Chapter 4. Included in the chapter was an explanation of the analysis used to calculate Poisson's ratio from the measured lateral and vertical strains. The following section is a qualitative analysis of the measured lateral strains and calculated Poisson's ratios as determined in this test program. The results are discussed separately for the smectite and illite samples for ease of presentation and values of lateral strain and Poisson's ratio are listed in Table 6.5 at several vertical strains.

B) Calculated Poisson's Ratio-Smectite

The four smectite series each had two samples from which lateral strains were measured. Excluding the PS-9 series (Figure 6.13) similar lateral strains were measured (Table 6.5) in the three series: LL-44 GPC-3 1800 cm (Figure 6.6), PS-18 Vertical (Figure 6.21) and PS-18 Horizontal (Figure 6.31). The agreement of the values indicates the ability of the lateral strain gauge to accurately measure lateral deformation. The different strains of the two PS-9 Vertical samples could be a function of stress history, both samples were overconsolidated. Sample #4, for this series, exhibited inconsistent behavior when compared to the other samples in the series. Sample PS-9 #3 has a lateral strain of approximately 15% at the end of the test, similar to that observed in the sample PS-18 Vertical #1. Samples

PS-18 Vertical #1 and #3 (normally and overconsolidated, respectively) would suggest that lateral strain is not a function of stress history. However, the analysis of the illite samples and the PS-9 Vertical samples indicate stress history does influence the lateral strains.

Comparing the calculated Poisson's ratios for the four series, it is observed that small differences in lateral strains are amplified in the plots of Poisson's ratio for the series LL-44 GPC-3 1800 cm (between 1 and 7% vertical strain, Figure 6.1), PS-18 Vertical (Figure 6.23) and PS-18 Horizontal (Figure 6.32). The PS-9 Vertical samples #3 and #4 (Figure 6.14) show large discrepancies due to the differences in lateral strains. Excluding the LL-44 samples, all the samples had values of Poisson's ratio less than 0.5 throughout the range of vertical strains (Table 6.5). Poisson's ratio exceeds 0.5 for the LL-44 samples at 10% vertical strain (Figure 6.7 and Table 6.5). The large Poisson's ratios are due to an incorrect assumption of the deformational shape of the sample.

For the majority of samples, Poisson's ratio peaks at very low strains (<2%) and then gradually increases with increasing vertical strain. This peak corresponds with the beginning of yielding for some of the samples (ex. PS-18 Vertical #1 and #3, Figure 6.23 and PS-9 Vertical #3 and #4, Figure 6.14).

C) Calculated Poisson's Ratio-Illite

The lateral strains measured for the four illite se-

ries, MA-02 GC-04 200 cm (Figure 6.40), MA-02 GC-04 1800 cm (Figure 6.49), PI-2 Vertical (Figure 6.79) and PI-2 Horizontal (Figure 6.88) show nearly identical values for several of the samples (Table 6.5). The normally consolidated MARA samples (228-235 cm, 238-245 cm, 105-115 cm and 86-94 cm) all exhibit near identical lateral strains at 10% vertical strain (8.9%, 9.8%, 8% and 9%, respectively). The overconsolidated samples (216-224 cm and 76-84 cm) both have smaller strains than the normally consolidated samples (Figures 6.40 and 6.49). This behavior is also observed in the PI-2 Horizontal samples (Figure 6.88). Sample PI-2 Horizontal #8 was consolidated to a higher stress than sample #9 and also has larger lateral strains. The lateral strains at 10% vertical strain for samples PI-2 Horizontal #6 and #7 (both normally consolidated) approximate those of the undisturbed normally consolidated samples (Table 6.5). The overconsolidated samples PI-2 Vertical #4 and #5 have lateral strains at 10% vertical strain similar to those observed for samples PI-2 Horizontal #8 and #9 (Table 6.5).

Comparing the Poisson's ratios of the four series MA-02 GC-04 200 cm (Figure 6.41) and 100 cm (Figure 6.50), PI-2 Vertical (Figure 6.80) and PI-2 Horizontal (Figure 6.89), several samples have values greater than 0.5 (see also Table 6.5). The four normally consolidated MARA samples (228-235 cm, 238-245 cm, 105-115 cm, and 86-94 cm) all exceed 0.5 before reaching 8% vertical strain. This is also true of the normally consolidated samples PI-2 Horizontal

#6 and #7. All the overconsolidated illite samples have Poisson's ratios less than one-half (ex. PI-2 Vertical #4 and #5 and PI-2 Horizontal #8 and #9) (see Table 6.5). As observed with the smectite samples, the majority of samples show a peak in Poisson's ratio (between 0.2 and 0.45) at low vertical strains (less than 3%), then a gradual increase in Poisson's ratio with increasing vertical strain. Although the peak does not occur at yield for all the samples, there appears to be a trend for the peak to occur in the proximity of the yield strain.

The calculated Poisson's ratio is a function of the deformational shape assumed. Evidently, the factor of $2/3$ does not adequately correct the calculated values of Poisson's ratio (see Section 4.5). The apparent correlation between stress history and Poisson's ratio observed in the illite samples is a result of the influence of stress history on the deformational shape of the samples.

D) Area Correction

Using the lateral deformation measurements, the true sample area was calculated and compared to the standard corrected area used in triaxial test calculations. The standard area correction calculates an average cross-sectional area and at large axial strains produces erroneous values. Bishop and Henkel (1962) suggest other equipment, such as a torsional shear apparatus, be used to determine shear strength at large strains to avoid the problems of calculating a sample cross-sectional area. The true area

was calculated from the measured diameter at mid-height (the $2/3$ factor used to calculate Poisson's ratio was not included). Figures 6.94 to 6.97 show typical comparisons, where the solid lines represent the standard corrected area and the dotted lines represent the true area. For the majority of samples, the standard correction underestimates the true area of the samples at the mid-plane.

Roscoe et. al, (1963) measured radial strains optically on drained extension and compression triaxial tests on sand samples. At an average axial strain of 10%, the standard area correction was 16% as compared to the "true" area correction of 33%. However, they suggest using the standard correction, recognizing an overestimate of the true stress results.

Figures 6.98 to 6.105 compare the stress-strain curves for several samples using the areas by both methods, where the solid lines represent the standard correction and the dotted lines the true area. Comparing the stress-strain curves from the two methods, for the majority of samples variations are not observed for vertical strains less than 5%. The stresses calculated by the two methods diverge after 5% vertical strain, with large discrepancies at the end of the test, 19.8% vertical strain.

A few samples, PS-9 #4, PI-2 #9 and all the PI-2 Vertical and PS-18 Horizontal samples had true areas less than the standard corrected areas. No reason for the smaller true areas can be given.

Lateral deformation measurements indicate the standard area correction is in error for vertical strains greater than 5%. The majority of samples have true areas greater than the standard area, resulting in deviator stresses after 5% axial strain that are in error.

E) Conclusions

Values of Poisson's ratio calculated from direct measurement of lateral deformation in an undrained test are greatly influenced by sample deformational shape. Assuming a parabolic shape, the ratio of lateral to vertical strain was multiplied by a correction factor of $2/3$. However, values of Poisson's ratio greater than one-half were still obtained. Assuming small strain theory applies, under no volume change conditions (undrained) Poisson's ratio is one-half. Poisson's ratio is an elastic parameter and as such should only be determined in the linear-elastic range of the stress-strain curve.

For a majority of samples, Poisson's ratio at small vertical strains exhibited peaks between 0.3 and 0.45 and then continued to increase with further vertical strain. The strain at which the peaks occurred corresponded to the yield strain of the samples. The values of Poisson's ratio calculated in this range are sensitive to small fluctuations in lateral strain. It is debatable whether or not to include the correction factor of $2/3$ at these low vertical strains. Considering all the possible sources of error, the direct measurement method of determining Poisson's ra-

tio appears to be inadequate. Drained triaxial tests would be a more accurate method of determining Poisson's ratio.

Lateral deformation measurements were used to calculate the true area of triaxial samples at mid-height. For a majority of samples, the standard area correction, used to calculate the average cross-sectional area during uniaxial compression, diverges from the true area after 5% vertical strain. The deviator stress calculated using the true area represents a lower bound value. To simplify calculations, Roscoe et. al, (1963) suggest the standard area correction be used, recognizing the stress calculated will be an overestimate.

Table 6.1

Material Properties

Sample ID	Grain Size				W _L %	W _P %	Activity	I _L	G _S
	Sand %	Silt %	Clay %	Median microns					
LL-44 GPC-3									
1875	0.7	42.3	57.0	1.5	205	96	1.9	1.0	2.72
PS-9	0.6	23.0	76.4	0.8	220	95	1.6	0.6	2.87
PS-18	0.3	26.6	73.1	0.8	227	92	1.8	0.3	3.00
MA-02 GC-04									
76 - 84	Trace	33.6	66.4	1.1	94	38	0.8	1.4	2.72
86 - 94	"	34.8	65.1	1.3	83	36	0.7	1.5	2.71
95 - 105	"	30.3	69.7	1.1	86	44	0.6	1.8	2.97
105 - 115	"	33.9	66.0	0.9	88	37	0.7	1.5	2.73
215 - 225	"	29.9	70.1	1.1	99	48	0.7	1.1	3.00
226 - 236	"	29.7	70.3	1.0	87	46	0.6	1.5	3.00
237 - 247	"	30.0	70.0	1.0	100	51	0.7	1.2	3.02

(Continued)

Table 6.1 (continued)

Sample ID	Grain Size				W _L %	W _P %	Activity	I _L	G _s
	Sand %	Silt %	Clay %	Median microns					
V-32 PC-115									
235 - 245	0.1	33.5	66.5	1.2	87	45	0.6	1.4	2.70
475 - 545	0.1	28.7	71.1	0.9	112	45	0.9	0.9	2.70
535 - 545	0.3	25.3	74.5	0.8	125	50	1.0	0.9	2.69
PI-2	0.3	33.4	66.3	1.0	83	40	0.7	1.2	2.84
PI-18	0.3	33.6	67.1	1.0	85	52	0.5	0.5	2.86

