STRESS-STRAIN-STRENGTH ANISOTROPY OF VARVED CLAYS

by

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Surachat Sambhandharaksa

Submitted to the Department of Civil Engineering on August, 1977 in partial fulfillment of the requirements for the Degree of Doctor of Science.

ABSTRACT

The objectives of this thesis are: (1) to contribute to a fundamental understanding of the nature of undrained shear strength anisotropy of varved clays; (2) to present best estimates of normalized stress-strain-strength parameters for undrained total stress analyses and for predictions of pore pressures and undrained deformations used in the design of structures on Connecticut Valley varved clays, with background information showing how these parameters were developed; and (3) to recommend suitable testing procedures for developing normalized soil parameters for other varved clay deposits.

To achieve these objectives, several types of K_o consolidated and isotropically consolidated strength tests were performed at various overconsolidation ratios (OCR). These include: (1) SHANSEP $\overline{CIUC}$ tests on Amherst and East Windsor samples with different inclinations; (2) SHANSEP $\overline{CK_oUC}$ and $\overline{CK_oUE}$ tests on vertical Amherst and East Windsor samples; (3) SHANSEP $\overline{CK_oUDSS}$ tests on vertical Northampton, Amherst, and East Windsor samples; and (4) SHANSEP $\overline{CIUE}$ tests on vertical and horizontal East Windsor samples. The testing program also includes $\overline{CK_oU}$ plane strain tests on normally consolidated Northampton samples and $\overline{CK_oUDSS}$ tests on normally consolidated inclined East Windsor samples. For the study of suitable testing procedures of varved clay, RECOMPRESSION $\overline{CK_oU}$ triaxial compression and extension tests at OCR's of 2 and 4, and RECOMPRESSION