Table 6.2
Triaxial Failure Parameters

Sample I.D.	σ_3 kg/cm ²	ϵ_f %	ν %	$(\sigma_1 - \sigma_3)_f$ kg/cm ²	σ_f kg/cm ²	$\bar{\sigma}_1/\bar{\sigma}_3$	A_f	q_f kg/cm ²	$\bar{p}_f$ kg/cm ²	w_i %	w_f %	$(\sigma_1 - \sigma_3)_{ult}$	E_i kg/cm ²	R_f
LL-44 GPC-3														
1859 - 1868	1.478	6.8	19.6	1.316	1.024	3.901	0.778	0.658	1.111	203.5	156.5	1.208		1.0
1873 - 1883	1.900	8.6	22.8	1.620	1.137	3.879	0.825	0.810	1.173	191.6	152.1	1.464		1.0
1845 - 1857	2.252	9.8	38.3	2.570	1.654	5.301	0.643	1.285	1.883	201.9	128.9	2.409		1.0
PS-9 Vertical														
#4	0.048	2.0	2.1	0.192	0.027	10.411	0.143	0.096	0.116	175.3	163.0	0.267	48.81	0.72
#3	0.141	10.0	1.8	0.207	0.091	5.093	0.451	0.101	0.151	166.4	153.0	0.210	25.59	0.96
#1	0.500	8.8	9.5	0.444	0.370	4.436	0.835	0.222	0.351	176.7	129.4	0.533	36.04	0.83
#2	0.051	15.8	19.3	0.744	0.625	4.280	0.841	0.372	0.598	170.2	121.6	0.799	79.59	0.96
PS-18 Vertical														
#5	0.097	5.0	-2.7	0.266	0.036	5.330	0.136	0.133	0.194	128.4	131.2	0.284	46.00	0.94
#3	0.302	3.2	2.0	0.450	0.159	4.160	0.353	0.225	0.368	126.2	129.6	0.393		1.0
#1	0.851	12.4	9.4	0.866	0.564	4.013	0.564	0.433	0.721	132.7	111.5	0.862	184.05	1.0
#4	1.400	12.8	12.8	1.210	0.995	3.983	0.995	1.605	1.010	127.7	106.8	1.240	208.5	0.98
PS-18 Horizontal														
#9	0.097	10.2	-2.4	0.300	0.003	4.169	0.009	0.150	0.245	127.8	112.2	0.315	32.53	0.95
#8	0.284	10.4	1.3	0.464	0.146	4.365	0.315	0.232	0.369	133.6	130.9	0.437	359.21	1.00
#10	0.795	9.8	8.9	0.716	0.520	3.606	0.727	0.358	0.633	126.4	122.5	0.841	48.28	0.85
#7	1.368	16.0	15.4	1.302	0.882	3.682	0.677	0.651	1.137	132.9	117.3	1.307	150.49	0.99
MA-02 GC-04														
216 - 224	0.113	6.8	4.4	0.132	0.074	4.378	0.561	0.066	0.105	105.8	94.6	0.140	26.0	0.94
238 - 245	0.563	7.2	17.3	0.422	0.405	3.679	0.958	0.211	0.369	107.9	88.7	0.411	614.7	1.0
228 - 235	0.844	10.0	19.4	0.616	0.613	3.655	0.996	0.308	0.539	108.3	76.6	0.611	299.8	1.0
MA-02 GC-04														
76 - 84	0.065	2.4	1.1	0.182	0.038	7.773	0.210	0.091	0.118	113.6	103.9	0.207	11.9	0.88
86 - 94	0.358	5.4	12.6	0.248	0.159	2.245	0.641	0.124	0.323	105.4	96.8	0.231		1.0
95 - 105	0.429	9.0	13.8	0.210	0.220	2.008	1.041	0.105	0.315	121.0	95.4	0.217	47.3	0.97
105 - 115	0.704	7.8	19.9	0.540	0.529	4.085	0.978	0.270	0.445	115.3	95.7	0.538	209.9	1.0

Table 6.2 (continued)

Sample I.D.	kg/cm ²	ϵ_f	ϵ_v	$(\sigma_1 - \sigma_3)_f$	kg/cm ²	$\bar{\sigma}_1 / \bar{\sigma}_3$	A_f	kg/cm ²	kg/cm ²	w_l %	w_f %	$(\sigma_1 - \sigma_3)_{ult}$	E_l kg/cm ²	R_f
V-32 PC-115														
206 - 216	0.563	10.2	13.7	0.386	0.347	2.795	0.898	0.193	0.409	99.9	74.8	0.425	67.25	0.91
219 - 229	1.126	10.2	20.2	0.692	0.698	2.622	1.009	0.346	0.773	102.5	74.5	0.755	121.04	0.92
232 - 242	1.688	10.4	24.8	0.996	0.995	2.436	0.999	0.498	1.192	105.6	67.8	1.072	188.44	0.93
												0.361	36.27	0.93
V-32 PC-115														
483 - 493	0.141	9.6	7.5	0.334	0.047	4.570	0.141	0.167	0.261	106.1	95.8			
499 - 509	0.844	10.9	12.7	0.716	0.541	3.363	0.756	0.358	0.661	111.3	84.4	0.361	36.21	0.93
523 - 533	1.689	11.5	26.0	1.272	1.077	3.081	0.847	0.636	1.248	116.8	-	0.736	245.51	0.97
PI-18 Vertical														
#4	0.162	6.0	-3.1	0.808	-0.102	4.071	-0.125	0.404	0.688	71.2	67.3	0.959	87.0	0.84
#3	0.400	10.2	0.6	0.822	0.210	5.325	0.255	0.411	0.601	70.4	67.8	0.829	229.7	0.99
#1	1.011	6.8	6.1	1.062	0.666	4.078	0.628	0.531	0.876	67.3	66.9	1.018	731.6	1.0
#2	1.583	8.6	8.5	1.568	0.991	3.649	0.629	0.784	1.376	70.4	60.1	1.857	153.8	0.84
PI-2 Vertical														
#4	0.026	2.6	0.5	0.666	0.020	12.131	0.294	0.033	0.040	90.0	87.2	0.062	16.6	1.0
#5	0.054	2.6	0.9	0.124	0.038	9.251	0.309	0.062	0.077	90.3	86.1	0.156	12.0	0.79
#2	0.294	1.8	12.7	0.232	0.161	2.750	0.694	0.116	0.249	91.9	74.4	0.226	97.7	1.0
#1	0.563	2.2	16.2	0.372	0.331	2.603	0.892	0.186	0.417	87.3	68.9	0.337		1.0
PI-2 Horizontal														
#9	0.030	4.4	-0.4	0.064	0.026	17.567	0.411	0.032	0.036	86.7	88.0	0.097	3.7	0.66
#8	0.053	4.2	0.4	0.144	0.043	15.825	0.296	0.072	0.082	90.4	83.2	0.176	14.5	0.82
#7	0.291	6.2	6.3	0.186	0.204	3.115	1.101	0.093	0.180	90.8	75.9	0.168		1.0
#6	0.563	3.6	20.3	0.282	0.400	2.724	1.419	0.141	0.304	90.4	67.8	0.332	43.5	0.85

Table 6.3

Triaxial Test Results

<u>Sample Series</u>	$\bar{a}$ <u>kg/cm²</u>	$\bar{\alpha}^0$ <u>Degrees</u>	$\bar{c}$ <u>kg/cm²</u>	$\bar{\phi}^0$ <u>Degrees</u>	K <u>kg/cm²</u>	<u>n</u>
LL-44 GPC-3 1800 cm	0.013	30.1	0.016	35.5		
PS-9 Vertical	0.009	31.2	0.011	37.3	70.80	0.56
PS-18 Vertical	0.007	30.6	0.009	36.3	181.97	0.59
PS-18 Horizontal	0.021	28.8	0.025	33.4	86.86	0.47
MARA-02 GC-04						
200 cm	0	29.7	0.0	34.8	257.04	1.14
100 cm	0.026	28.5	0.031	33.0	257.04	1.14
VEMA-32 PC-115						
200 cm	0.038	21.3	0.041	22.9	117.85	0.66
400 cm	0.045	25.3	0.051	28.3	117.85	0.66
PI-18 Vertical	0.087	26.8	0.101	30.4	133.64	0.25
PI-2 Vertical	0.012	22.6	0.013	24.6	240.99	0.74
PI-2 Horizontal	0.023	21.2	0.025	22.8	70.15	0.84

Table 6.4
Normalized Strengths

Sample I.D.	$\bar{\sigma}_c$ (kg/cm ²)	Af	Actual Values	Method of Determination			
				1	2	3	4
LL-44 GPC-3							
1859 - 1868	1.478	0.778	0.445	0.446	0.439	1.406	0.397
1873 - 1883	1.900	0.825	0.427	0.427	0.422	1.401	0.390
PS-9 Vert.							
#1	0.500	0.835	0.444	0.443	0.431	1.582	0.408
#2	0.851	0.841	0.437	0.436	0.429	1.564	0.402
PS-18 Vert.							
#1	0.851	0.564	0.509	0.558	0.550	1.472	0.438
#4	1.400	0.995	0.432	0.377	0.373	1.464	0.376
PS-18 Horiz.							
#10	0.795	0.767	0.450	0.446	0.425	1.283	0.396
#7	1.368	0.677	0.476	0.473	0.461	1.259	0.399
MA-02 GC-04							
238 - 245	0.844	0.958	0.375	0.375	0.375	1.329	0.367
228 - 235	0.563	0.996	0.365	0.364	0.364	1.329	0.364
105 - 115	0.704	0.978	0.384	0.382	0.358	1.277	0.379
V-32 PC-115							
206 - 216	0.563	0.90	0.343	0.348	0.297	0.747	0.333
219 - 229	1.126	1.00	0.307	0.304	0.280	0.692	0.304
232 - 242	1.688	1.00	0.295	0.296	0.280	0.674	0.296
499 - 509	0.844	0.76	0.424	0.423	0.380	1.003	0.377
523 - 533	1.689	0.85	0.377	0.376	0.356	0.952	0.349

PI-18 Vert.

#1	1.011	0.694	0.525	0.495	0.423	1.199	0.430
#2	1.583	0.892	0.496	0.402	0.362	1.136	0.382

PI-2 Vert.

#2	0.294	0.694	0.395	0.393	0.358	0.782	0.338
#1	0.563	0.892	0.331	0.330	0.314	0.749	0.313

PI-2 Horiz.

#7	0.291	1.101	0.320	0.318	0.264	0.762	0.332
#6	0.563	1.419	0.251	0.250	0.226	0.700	0.298

$$1 \quad q_f/\bar{\sigma}_c = \frac{\bar{c} \cos \bar{\theta} + \bar{\sigma}_c \sin \bar{\theta}}{(1+(2A_f-1) \sin \bar{\theta}) \bar{\sigma}_c}$$

$$2 \quad q_f/\bar{\sigma}_c = \frac{\sin \bar{\theta}}{1+(2A_f-1) \sin \bar{\theta}}$$

$$3 \quad q_f/\bar{\sigma}_c = \frac{\bar{c} \cos \bar{\theta} + \bar{\sigma}_c \sin \bar{\theta}}{(1-\sin \bar{\theta}) \bar{\sigma}_c}$$

$$4 \quad q_f/c = \frac{(K_o+A_f(1-K_o)) \sin \bar{\theta} + (\bar{C}/\bar{\sigma}_c) \cos \bar{\theta}}{1+(2A_f-1) \sin \bar{\theta}}$$

$$K_o = 0.5$$

Table 6.5

Lateral Strain and Poisson's Ratio

Sample I.D.	ϵ_L % at				ν at			
	2.4%	5%	10%	15%	2.4%	5%	10%	15%
Smectite:								
LL-44 GPC-3								
1845-1857	1.12	2.73	7.36	14.04	0.31	0.36	0.49	0.62
1859-1868	1.28	2.97	7.23	13.07	0.35	0.40	0.48	0.58
PS-9 Vertical								
#3	0.88	2.57	6.05	10.69	0.24	0.34	0.40	0.48
#4	1.26	2.42	4.79	9.29	0.35	0.32	0.32	0.41
PS-18 Vertical								
#1	1.28	3.06	6.91	10.88	0.36	0.41	0.46	0.48
#3	1.10	2.88	6.91	10.89	0.30	0.38	0.46	0.48
PS-18 Horizontal								
#8	0.62	1.89	4.38	7.40	0.17	0.25	0.29	0.33
#10	1.04	2.16	4.64	7.74	0.29	0.29	0.31	0.34
Illite:								
MA-02 GC-04								
216-224	1.19	3.05	7.10	11.07	0.33	0.41	0.47	0.49
228-235	1.44	3.64	8.87	15.01	0.40	0.49	0.59	0.67
238-245	1.94	4.35	9.90	16.23	0.54	0.58	0.66	0.72

(Continued)

Table 6.5 (continued)

Sample I.D.	$\epsilon_L\%$ at				$\dot{\gamma}$ at			
	2.4%	5%	10%	15%	2.4%	5%	10%	15%
MA-02 GC-04								
76-84	0.66	1.54	4.65	8.69	0.18	0.21	0.31	0.39
86-94	1.35	3.36	8.39	12.86	0.37	0.45	0.56	0.57
105-115	1.61	3.52	8.18	13.24	0.45	0.47	0.54	0.59
PI-2 Vertical								
#4	0.54	0.60	1.85	3.27	0.15	0.08	0.12	0.14
#5	1.09	2.23	4.12	7.77	0.30	0.30	0.28	0.34
PI-2 Horizontal								
#6	2.09	4.64	10.16	16.33	0.58	0.62	0.68	0.73
#7	1.97	5.03	11.11	17.27	0.54	0.67	0.74	0.77
#8	0.70	1.64	5.22	8.36	0.19	0.22	0.35	0.37
#9	0.33	0.50	1.30	2.18	0.09	0.07	0.09	0.10

STRAIN (in)
DEV. STRESS vs. STRAIN
LL-44 GPC-3 1800 CM
URI/HGL

Figure 6.1

STRAIN %
PORE PRESSURE vs. STRAIN
LL-44 GPC-3 1800 CM.

Figure 6.2

Figure 6.3

STRAIN CO
A-FACTOR vs. STRAIN
LL-44 GPC-3 1800 CM
URI/MGL

Figure 6.4

STRAIN ∞
VERT./LAT. vs. STRAIN
LL-44 GPC-3 1800 CM.

Figure 6.5

VERT. STRAIN ∞
LAT. STRAIN vs. VERT. STRAIN
LL-44 GPC-3 1800 CM.

Figure 6.6

VERT. STRAIN (X)
POISSON'S RATIO vs. VERT. STRAIN
LL-44 GPC-3 1800 CM.

Figure 6.7

STRAIN (%)

DEV. STRESS vs. STRAIN

P.S.-9 VERT.

Figure 6.8

STRAIN (%)
PORE PRESSURE vs. STRAIN
P.S.-9 VERT.

Figure 6.9

Figure 6.10

STRAIN (%)
A-FACTOR vs. STRAIN
P.S.-9 VERT.

Figure 6.11

STRAIN (X)
VERT./LAT. vs. STRAIN
P.S.-9 VERT.

Figure 6.12

VERT. STRAIN (%)
LAT. STRAIN vs. VERT. STRAIN
P.S.-9 VERT.

Figure 6.13

VERT. STRAIN (%)
POISSON'S RATIO vs. VERT. STRAIN
P.S. -9 VERT.

Figure 6.14

STRAIN (%)
DEV. STRESS vs. STRAIN
P.S.-9 VERT.

Figure 6.15

DEV. STRESS (KG/CM**2)
 TANG. MOD. vs. DEV. STRESS
 P.S. -9 VERT.

Figure 6.16

STRAIN %
DEV. STRESS vs. STRAIN
P.S.-18 VERT.
URI/MGL

Figure 6.17

STRAIN %
PORE PRESSURE vs. STRAIN
P.S. -18 VERT.
URI/MGL

Figure 6.18

Figure 6.19

STRAIN ∞
A-FACTOR vs. STRAIN
P.S.-18 VERT.
URI/MGL

Figure 6.20

STRAIN (%)
VERT./LAT. vs. STRAIN
P.S.-18 VERT.

Figure 6.21

VERT. STRAIN (%)
 LAT. STRAIN vs. VERT. STRAIN
 P.S.-18 VERT.

Figure 6.22

VERT. STRAIN ∞
POISSON'S RATIO vs. VERT. STRAIN
P.S.-10 VERT.

Figure 6.23

STRAIN ∞
DEV. STRESS vs. STRAIN
P.S.-18 VERT.

Figure 6.24

DEV. STRESS OGG/CN**2
TANG. MOD. vs. DEV. STRESS
P.S.-18 VERIT.

Figure 6.25

STRAIN IN
DEV. STRESS vs. STRAIN
P.S.-18 HORIZ.

Figure 6.26

STRAIN °
PORE PRESSURE vs. STRAIN
P.S.-18 HORIZ.

Figure 6.27

Figure 6.28

STRAIN ∞
A-FACTOR vs. STRAIN
P.S.-18 HORIZ.

Figure 6.29

STRAIN ∞
VERT./LAT. vs. STRAIN
P.S.-18 HORIZ.

Figure 6.30

VERT. STRAIN (%)
LAT. STRAIN vs. VERT. STRAIN
P. S. -18 HORIZ.

Figure 6.31

VERT. STRAIN (%)
POISSON'S RATIO vs. VERT. STRAIN
P.S.-18 HORIZ.

Figure 6.32

STRAIN 00
DEV. STRESS vs. STRAIN
P.S.-18 HORIZ.

Figure 6.33

DEV. STRESS (KG/CM**2)
TANG. MOD. vs. DEV. STRESS
P.S.-18 HORIZ.

Figure 6.34

STRAIN (X)
DEV. STRESS vs. STRAIN
MA-02 GC-04 200 CM.

Figure 6.35

STRAIN (%)
PORE PRESSURE vs. STRAIN
MA-02 GC-04 200 CM.

Figure 6.36

Figure 6.37

STRAIN ∞
A-FACTOR vs. STRAIN
MA-02 GC-04 200 CM.

Figure 6.38

STRAIN (X)
VERT./LAT. vs. STRAIN
MA-02 GC-04 200 CM.

Figure 6.39

VERT. STRAIN (%)
 LAT. STRAIN vs. VERT. STRAIN
 MA-02 GC-04 200 CM.

Figure 6.40

VERT. STRAIN (%)
POISSON'S RATIO vs. VERT. STRAIN
MA-02 GC-04 200 CM.

Figure 6.41

STRAIN %
DEV. STRESS vs. STRAIN
MA-02 GC-04 200 CM.

Figure 6.42

DEV. STRESS OG/CM**2
 TANG. MOD. vs. DEV. STRESS
 NA-02 GC-04 200 CM
 URI/MEL

- Figure 6.43

STRAIN %
DEV. STRESS vs. STRAIN
MA-02 GC-04 100 CM.

Figure 6.44

STRAIN %
PORE PRESSURE vs. STRAIN
NA-02 GC-04 100 CM.

Figure 6.45

Figure 6.46

STRAIN CO
A-FACTOR vs. STRAIN
MA-02 GC-04 100 CM.

Figure 6.47

STRAIN ∞
VERT. /LAT. vs. STRAIN
MA-02 GC-04 100 CM.

Figure 6.48

VERT. STRAIN %
 LAT. STRAIN vs. VERT. STRAIN
 MA-02 GC-04 100 CM.

Figure 6.49

VERT. STRAIN ∞
 POISSON'S RATIO vs. VERT. STRAIN
 MA-02 GC-04 100 CM.

Figure 6.50

STRAIN %
 DEV. STRESS vs. STRAIN
 MA-02 GC-04 100 CM.

Figure 6.51

DEV. STRESS (KG/CM**2)
 TANG. MOD. vs. DEV. STRESS
 NA-02 GC-04 100 CM.

Figure 6.52

STRAIN (in)
DEV. STRESS vs. STRAIN
V-32 PC-115 200 CM.

Figure 6.53

STRAIN (%)
PORE PRESSURE vs. STRAIN
V-32 PC-115 200 CM.

Figure 6.54

Figure 6.55

STRAIN CO
A-FACTOR vs. STRAIN
V-32 PC-115 200 CM.

Figure 6.56

STRAIN (X)
VERT./LAT. vs. STRAIN
V-32 PC-115 200 CM.

Figure 6.57

STRAIN %
DEV. STRESS vs. STRAIN
V-32 PC-115 200 CM.

Figure 6.58

DEV. STRESS (KG/CM**2)
 TANG. MOD. vs. DEV. STRESS
 V-32 PC-115 200 CM.

Figure 6.59

STRAIN CM
DEV. STRESS vs. STRAIN
V-32 PC-115 400 CM.

Figure 6.60

STRAIN %
PORE PRESSURE vs. STRAIN
V-32 PC-115 400 CM.

Figure 6.61

Figure 6.62

STRAIN ∞
A-FACTOR vs. STRAIN
V-32 PC-115 400 CM.

Figure 6.63

STRAIN (%)
VERT./LAT. vs. STRAIN
V-32 PC-115 400 CM.

Figure 6.64

STRAIN (%)
DEV. STRESS vs. STRAIN
V-32 PC-115 400 CM.

Figure 6.65

DEV. STRESS (KG/CM**2)
TANG. MOD. vs. DEV. STRESS
V-32 PC-115 400 CM.

Figure 6.66

STRAIN (%)
DEV. STRESS vs. STRAIN
P. I. -18 VERT.

Figure 6.67

STRAIN %
PORE PRESSURE vs. STRAIN
P.I.-18 VERT.

Figure 6.68

Figure 6.69

STRAIN ∞
A-FACTOR vs. STRAIN
P. I. -18 VERT.

Figure 6.70

STRAIN (%)
VERT. /LAT. vs. STRAIN
P. I.-18 VERT.

Figure 6.71

DEV. STRESS vs. STRAIN
P.I.-18 VERT.

Figure 6.72

DEV. STRESS OXG/CM**2
TANG. MOD. vs. DEV. STRESS
P.I.-18 VERT.

Figure 6.73

STRAIN (%)
DEV. STRESS vs. STRAIN
P. I.-2 VERT.

Figure 6.74

STRAIN %
PORE PRESSURE vs. STRAIN
P.1.-2 VERT.

Figure 6.75

STRAIN (%)
A-FACTOR vs. STRAIN
P.I.-2 VERT.

Figure 6.77

STRAIN ∞
VERT./LAT. vs. STRAIN
P.1.-2 VERT.
URL/NGL

Figure 6.78

VERT. STRAIN (%)
LAT. STRAIN vs. VERT. STRAIN
P. I. -2 VERT.

Figure 6.79

VERT. STRAIN %
POISSON'S RATIO vs. VERT. STRAIN
P. I.-2 VERT.

Figure 6.80

STRAIN ∞
DEV. STRESS vs. STRAIN
P. I. -2 VERT.

Figure 6.81

DEV. STRESS (KG/CM**2)
TANG. MOD. vs. DEV. STRESS
P.I.-2 VERT.

Figure 6.82

STRAIN (inches)

DEV. STRESS vs. STRAIN

P. 1.-2 HORIZ.

Figure 6.83

STRAIN %
PORE PRESSURE vs. STRAIN
P. I. -2 HORIZ.

Figure 6.84

STRAIN ∞
A-FACTOR vs. STRAIN
P. I. -2 HORIZ.

Figure 6.86

STRAIN CO
VERT. / LAT. vs. STRAIN
P. 1.-2 HORIZ
URI/MGL

Figure 6.87

VERT. STRAIN %
 LAT. STRAIN vs. VERT. STRAIN
 P.1.-2 HORIZ

Figure 6.88

VERT. STRAIN CO
POISSON'S RATIO vs. VERT. STRAIN
P. I. -2 HORIZ.

Figure 6.89

STRAIN %
DEV. STRESS vs. STRAIN
P.1.-2 HORIZ

Figure 6.90

DEV. STRESS O/G/CM**2
TANG. MOD. vs. DEV. STRESS
P.I.-2 HORIZ.

Figure 6.91

Figure 6.92

Figure 6.93

STRAIN (%)
AREA vs. STRAIN
LL-44 GPC-3 1800 CM.

Figure 6.94

STRAIN (%)
AREA vs. STRAIN
P.S.-18 VERT.

Figure 6.95

STRAIN (%)
AREA vs. STRAIN
P.S.-18 HORIZ.

Figure 6.96

STRAIN (%)
AREA vs. STRAIN
MA-02 GC-04 200 CM.

Figure 6.97

STRAIN °
DEV. STRESS vs. STRAIN
LL-44 GPC-3 1800 CM.

Figure 6.98

STRAIN %
DEV. STRESS vs. STRAIN
P.S.-9 VERT.

Figure 6.99

STRAIN %
DEV. STRESS vs. STRAIN
P.S.-18 VERT.

Figure 6.100

STRAIN %
DEV. STRESS vs. STRAIN
P.S.-19 HORIZ.

Figure 6.101

STRAIN (°)
DEV. STRESS vs. STRAIN
MA-02 GC-04 200 CM.

Figure 6.102

DEV. STRESS vs. STRAIN
HA-02 GC-04 100 CM.

Figure 6.103

STRAIN %
DEV. STRESS vs. STRAIN
P.I.-2 VERT.

Figure 6.104

STRAIN IN
DEV. STRESS vs. STRAIN
P.1.-2 HORIZ.

Figure 6.105

CHAPTER 7

DISCUSSION OF RESULTS

7.1 Introduction

Isotropically consolidated undrained (CIU) triaxial tests were performed on eleven series of undisturbed and remolded illite and smectite samples, for which Mohr-Coulomb parameters of effective cohesion $\bar{c}$ and effective friction angle $\bar{\phi}$ were determined. Other stress-strain and Mohr-Coulomb parameters, including hyperbolic deviatoric stresses and tangent moduli, lateral strains and Poisson's ratio, were determined and have been presented in Chapter 6 in the form of plots and tables. This chapter presents a condensed analysis of the material presented in Chapter 6. The values of $\bar{c}$ and $\bar{\phi}$ are summarized in Table 7.1 for each series tested, along with results reported in the literature for Pacific sediments. The available data reflects the limited number of investigations on similar sedimentary materials. The results are discussed and comparisons made in accordance with the objectives of the study (Section 1.6). The average normalized drained and undrained strengths and water contents at failure are listed in Table 7.2. Pertinent equations and procedures to determine the hyperbolic stress-strain values are presented. A discussion of the measured lateral deformations and calculated Poisson's ratios concludes this chapter.

7.2 Mohr-Coulomb Parameters

The parameters of $\bar{c}$ and $\bar{\phi}$ for the smectite samples were not significantly influenced by the type of remolding and reconsolidation used in this program (see Section 2.8). Remolded sample series had friction angles greater (PS-9 Vertical, $\bar{\phi} = 37.3^\circ$ and PS-18 Vertical, $\bar{\phi} = 36.2^\circ$) and less than (PS-18 Horizontal, $\bar{\phi} = 33.4^\circ$) the single undisturbed series (LL-44 GPC-3 1800 cm, $\bar{\phi} = 35.5^\circ$). However, it is noted that the core samples probably have some amount of undetermined disturbance due to sampling effects and pressure relief. The cohesion intercepts were all less than or equal to 0.025 kg/cm^2 and therefore contributed to only a small fraction of the sample strengths. The values obtained in this study for undisturbed smectite samples compare favorably with those reported by Lee and Hamilton (1974) (see Table 7.1).

The remolded illite samples had friction angles considerably lower (6.4° average) than the undisturbed MARA samples (see Table 7.1). Probable sample disturbance (caused by corer design) resulted in lower friction angles for the two "undisturbed" VEMA sample series. Silva and Clukey (1975) reported test results on illite samples from the central North Pacific obtained with a sphincter corer (21.3 cm I.D., 21.9 cm O.D., and 91 cm length) and a spade box corer (7.4 cm I.D., 7.6 cm O.D., and 61 cm length). Friction angles of 34.0° and 35.8° were obtained from CIU triaxial tests. An average sensitivity, from vane shear

measurements, of 5.7 was determined for the box core sample, indicating a high quality "undisturbed" core. Noorany (1971) reported a friction angle of 32 degrees and a cohesion intercept of 0.02 kg/cm^2 for a pelagic clay (probably illite; $w\% = 123$) taken from a box corer in the North Pacific. Lee and Hamilton (1974) reported $\bar{c}$ and $\bar{\phi}$ values of $0.018\text{--}0.034 \text{ kg/cm}^2$ and $35\text{--}36$ degrees (see Table 7.1) for undisturbed illite samples taken in the North Pacific. Although slightly higher, the friction angles reported compare favorably with the MARA core results of this study.

Mohr-Coulomb parameters of cohesion intercept and friction angle determined for three "undisturbed" sample series, LL-44 GPC-3, MARA-02 GC-04 100 cm and 200 cm are similar to values reported in the literature. Triaxial test data from remolded deep-sea samples could not be found in the literature. Olson (1974) studied the effects of electrolyte and pH on the residual friction angle of remolded sedimented samples of pure kaolinite, illite, and montmorillonite. The friction angles for all three clays were controlled by physical factors of particle size and shape rather than chemical factors. The larger illite particle sizes resulted in larger residual friction angles for the illite in comparison to the montmorillonite. Olson's conclusions seemingly contradict the results obtained in this study. However, there are several significant differences between the samples tested in the two programs; the major one being the composition of the samples. Olson used

pure clays while the two sediments, illite and smectite, are composed of several different clays and clay sized particles (see Section 2.7). The grain size distribution of Olson's samples varied considerably (particle height to diameter ratios varied from 2 to 500), whereas the illite and smectite samples had similar grain sizes and grain size distributions. Also, Olson's study concentrated on comparing the residual friction angles as opposed to peak friction angles.

7.3 Undisturbed versus Remolded Samples

The friction angle of the undisturbed series (LL-44 GPC-3) is only 1.3° less than the average friction angle of the two remolded series (neglecting the Horizontal samples). The normalized drained strengths ($q_{fd}/\bar{\sigma}_c$) indicate the remolded smectite samples are slightly stronger than the undisturbed samples (see Table 7.2). The average normalized undrained shear strengths of the undisturbed smectite samples are about the same (0.44) as the remolded series (0.44 for the PS-9 Vertical and 0.47 for the PS-18 Vertical samples). Remolded samples should have higher $q_{fu}/\bar{\sigma}_c$ ratios due to the denser packing of the particles and lower water contents at failure. The high $q_{fu}/\bar{\sigma}_c$ ratios of the undisturbed samples are probably due to the inherent structure of the samples and possible chemical interparticle bonding. To compare the undisturbed and remolded samples at equivalent water contents (i.e. the same void ratio), the confining stress of the undisturbed samples would have to be in-

creased, resulting in higher shear strengths. As an example, at equivalent water contents the undisturbed LL-44 GPC-3 samples would have an undrained strength 2.5 times that of the PS-18 Vertical samples.

Neglecting the VEMA samples, the undisturbed illite samples have friction angles significantly higher (6.4° average) than the remolded samples (see Table 7.1). A difference of 10.2 degrees is observed between the MARA-02 200 cm series and the PI-2 Vertical series (34.8° versus 24.6° , respectively). The larger friction angles are attributed to the structure and interparticle bonding in the undisturbed samples. The normalized drained strengths (Table 7.2), being a function of both $\bar{c}$ and $\bar{\phi}$, are larger for the undisturbed samples. The average normalized undrained strength of the MARA samples (Table 7.2) is considerably less than the PI-18 Vertical samples and only slightly higher than the PI-2 Vertical samples (0.37, 0.45 and 0.36, respectively). However, to compare the undisturbed samples at water contents equivalent to those of the PI-18 samples would require a confining stress three times greater, resulting in an undrained strength twice that of the PI-18 samples.

As discussed in Chapter 6, the lower friction angles of the VEMA samples are due to the sensitivity of the illite and the tendency for the small diameter (6.4 cm) standard piston corer to cause sample disturbance. The stress-strain plots for these samples indicate similar behavior during

compression and thus suggest the samples were subjected to uniform or equivalent degrees of disturbance.

On the other hand, it appears that the larger diameter (11.4 vs. 6.4 cm) Giant Piston Corer samples were not appreciably disturbed, since the effective friction angle of the Long Lines series was quite high. Based on this data, it is concluded that samples taken with the Giant Piston Corer are of better quality than the samples from the standard piston corer.

7.4 Smectite versus Illite Samples

Comparing the remolded vertically oriented smectite and illite samples, the smectite samples have larger friction angles (26% higher) and thus greater normalized drained strengths (50% higher). Excluding the PI-18 Vertical series, the undrained strengths of the smectite samples are greater than the illite samples (see Table 7.2).

The most significant difference between the two materials is their susceptibility to remolding and disturbance. The friction angles of the smectites were apparently not influenced by remolding (see Table 7.1), while the illite samples exhibited a significant reduction in friction angle. Thus, the strength properties of illite appear to be influenced strongly by the degree of disturbance and remolding (remolded samples in this study were first completely homogenized and then reconsolidated in the Hole Closure tanks (see Section 2.8). If this hypothesis is correct, the transition sediments should have intermediate

friction angles between the remolded illite and the undisturbed or remolded smectite in a disturbed core. The series VEMA-32 400 cm was taken in the upper transition zone of PC-115. The friction angle of this series is 28.3 degrees, 5.6° higher than the VEMA-32 200 cm series and 7.2° less than the LL-44 GPC-3 samples. To support this hypothesis, the vane shear measurements of GPC-3 indicate that the sensitivity decreases with depth. Thus, the sensitivity changes with changing sedimentary material, the illite having the highest sensitivity and the smectite the lowest (see Section 2.4). This statement assumes the disturbance due to the coring operation was uniform for the entire core. Similar trends in vane shear sensitivity were reported by Lee and Hamilton (1974).

7.5 Vertical versus Horizontal Samples

Two remolded horizontally oriented sample series (smectite and illite) were tested in attempts to determine the effects of sample orientation on the shear strength behavior of the two sedimentary materials. In both series, the horizontally oriented samples exhibited lower friction angles and larger cohesion intercepts (Table 7.1). In terms of drained strength, the smectite samples appear to be influenced more by sample orientation (Table 7.2). The results from the undrained analysis are inconclusive. Further testing is required to establish a definite quantitative analysis of changing sample behavior due to sample

orientation. However, it can be stated that orientation has an effect upon the strength parameters. Also, it should be noted that anisotropy effects in the natural sediment column which was deposited at a very slow rate (approximately 1mm/m.y.) are probably much different than in the laboratory reconstituted-reconsolidated samples.

7.6 Hyperbolic Theory

The hyperbolic stress-strain theory assumes a hyperbola can be used to approximate any experimental stress-strain curve. Equation 5-12 represents the hyperbolic stress-strain curve as a function of the initial Young's modulus and the ultimate deviator stress. To determine tangent Young's modulus, Equation 5-12 is differentiated with respect to strain. Equation 5-17 is the final equation for the tangent modulus assuming a Mohr-Coulomb failure theory. Knowing six material property terms ($\bar{c}$, $\bar{\phi}$, A_f , K , n , R_f), the tangent modulus for any confining stress and deviatoric stress can be calculated.

As discussed in Chapter 6, the hyperbolic theory models the stress-strain response of the two sedimentary materials very well. The hyperbolic theory is only applicable for stress-strain conditions prior to failure and therefore cannot model strain softening. Also, the loading conditions used in the lab to determine the material parameters for the hyperbolic model should simulate the loading conditions expected in the field.

7.7 Poisson's Ratio

From elastic theory, Poisson's ratio has an upper bound of one-half and a lower bound of zero. However, drained triaxial tests on dilatant soils have resulted in poisson's ratios greater than 0.5 (see Section 4.4). It is questionable whether or not soil can be considered an elastic material. Hardin (1978) states the purely elastic strains in clays are only measured during small amplitude cyclic loading. If elastic parameters are determined, they should only be determined in the "linear-elastic" range of the stress-strain curves.

In an undrained (no volume change condition) triaxial test, assuming small strain theory is adequate, Poisson's ratio should approach a value of 0.5. As discussed in Chapter 6, Poisson's ratios greater than 0.5 were calculated for several samples tested. As an example, the two samples LL-44 GPC-3 1845-1857 cm and 1859-1868 cm have Poisson's ratios greater than 0.5 at vertical strains exceeding 10%. If large strain theory were assumed (second order terms included), the Poisson's ratios at 15% vertical strain for these samples (0.62 and 0.58, respectively) would still exceed the theoretical values for no volume change conditions. The correction factor of $2/3$ (to account for sample deformational shape) was included in the calculation of Poisson's ratio for these samples (see Section 4.5). The assumed deformational shape of the samples during uniaxial compression remains the largest contributor to the erroneous values

of Poisson's ratio.

Table 1.

Mohr-Coulomb parameters:
Cohesion Intercept and Friction Angle

Sample Series

 $\bar{c}$
(kg/cm²) $\bar{\phi}$
degreesSan Jose

LI-21, GC-3 180 cm	0.012	35.5
25-9 Vertical	0.012	37.3
25-11 Vertical	0.009	36.3
25-18 Horizontal	0.005	33.4

Illite

MA-32, GC-54, 200 cm	0	34.8
MA-32, GC-54, 100 cm	0.032	33.0
SI-18 Vertical	0.10	30.4
SI-2 Vertical	0.013	34.0
SI-3 Horizontal	0.025	32.8
SI-12 GC-115 200 cm	0.041	32.0
SI-13 GC-115 400 cm*	0.051	30.3

Lee and Hamilton (1974)

Illite (depth: upper 10
meters) 0.018-0.034 35-36Smectite (depth: upper
10 meters) 0.039-0.042 32-38

Silva and Cluskey (1975)

Illite (depth: upper meter) 0 36
0 35.8

Boorany

Pelagic Clay (Illite) 0.02 32

*Transition: Illite to smectite

Table 7.1

Mohr-Coulomb Parameters:
Cohesion Intercept and Friction Angle

Sample Series

$\bar{c}$
(kg/cm²) $\bar{\phi}$
degrees

Smectite

LL-44, GPC-3, 1800 cm	0.016	35.5
PS-9 Vertical	0.011	37.3
PS-18 Vertical	0.009	36.3
PS-18 Horizontal	0.025	33.4

Illite

MA-02, GC-04, 200 cm	0	34.8
MA-02, GC-04, 100 cm	0.031	33.0
PI-18 Vertical	0.101	30.4
PI-2 Vertical	0.013	24.6
PI-2 Horizontal	0.025	22.8
V-32 PC-115 200 cm	0.041	22.9
V-32 PC-115 400 cm*	0.051	28.3

Lee and Hamilton (1974)

Illite (depth: upper 10 meters)	0.018-0.034	35-36
Smectite (depth: upper 10 meters)	0.035-0.042	37-38

Silva and Clukey (1975)

Illite (depth: upper meter)	0	34
	0	35.8

Noorany

Pelagic Clay (Illite)	0.02	32
-----------------------	------	----

*Transition: illite to smectite

Table 7.2

Average Normalized Undrained
and Drained Strengths

<u>Sample Series</u>	<u>$q_{fu}/\bar{\sigma}_c$</u>	<u>$q_{fd}/\bar{\sigma}_c$</u>	<u>$w_f\%$</u>
LL-44 GPC-3 1800 cm	0.437	1.404	154.3
PS-9 Vertical	0.440	1.573	125.5
PS-18 Vertical	0.468	1.468	109.2
PS-18 Horizontal	0.460	1.271	119.9
MA-02 GC-04 100 and 200 cm	0.374	1.312	87.0
V-32 PC-115 200 cm	0.316	0.704	72.4
V-32 PC-115 400 cm	0.400	0.978	84.4
PI-18 Vertical	0.449	1.168	63.5
PI-2 Vertical	0.362	0.766	71.7
PI-2 Horizontal	0.284	0.731	71.9

CHAPTER 8

CONCLUSIONS

1) Mohr-Coulomb parameters were determined for two sedimentary materials illite and smectite for both remolded and undisturbed samples.

2) Compared under the same conditions i.e. same water content or void ratio, the undisturbed samples have greater drained and undrained strengths. Friction angles for the smectite samples were not affected considerably by remolding, but the illite samples exhibited a sizeable reduction in friction angle after remolding and reconsolidation in a tank.

3) Smectite samples had friction angles greater than the illite samples (neglecting the horizontally oriented samples). When compared at equivalent water contents, the smectite samples have greater drained and undrained strengths. The illite samples are affected more by disturbance and remolding.

4) The stress-strain properties of remolded samples are changed by sample orientation. In comparison to the vertically oriented samples, horizontal samples had smaller friction angles and larger cohesion intercepts.

5) The hyperbolic stress-strain analysis may be used to model the two materials, illite and smectite. The hyperbolic theory is only applicable for stress-strain conditions prior to failure and therefore cannot model strain softening.

6) Direct (on sample) measurement of lateral deformations have been made with a new lateral strain gauge. Calculated Poisson's ratios are a function of the assumed deformational shape of the sample. Drained triaxial tests would provide a better method of determination.

A.1 Introduction

The purpose of this manual is to provide a comprehensive guide to the use of the CIU Triaxial Testing Manual. This manual is intended for use by personnel responsible for the operation and maintenance of the CIU Triaxial Testing Manual. The manual is divided into two main sections: A.1 Introduction and A.2 Equipment. The introduction section provides a general overview of the manual and its purpose. The equipment section provides a detailed description of the equipment used in the testing process. The manual is written in a clear and concise manner, using simple language and diagrams to illustrate the testing process. The manual is intended to be used as a reference guide for personnel responsible for the operation and maintenance of the CIU Triaxial Testing Manual. The manual is written in a clear and concise manner, using simple language and diagrams to illustrate the testing process. The manual is intended to be used as a reference guide for personnel responsible for the operation and maintenance of the CIU Triaxial Testing Manual.

APPENDIX A

CIU TRIAXIAL TESTING MANUAL

A.2 Equipment

Special Equipment

1. Compressor	11. Porous Stones
2. Triaxial Cell	12. Top Cap
3. Hydraulic Pump	13. Lateral Strain Gauge
4. Sample Grabs	14. Transducers
5. Membranes	15. Hewlett-Packard 9825A

(Continued)

A.1 Introduction

The purpose of this manual is to guide users in the preparation, set-up and operation of a CIU (Consolidated Isotropically Undrained) triaxial test using the URI/MGL (Marine Geomechanics Lab) system located in the Civil Engineering Department of URI. Potential problem areas in the test procedures have been pointed out as well as improvements that have been made to the procedures. All equipment required to complete these procedures are listed in Section A.2. Also included are brief instructions in the use of the HP (Hewlett Packard) programs that have been developed by the author. These programs will greatly reduce the work involved in running the tests and performing calculations. It is assumed that the person using these directions has a basic understanding of soil mechanic principles and terminology, triaxial testing procedures and operation of the HP calculator and peripherals. These procedures apply to essentially saturated marine sediments which have salt water in the pores.

A.2 Equipment

Special Equipment

1) De-aerator	6) Porous Stones
2) Triaxial Cell	7) Top Cap
3) Geonor Sample Stand	8) Lateral Strain Gauge
4) Sample Cradle	9) Transducers
5) Membranes and
O-rings	10) Hewlett-Packard 9825A

General Equipment

1) Dental Spatula	7) Assorted Wrenches
2) Data Sheets	8) Silicon Spray
3) Stopcock Grease	9) Extrusion Piston
4) Filter Paper	10) Wire Saw
5) Water Content Tins	11) Oil and Oil Pump Can
6) Timer	

A.3 Reference Literature

Bishop and Henkel (1962), "The Measurement of

Soil Properties in the Triaxial Test".

Lambe (1951), "Soil Testing for Engineers".

Hewlett-Packard 9825A Desktop Computer (1976)-

Programming Manuals.

A.4 Sample Set-up

Procedures in this section include preparation of the sample, de-airing the cell manifold and setting the sample up in the triaxial cell.

a) De-airing the Cell

Pump saltwater into the de-aerator using the vacuum pump and opening the valves to the water supply. After the de-aerator is 3/4 full, close the water supply valve and turn on the de-aerator while the vacuum pump is operating. When the water column is clear (no air bubbles present) the water is ready and the vacuum should be released from the chamber.

The 1/4 inch nylon tubing from the de-aerator is connected to both the single pedestal drainage port (valve H) and the manifold (see value E, Figure A.1). As water flows through the lines tap the cell to dislodge any air bubbles. The nylon line remains connected to the manifold for later use.

b) Sample Preparation

The triaxial data sheets used for these tests have been formatted in such a way that if all the necessary information is written on the sheets, no step in the procedures will be omitted. Sample data sheets are presented in Section A.11. Two sheets are needed for the sample set-up, the cover sheet and the sample sheet.

Using a dental spatula, remove the end caps from the sample cylinder, coat the sides of the extruding piston with a thin film of silicon lubricant and then place the end of the sampling tube with the cutting edge on the top of the piston. With one continuous movement, push the sample cylinder down over the piston. The sample can now be transferred from the top of the piston to the sample cradle, by placing the cradle against the sample and then rotating them to a horizontal position.

The sample ends are cut with a wire saw and water content samples are taken from the trimmings. For the final trimming, a spacer is used to push the sample into the cradle to obtain a 7.62 cm (3.0") sample. The water content samples and the sample and cradle are weighed immedi-

ately.

c) Completion of Sample Set-up

To aid in placing the membranes and O-rings on the sample, a Geonor sample stand is used (Figure A.2). This stand sits on the base of the triaxial cell and may be centered by observing the pedestal through the membrane stretcher. After the stand is in place, coat the sides of the pedestal and top cap with stopcock grease, place a porous stone and filter paper cover on the end of the sample and transfer the sample from the cradle onto the pedestal. A second filter paper and porous stone should be placed on the top of the sample. Nine 0.64 cm (1/4 inch) strips of wetted filter paper (Whatman's No. 54) are placed 0.64 cm (1/4 inch) apart around the perimeter of the sample. The strips should extend beyond the length of the sample and contact the porous stones at each end.

Enclose the sample with two rubber membranes, the second should have a thin layer of stopcock grease applied to the inside. Both membranes should be trimmed level with the top cap before placing the O-rings at each end of the sample.

Top drainage can be used to decrease consolidation time and/or to measure pore pressure during consolidation when a back pressure is applied to the sample. The top drainage line is a nylon line which is connected to the lexan top cap, wrapped around the sample (see Figure A.3), and threaded into the top drainage port of the cell base

(see Figure A.1). The top drainage line must be wrapped tight enough around the sample to allow the top of the cell to be lowered onto the base.

If a lateral strain gauge clip is to be used, the following steps are required:

- 1) Find the mid-height of the sample at two points diametrically opposite each other. It is better to locate these points over a filter strip, the strip will distribute some of the bearing stress.

- 2) Using Eastman 910 epoxy, epoxy the metal tabs onto the membranes at the selected points.

- 3) Place the gauge around the sample and turn the gauge such that the lead wires wrap around the sample.

- 4) Position the gauge on the sample(see Figure A.4) and deflect the gauge enough to allow the gauge to hang from the tabs (the gauge deflection must also allow for diameter decrease due to consolidation).

Before positioning the top of the cell on the base, several items should be checked. Insure that:

- 1) the piston is clamped in position at the top of the cell, to assure that the sample is not damaged.

- 2) the oil port is open to vent the air.

- 3) the top of the cell is positioned so that the threaded hole for the vertical strain clamp is in front.

- 4) the top drainage line and/or the lateral strain gauge lead wires will not be disturbed when the top of the cell is placed on the base.

After the top is in place, tighten the locking nuts and close the cell pressure valve.

A.5 Cell Preparation

This section describes both connecting the drainage and pressure lines and the connection of the pore pressure transducers. It is very important to make wet connections in the following procedures, otherwise air will be trapped in the lines producing erroneous results. Wet connections are made by allowing fluid to flow through two fittings before joining them together.

a) Filling the Cell Chamber

When using salt water as a confining fluid, fill the cell chamber from the de-aerator by opening valves E and G (see Figure A.1) and allowing the fluid to enter through the top drainage port. If a top drainage line is in place, a line connecting valve C to the cell pressure valve (valve I) should be used (see Figure A.5). When using silicon oil as a confining fluid, pressurize the oil reservoir and connect the nylon line from the reservoir to the cell pressure valve (valve I in Figure A.1). A layer of 10-30 motor oil on top of the cell fluid is used to lubricate the piston and prevent cell fluid leakage. Place the oil in the cell by using an oil pump can, attached to the oil port at the top of the cell.

b) Pressure Transducers

Two types of transducers are used in the lab; differential and absolute. The type of transducer used is a

function of the desired consolidation pressure. The differential transducers are used for confining stresses less than 0.56 kg/cm^2 (8 psi). The two types of transducers require different methods of testing, line attachment and pore, cell or back pressure calculation.

Note: Different electrical cables have been made for each transducer, be sure to match the cable to the transducer. Also, in all transducers, silicon oil should be entered into the ports using a plastic syringe, to prevent salt-water from corroding the transducer.

Connect the absolute transducers (Dynisco: 0-100 PSI) to valve A on the manifold. The back pressure can be monitored by opening valves B and D (see Section A.5 (C)). The cell pressure can be monitored by using the line from the cell pressure valve to valve C (see Figure A.5). When using the differential transducers, it is important not to exceed the maximum differential pressure of the transducer. Therefore, the differential transducers must always be monitoring the cell and back pressure except during undrained conditions. Connect the cell pressure line to the high pressure port of the transducer (see Figure A.6) and the low pressure port connected to either valve A or B.

c) Drainage Lines

The drainage line to the burrettes must be free of air bubbles and is connected using a wet connection. Using an absolute transducer, connect the drainage line to valve B

(Figure A.1). When using the differential transducers, the drainage line is connected to either valve A or B, depending upon which transducer is used. The back pressure is applied through the burette system, therefore it is important to allow at least a 4-5 ml. decrease in volume for overconsolidated samples. Burette readings should be taken as required on the data sheets.

A.6 Line Pressures

This section gives procedures used to set cell, back and confining pressures. It is possible and desirable in most cases to determine the pressures using the pressure transducer instead of the bourdon tube pressure gauge.

Note: All valves on the panel should be checked to determine if they are in the correct position.

a) Confining Pressure

The sample confining stresses may be applied by either of two methods. For confining pressures less than 2 psi, the cell and back pressure reservoirs (15 and 16 on Figure A.7) may be adjusted to obtain a head differential. For applying pressures greater than 2 psi, the reservoirs remain at equal levels and the cell pressure regulator (see 5 on Figure A.4) is adjusted to obtain the correct pressure.

b) Back Pressure

The back pressure is applied to the system by adjusting the back pressure regulator (see 6 on Figure A.7). When using the static head confining pressure, a back pres-

sure is applied to both sides of the system by turning the switching valve (see 13 on Figure A.7) to the right. When air pressure is used throughout, the cross-over valve (see 4 on Figure A.7) should be positioned (turned to the left) to back pressure the system, increasing the pressure of both sides equally (some slight adjustment of the cell pressure may be necessary).

c) Checking Pressures

Using the HP calculator, scanner and voltmeter, it is possible to monitor the transducer output and obtain a reading directly in psi. This reading may be checked with the bourdon tube pressure gauge. Using this method, accurate adjustment of all pressures is possible, eliminating the errors inherent in the pressure gauge. Enter the transducer scaling factors into the voltmeter via the front panel keys. Loading the special function keys will enable the user to obtain manual readings from the digital volt meter (DVM) (see HP operating manual). After the scaling factors have been entered:

1. Load tape into the calculator.
2. Type in "trk 0;ldk 12" and press EXEC key.
The special function keys are now loaded.
3. To obtain transducer output in volts, key in f1, f2, channel #, f6.
4. To obtain scaled output, key in f1, shift f0, channel #, f6.
5. To take one reading, press key f5.

6. To take multiple readings of one channel, key in shift f2.

A.7 Test Procedures

This section covers the procedures used to first consolidate the sample, second apply a back pressure and third check the sample response.

a) Consolidation

The sample consolidation is started by setting the correct cell pressure and recording the initial burette readings. Open the drainage and cell pressure valves and record the change in volume with time as indicated on the data sheet. Care should be used, not to fill the burettes to a level greater than the 10 ml. mark on the burette. The consolidation pressure is applied in one load increment in most cases but may be applied in several increments if desired.

b) Back Pressure

As indicated in Section A.6 (C), the back pressure is applied to both sides of the system. Steps of 10 psi per hour have been used successfully, to a final back pressure of 40 psi. Again care should be used not to allow the fluid level in the burettes to fall below the 0 ml. mark. If a large drop in fluid level is noticed when applying the back pressure, air could be trapped in the lines or pressure transducer.

c) Response

After a suitable time period under the maximum back

pressure, the sample pore pressure response should be checked. A program has been written for the HP to check the sample response using both absolute and differential transducers. This program requires the DVM and scanner to be operating and the pressure transducer plugged into channel 2 on the terminal box. Load the program, press RUN, enter sample I.D. and depth, enter the transducer constants (slope and intercept) and then close the drainage and cell pressure valves. Record the existing cell pressure with the pressure transducer and then the increased pressure. When RECORD PORE PRESSURE is displayed, insure that the pore pressure is being monitored by the pressure transducer. Press CONT key, allow one reading to be taken and then open the cell pressure valve. Readings are taken by the program every 10 seconds and the response at that time calculated (must press STOP key to stop the program). If the response does not reach 95 percent in 2-3 minutes, it is possible that air is in the lines and appropriate procedures should be followed.

A.8 HP Triax Program Preparation

Several programs have been written for the HP to analyze and plot triaxial test results. A flowchart of the overall progression of triaxial programs and data files is presented in Figure A.8. Other programs such as water content and least square fit are available on the tape for general use. These programs are presented in A.13, so that they may be referred to if problems develop.

This section describes how to: 1) prepare data files to store program data, 2) store the sample data and 3) run the programs. Three files, Heading, Sample and Test Data files, are discussed in this section and later sections of this manual. The three files contain information on the sample to be tested or on samples already tested. One important item which should be mentioned is the tapes used in the calculator. These tapes have two tracks (track 0 and 1) which must not be confused. The operator must be continually aware of which track the calculator is using. It is possible to eliminate important data files if the incorrect track were used for recording or tape marking. All the programs and plotting output are stored on track 0 and all the sample data files are on track 1.

a) Heading Program

This program stores data, entered by the user, on two files. The first file ("Heading File") contains information such as sample I.D., response of sample, initial length and diameter, etc. The second file ("Sample Data File") contains variables necessary to run the main triax program.

Before running this program, fill out completely the sample summary sheet, so that the questions for the Heading program can be answered correctly. Also, the tape must be marked to store the data, 240 bytes for the Heading file (file 1) and 120 bytes for the Sample Data file (file 2).

Note: the Mark Tape program (track 0, file 0) may be used

to mark the files on the tape. Mark a third file (3220 bytes) at this time, for the storage of the transducer output, which will be generated during the shear test. This file will be the "Test Data File". From the tape, load the Heading Program, answer all the questions (see A.13) and store the data on track 1, the Heading data in the first data file (240 byte file) and the Sample data in the second file (120 byte file).

b) Write Heading Program

This program loads the Heading data file from Section A.8(a) and writes out the information in a standard format. To run this program, load the Write Heading program. When the program asks for the "File to be Loaded", enter the file number for the Heading file, the output will be printed on the line printer.

c) Triax Real Time Program

This program, records all transducer outputs, performs calculations and prints out the data while the test is in progress. The program continually monitors vertical strain and at every 0.2 percent strain takes readings on all transducers. Before running this program, the sample should be ready to shear with drainage lines closed (see section A.9 (c)), all transducers plugged into the correct position and the piston seated on the top cap.

The program which starts the main triax program is TRIAX. init. This program initializes the scanner and DVM to the ready state. Several questions will be asked and one

should respond by entering the appropriate information and pressing the CONT. key (see program in A.13). After the first reading, turn the press on and the program will continue until interrupted or 99 readings have been taken. The data should be recorded in the "Test Data File" after the program asks to record the data.

A.9 Final Test Preparations

This section outlines the final steps required before turning the processing over to the HP.

a) Seating the Piston

Place the cell on the press being careful not to pull on the nylon lines. The cell is raised using the coarse adjustment on the load press until the piston is in near contact with the top cap. From this point, the fine adjustment is used to raise the cell. While raising the cell, rotate the piston by hand to seat it properly on the top cap. The piston is seated when the load dial gauge increases dramatically as the fine adjustment is turned.

b) Transducers

The transducers are required by the program to be in a definite sequence. The transducers should be plugged into the terminal box in the following order:

<u>Channel #</u>	<u>Transducer</u>
0	Strain
1	Load
2	Pore Pressure
3	Lateral Strain

c) Things to Check

The following list contains items which are commonly forgotten and should be checked before the triax program is run. Check to see if:

- 1) The transducer outputs are plugged in correctly and are operating.
- 2) The special function keys have been erased.
- 3) The piston clamp has been removed or moved to the top of the piston.
- 4) The vertical strain clamp is positioned to avoid contact with the proving ring.
- 5) All wires and tubes are free to move with the cell.
- 6) Correct strain rate has been set on the press.
- 7) Close drainage!!!

A.10 Plotting Test Data

Now that the test data has been stored on tape, it is possible to retrieve this information at any time and rerun the calculations and obtain plots of the test parameters. The following Triax Plotting program is completely separate from the Real Time Triax program used to record the test data.

- a) This second triax program performs the same calculations as the first, however it stores plotting output on seven files (one plot per file). Therefore, seven files of 1620 bytes must be marked on the tape before this program may be run. Load the program, press RUN, the program will

ask that the file number of the Sample Data file and the Test Data file be entered into the program. Several other questions will be asked in reference to the track and files onto which the plotting output will be stored. After the first set of calculations have been completed, the program will stop and "ENTER TAPE" will appear in the display. Press the CONT key, the program will then proceed through the calculations seven times, recording a different set of parameters on tape each time.

b) Plotting Program

This program loads the plotting output generated (see previous section) and plots the data with identifying labels. Load "PLOT PROG", the program will ask for the file number for plotting. After the output has been loaded, the sample I.D. and consolidation pressure (in kg/cm^2) should be entered. The program will also ask for the, X and Y axis labels, the maximum and minimum X and Y values and the tic mark spacing. After plotting the grid and data points, the program will ask for the units for the X and Y labels. A special plot is produced when the CONT key is depressed in response to the question, "Consolidation Pressure?". Data will be plotted without asterisks and the program will ask for another file to be plotted on the same plot. This feature is used to plot the stress paths for several samples.

The following list gives the order in which the plots are stored in the seven files:

<u>File #</u>	<u>Plot</u>
1	Pore Pressure Vs. Strain
2	Dev. Stress vs. Strain
3	q' vs. $\bar{p}'$
4	A-Factor vs. Strain
5	Vert/Lat vs. Strain
6	Lat. Strain vs. Vert. Strain
7	Poisson's Ratio vs. Strain

A.11 Summary

Hopefully the instructions in this manual will prove to be helpful and contain sufficient detail to avoid great confusion. If questions arise concerning the HP programs, the programs in section A.13 should be reviewed.

A.12 Data Sheets (see attached)

- 1) Cover Sheet
- 2) Sample Data Sheet
- 3) Consolidation Sheet
- 4) Back Pressure and Response Sheet
- 5) Sample Summary Sheet

A.13 Computer Listing (see attached)

Sheet ____ of ____

University of Rhode Island
 Dept. of Ocean Engineering
 Marine Geomechanics Lab.

TRIAXIAL TEST

Sample _____ Date _____

Tested by _____

Depth _____

Project _____

Cell # 1 2 3 _____ Type of Test: CIU CAU _____

Top Drainage: Yes No Lateral Strain Measurement: Yes No

Method: _____ LSG: _____

Pressure Transducer I.D. _____ Constants _____

WATER CONTENT

LOCATION				
CONTAINER NO.				
WT. SAMPLE + TARE WET				
WT. SAMPLE + TARE DRY				
WT. OF WATER				
TARE WT.				
WT. OF DRY SOIL				
WATER CONTENT				

Sheet ____ of ____

Sample I.D.: _____

Initial Length (cm): _____

Initial Diameter (cm): _____

Sample Volume (cm³): _____

Weight of Tray (gm): _____

Weight of Tray and Sample (gm): _____

Weight of Sample (gm): _____

Bulk Density (gm/cm³): _____

of Filter Strips: _____

of Membranes: _____

Membrane Thickness (cm): _____

Comments:

Change in Sample Height:

Sighting on: _____

Initial Height Reading: _____

Final Height Reading: _____

Change in Height (cm): _____

Sketch Sample Below (after shearing)

Sheet ____ of ____

Sample I.D.: _____

Date: _____

Sample Back Pressure Schedule

Time	Cell Pressure			Back Pressure			Burette Reading	
	PSI	MV	MV:PSI	PSI	MV	MV:PSI	Left Bur.	Right Bur.

Sample Pore Pressure Response

Date: _____

Before Test: Cell Pressure ____ PSI ____ MV ____ MV:PSI

Back Pressure ____ PSI ____ MV ____ MV:PSI

Cell Pressure after ΔP ____ PSI ____ MV ____ MV:PSI $\Delta P =$ _____

After Test: Cell Pressure ____ PSI ____ MV ____ MV:PSI

Back Pressure ____ PSI ____ MV ____ MV:PSI

Time	Pore Pressure	
	MV	PSI
0		
.5		
1.0		
2.0		
3.0		
4.0		

____ % Response in ____ minutes.

Sheet ____ of ____

Summary Sheet

Sample: _____

Type of Test: _____ Project: _____

Sample I.D.: _____ Depth: _____

Date Tested: _____ Tested by: _____

Consolidation Pressure (PSI): _____ Back Pressure (PSI): _____

% Pore Press. Response _____ in _____ minutes

filter strips: _____ Membrane Modulus (tsf): _____

Membrane Thickness (cm): _____ Proving Ring #: _____

Strain Rate (mm/min): _____ % per hr.: _____

Initial Diameter (cm): _____ Initial Length (cm): _____

 Δ Vol. (cm³): _____ ΔH (cm): _____

Back Pressure Correction: _____

XDCR Constants: _____ LSG: _____

Tape# _____ Track # _____ File # _____ Data _____

Initial w% _____ w_f _____ σ_3 (Kg/cm²)Failure Criterion: ($\sigma_1 - \sigma_3$) max. $\bar{\sigma}_1/\sigma_3$ max. Other _____($\sigma_1 - \sigma_3$)_f _____ q_f _____ $\bar{p}_f$ _____A_f _____ ϵ_f _____ $\bar{\sigma}_1/\bar{\sigma}_3$ _f _____($\sigma_1 - \sigma_3$)_{ult} _____ E_i _____ R_f _____

```

0: "Response":
1: dim A$(2,20)
2: enr "Sample I.D.:",A$(1)
3: enr "Sample Depth",A$(2):ert A$(2)
4: enr "XDCR const:slope",r1
5: enr "XDCR const:inter",r2
6: enr "Enter Diff. XDCR? Yes=1,No=0",Z
7: cll 'init'
8: cll 'dum'(1,7)
9: dsp "Press CONT. to record P"istr
10: cll 'scn1'(3)
11: 3+I:fxd 6:ert 'store'(1,'rdum')
12: rlrI-r2+B:fmt "P = ,+6.2," psl": i 16:B:I+1-I
13: dsp "Set increased P: Press CONT. :str
14: cll 'scn1'(3)
15: ert 'store'(I,'rdum')
16: fmt "P+",b,"P = ,+6.2," psl":8+J
17: rlrI-r2+A:ert 16,J:A:I+1-I
18: dsp "Record Pore Pressure"istr
19: wait 5000:beep
20: cll 'scn1'(3)
21: cll 'store'(I,'rdum'):irlrI-r2+C:fxd 2:ert C:I+ I=5:C-D:I+1+I
22: fmt "Response:",f5,1," %"
23: if Z=0:ert 16,(C-D)/(A-B)*100
24: if Z=1:ert 16,(A-C)/(A-B)*100
25: wait 10000:jmp -5
26: end
27: "init":
28: if p0=0:rem 7:clr 7
29: fmt 9:f:dev "clk",708+p8,"ctr",725+p9,"dum",722+p10,"ptr",715+p11
30: dev "scn",708+p12,"scn1",710+p13,"scn2",711+p14,"scn3",712+p15
31: dev "sum",724+p16:8+p7
32: wtb p7:nds(7,p3,p4,p5)+p6:r0+2:(p7-8):bit(2,p5)+r0:1-p7-p7
33: if p7-17:eto -1
34: wtb "ptr.9",27:69
35: ret
36: "dum":
37: fmt 9:f:clr "dum":ert "dum.9",T3
38: if p1<1 or p1>5:1-p5:1+p1
39: if p2<1 or p2>7:p5+2+p5:7+p2
40: if p3<0 or p3>1:p5+4+p5:0+p3
41: fmt 9:"F",fz1,0,"R",fz1,0,"H",fz1,0,"I",fz1,0
42: ert "dum.9",int(p1),int(p2),int(p3),int(p4)
43: ret
44: "scn1":
45: if p0=0:wtb "scn:scn1:scn2:scn3":clr 731:ret
46: fmt 9:"0,1,2,3,4,5,6,7.",z:fmt 8:fz2,0,z
47: ert "scn:scn1:scn2:scn3.9"
48: if p0=0:fmt 9:f:ert "scn:scn1:scn2:scn3.9":ret
49: if p0>319:p0-1+p0:eto -1
50: if p0>239:ert "scn3.8",p0-240:p0-1+p0:eto -2
51: if p0>159:ert "scn2.8",p0-160:p0-1+p0:eto -3
52: if p0>79:ert "scn1.8",p0-80:p0-1+p0:eto -4
53: ert "scn.8",p0:p0-1+p0:eto -5
54: "rdum":
55: tra "dum":fmt 9,f:ired "dum.9",p1
56: ret p1
57: "store":
58: p2*1000+r:p1
59: ret p2
*4202

```



```

0: "TPIAX.init":
1: cll 'init'
2: cll 'dum'(1,7)
3: ent "Lateral Strain? Yes=1,No=0",2
4: dsp "Close drainage valve!!!!"istp
5: dsp "READY !!! ;Press CONTINUE!"istp
6: ent "Load File #? Abs=15,Dif=20",A:ld+ A
7: end
8: "init":
9: if p0=0;rem 7;clr 7
10: fct 9,f;dev "clk",700+p8,"ctr",725+p9,"dum",722+p10,"str",715+p11
11: dev "scn",709+p12,"sch1",710+p13,"sch2",711+p14,"sch3",712+p15
12: dev "sum",724+p16;8+p7
13: utb p6;f;ds(7,p3,p4,p5)+p6;r0+2;p7-8;bit(2,p5+r0;1+p7+p7
14: if p7<17;sto -1
15: utb "str.9",27,69
16: ret
17: "dum":
18: fct 9,f;clr "dum";wrt "dum.9","T3"
19: if p1<1 or p1>5;1+p5;1-p1
20: if p2<1 or p2>7;p5+2+p5;7-p2
21: if p3<0 or p3>1;p5+4+p5;0-p3
22: fct 9,"F",+z1.0,"R",+z1.0,"H",+z1.0,"D",+z1.0
23: wrt "dum.9",int(p1),int(p2),int(p3),int(p4)
24: ret
*31364

```

```

0: "TRI.RT.abs":
1: dim A[12,1],B[100,4],C[2]
2: ent "Track 0 or 1?",A:trk A
3: ent "Enter # of file to be loaded?",A
4: ldf A,A[*]
5: ent "Enter LSG calibration.",C[2]
6: .454/2.54+2+C
7: A[2,1]*C+A[2,1];A[1,1]*C+A[1,1]
8: A[7,1]-A[9,1]+D
9: ((A[6,1]+2/4)*A[7,1]-A[8,1])/D+E
10: r(4E/π)+F
11: A[4,1]*2000*.000488243+A[4,1]
12: fmt 11x,"STR",17x,"P",14x,"U",7x,"A",6x,"P/A",5x,"V/L";wrt 6
13: D*.2/100+Q
14: "count":I+1+I+C[1]
15: if I=1;eto "readings"
16: W+Q+W
17: wait 10000;c11 'scn1'(1);c11 'rdvm'(V)
18: if Z=9999;eto 58
19: fxd 3;dsp "Sample has strained ",((V-B[1,1])/6.496/D)*100," %"
20: if (V-B[1,1])/6.496<W;jmp -3
21: "readings";for J=1 to 4
22: c11 'scn1'(J)
23: fxd 6;prt 'array'(I,J,'rdvm')
24: if J=1;B[I,J]+r1
25: if I=1 and J=1;B[I,J]+r17
26: if J=2;B[I,J]+r3
27: if I=1 and J=2;B[I,J]+r0
28: if J=3;B[I,J]*1000+r5
29: if Z=0;jmp 2
30: if J=4;B[I,J]*1000+r18;if I=1;r18+r19
31: next J
32: (r1-r17)/6.496/D*100+r2
33: (r3-r0)/9.299*96.95+r4
34: (r5+A[11,1]-A[12,1]+A[10,1])/14.22+r6
35: E/(1-r2/100)+r7
36: .266*A[3,1]/r7*.070307+K
37: if I=1;0+K
38: A[5,1]/F+L
39: (1+2*(r2/100)-r(1/(1-r2/100)))*L+H
40: r4/r7-(K+2H)+r8
41: if I=1;r6+r16
42: r6-r16+r11
43: A[1,1]-r11+r9
44: r8+r9+r10
45: (r10-r9)/2+r12
46: (r10+r9)/2+r13
47: r10/r9+r15
48: if I=1;0+r14;jmp 2
49: r11/r8+r14
50: (r18-r19)*C[2]*2.54/F*100+r20
51: if r2=0;0+r21;jmp 2
52: 2/3*r20/r2+r21
53: fmt 1,/,1x,f7.3,2x,f6.2,2x,f7.3,6f8.3
54: fmt 2,4x,8f9.3
55: wrt 6,1,r1,r2,r3,r4,r5,r6,r7,r8,r15
56: wrt 6,2,r11,r10,r9,r12,r13,r14,r20,r21
57: if I<99;eto "count"
58: ent "Record data; Track 0 or 1?",L:trk L
59: ent "Enter # of last file on tape",L:ldf L
60: ent "Store output in file #?",N
61: if N<=L;dsp "!!! WARNING: You may be writing over files!";sto
62: if N<=L;eto 59
63: if N>L;rcf N;B[*],C[*]
64: ent "Data entered correctly?,Y=1,N=0",Z
65: if Z=0;eto 60
66: end
67: "scn1":
68: if p0=0;wtb "scn,scn1,scn2,scn3";clr 731;ret
69: fmt 9,"0,1,2,3,4,5,6,7,",z;fmt 8,fz2.0,z
70: wrt "scn,scn1,scn2,scn3.9"
71: if p0=0;fmt 9,f;wrt "scn,scn1,scn2,scn3.9";ret
72: if p0>319;p0-1+p0;eto -1
73: if p0>239;wrt "scn3.8",p0-240;p0-1+p0;eto -2
74: if p0>159;wrt "scn2.8",p0-160;p0-1+p0;eto -3
75: if p0>79;wrt "scn1.8",p0-80;p0-1+p0;eto -4
76: wrt "scn.8",p0;p0-1+p0;eto -5
77: "rdvm":
78: tra "dvm";fmt 9,f;fired "dvm.9",p1
79: ret p1
80: "array":
81: p3+B[p1,p2]
82: ret p3
*15

```

Figure A.1
Cell Manifold and Base

Figure A.2 Geonor Sample Stand

Figure A.3 Sample with Top Drainage

**Sample with Lateral Strain
Gauge and Metal Tabs
Attached**

Figure A.4

Figure A.5 Cell Pressure Line Attachment:
Absolute Pressure Transducer

Figure A.6 Cell Pressure Line Attachment:
Differential Pressure Transducer

- (1) Main Panel Air Shut Off Valve
- (2) Cell Pressure Air Supply Regulating Valve
- (3) Back Pressure Air Supply Regulating Valve
- (4) Cross-over Valve
- (5) Cell Pressure Regulator
- (6) Back Pressure Regulator
- (7) Regulating Valve to Cell Pressure Quick-Disconnect
- (8) Regulating Valve to Back Pressure Quick-Disconnect
- (9) Quick-Disconnect
- (10) Quick-Disconnect
- (11) Cell Pressure Regulating Valve
- (12) Back Pressure Regulating Valve
- (13) Switching Valve
- (14) Burrettes
- (15) Cell Pressure Reservoir
- (16) Back Pressure Reservoir

Figure A.7 Triaxial Pressure System

Figure A.8 Flowchart

REFERENCES

- Anderson, D.R. (1979a), "Subseabed Disposal Program Plan", SAND79-1157, Sandia Laboratories, Albuquerque, N.M.
- Anderson, D.R. (1979b), "Nuclear Waste Disposal in Subseabed Geologic Formations: the Seabed Disposal Program", SAND79-2211, Sandia Laboratories, Albuquerque, N.M.
- Bishop, A.W. and Henkel, D.J. (1962), "The Measurement of Soil Properties in the Triaxial Test", 2nd edition, London: Edward Arnold.
- Bishop, A.W. and Hight, D.W. (1977), "The Value of Poisson's Ratio in Saturated Soils and Rocks Stressed under Undrained Conditions", Geotechnique, Vol. 27, No. 3, pp. 369-384.
- Boyce, J.R. and Brown, S.F. (1976), "Measurement of Elastic in Granular Material", Geotechnique, Vol. 26, No. 3, pp. 637-640.
- Brown, S.F. and Snaith, M.S. (1974), "The Measurement of Recoverable and Irrecoverable Deformations in the Repeated Load Triaxial Test", Geotechnique, Vol. 24, No. 2, pp. 255-259.
- Chavez, P.F., Dawson, P.R., and Schuler, K.W. (1979), "Two-Dimensional Axisymmetric Penetrometer Simulations", Internal Report, Sandia Laboratories, Albuquerque, N.M.
- Christian, J.T., and Desai, C.S. (1977), "Constitutive Laws for Geologic Media", Numerical Methods in Geotechnical Engineering, edited by C.S. Desai and J.T. Christian, McGraw-Hill
- Daniel, P.E. and Olson, R.E. (1974), "Stress-Strain Properties of Compacted Clays", Journal of Geotechnical Engineering Division, ASCE, Vol. 100, No. GT10, Oct., pp. 1123-1136.
- Doyle, P.S. and Riedel, W.R. (1979), "Cretaceous to Neogene Ichtholiths in a Giant Piston Core from the Central North Pacific", Micropaleontology, Vol. 25, No. 4, pp. 337-364.
- Duncan, J.M. and Chang, C-Y (1970), "Nonlinear Analysis of Stress and Strain in Soils", Journal of the Soil Mechanics and Foundations Division, ASCE, Vol.

96, No. SM5, Sept., pp. 1629-1653.

- El-Ruwayih, A.A. (1976), "Design Manufacture and Performance of a Lateral Strain Device", Geotechnique, Vol. 26, No. 1, pp. 215-216.
- Hamilton, E.L. (1971), "Elastic Properties of Marine Sediments", Journal of Geophysical Research, Vol. 76, No. 2, pp. 579-604.
- Hardin, B.O. (1978), "The Nature of Stress-Strain Behavior for Soils", Proceedings of the Specialty Conference on Earthquake Engineering and Soil Dynamics, ASCE, Pasadena, Calif., June 19-21, 1978, pp. 3-90.
- Heath, G.R. (1978), "Geochemical and Sedimentological Assessment of Deep-Sea Sediments", Subseabed Disposal Program Annual Report: Jan. - Dec., 1978, Vol. 2, P.I. Progress Report; SAND79-1618, Sandia Laboratories, Albuquerque, N.M.
- Henkel, D.J. (1960), "The Shear Strength of Saturated Remoulded Clays", ASCE Research Conference on Shear Strength of Cohesive Soils, Boulder, Colorado, pp. 533-554.
- Hollister, C.D. (1978), "Geologic Siting Considerations for the Disposal of Radioactive Waste into Submarine Geologic Formations", Subseabed Disposal Program Annual Report: Jan. - Dec., 1978, Vol. 2, P.I. Progress Report; SAND79-1618, Sandia Laboratories, Albuquerque, N.M.
- Janbu, N. (1963), "Soil Compressibility as Determined by Oedometer and Triaxial Tests", European Conference on Soil Mechanics and Foundation Engineering, Wiesbaden, Germany, Vol. 1, pp. 19-25.
- Jurgenson, L. (1934), "The Shearing Resistance of Soils", Boston Society of Civil Engineers Journal, Vol. 21, No. 3, pp. 242-275.
- Kay, S.W. (1974), "Hondo - A Finite Element Computer Code for the Large Deformation Dynamic Response of Axisymmetric Solids", Internal Report, SLA-74-0039, Sandia Laboratories, Albuquerque, N.M.
- Kirkpatrick, W.M. and Belshaw, D.J. (1968), "On the Interpretation of the Triaxial Test", Geotechnique, Vol. 18, No. 3, pp. 336-350.
- Konder, R.L. (1963), "Hyperbolic Stress-Strain Response:

Cohesive Soils", Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 89, No. SM1, Feb., pp. 115-143.

- Krizek, R.J. (1977), "Fabric Effects on Strength and Deformation of Kaolin Clay", 9th International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, pp. 169-176.
- Lambe, T.W. (1951), Soil Testing for Engineers, John Wiley & Sons, Inc.
- Lambe, T.W. and Whitman, R.V. (1969), Soil Mechanics, John Wiley & Sons, Inc.
- Lee, H.J. (1974), "The Hole of Laboratory Testing in the Determination of Deep-Sea Sediment Engineering Properties", Deep-Sea Sediments; Physical and Mechanical Properties, edited by A.L. Inderbitzen, Plenum Press, pp. 111-126.
- Lee, H.J. and Hamilton, E.L. (1974), "Geotechnical Properties of the Deep Sea Floor, Northeast Pacific", an addendum to Marine Geology of a Region in the Northeast Pacific: Data Report (unpublished).
- Menzies, B.K. (1976), Discussion of "Design Manufacture and Performance of a Lateral Strain Device", by El-Ruwayih, A.A., Geotechnique, Vol. 26, No. 2, pp. 542-544.
- Meyerhof, G.G. (1979), "Geotechnical Properties of Offshore Soils", First Canadian Conference on Marine Geotechnical Engineering.
- Micro-Measurements Catalog (1979); Measurement Group, Vishay Intertechnology, Inc.; Raleigh, North Carolina.
- Mitchell, J.K. (1976), Fundamentals of Soil Behavior, John Wiley & Sons, Inc.
- Noorany, I. (1971), "Engineering Properties of Submarine Clays from the Pacific", Proceedings of the First International Conference on Port and Ocean Engineering under Arctic Conditions, Trondheim, Norway, (reprint) pp. 1-21.
- Olson, R.E. (1974), "Shearing Strengths of Kaolinite, Illite, and Montmorillonite", Journal of the Geotechnical Engineering Division, ASCE, Vol. 100, GT11, pp. 1215-1229.

- Roscoe, K.H., Schofield, A.N., and Thurairajah, A. (1963), "An Evaluation of Test Data for Selecting a Yield Criterion for Soils", ASTM, Laboratory Shear Testing of Soils, STP No. 361, pp. 111-128.
- Sechler, E.E. (1952), Elasticity in Engineering, John Wiley & Sons, Inc.
- Schuler, K.W. (1978), "Lateral Deformation Gauge for Rock Mechanics Testing", Internal Report, Sandia Laboratories, Albuquerque, N.M.
- Silva, A.J. (1977), "Physical Processes in Deep Sea Clays", Oceanus, Vol. 20, No. 1, pp. 31-40.
- Silva, A.J. (1979), "Geotechnical Properties of Deep-Sea Clays; A Brief Discussion", First Canadian Conference on Marine Geotechnical Engineering.
- Silva, A.J., Hollister, C.D., Laine, E.P., and Beverly, B.E. (1976), "Geotechnical Properties of Deep Sea Sediments: Bermuda Rise", Marine Geotechnology, Vol. 1, No. 3, pp. 195-232.
- Silva, A.J. and Calnan, D.I. (1978), "Geotechnical Aspects of Subsurface Seabed Disposal of High Level Radioactive Wastes", Annual Progress Report No. 4: Jan. - Dec. 1977, to DOE, University of Rhode Island.
- Silva, A.J. and Clukey, E.C. (1975), "Geotechnical Properties of Ocean Sediments as Related to Disposal of Radioactive Material", Seabed Disposal Program: A First Year Report, December, 1974, edited by W.P. Bishop, SAND74-0410, pp. 243-268.
- Silva, A.J. and Hollister (1973), "Geotechnical Properties of Ocean Sediments Recovered with Giant Piston Corer: 1. Gulf of Main", Journal of Geophysical Research, Vol. 78, No. 18, June, pp. 3597-3616.
- Silva, A.J. and Hollister, C.D. (1979), "Geotechnical Properties of Ocean Sediments Recovered with the Giant Piston Corer: Blake-Bahama Outer Ridge", Marine Geology, Vol. 29, pp. 1-22.
- Skempton, A.W. (1954), "The Pore-Pressure Coefficients A and B", Geotechnique, Vol. 4, pp. 143-147.
- Terzaghi, K. and Peck, R.B. (1967), Soil Mechanics in Engineering Practice, John Wiley & Sons, Inc.
- Vagneron, J., Lade, P.V., and Lee, K.L. (1976), "Evaluation

of Three Stress-Strain Models for Soils", Numerical Methods in Geomechanics, Vol. 3, edited by C.S. Desai, published by ASCE.

Whitman, R.V. (1960), "Some Considerations and Data Regarding the Shear Strength of Clays", ASCE Research Conference on Shear Strength of Cohesive Soils, Boulder, Colorado, pp. 581-614.

Wong, K.S. and Duncan, J.G. (1974), "Hyperbolic Stress-Strain Parameters for Nonlinear Finite Element Analyses of Stresses and Movements in Soil Masses", Report No. TE-74-3 to NSF, Univ. of California, Berkeley, Calif.

Wong, P.K.K. and Mitchell, R.J. (1975), "Yielding and Plastic Flow of Sensitive Cemented Clay", Geotechnique, Vol. 25, No. 4, pp. 763-782.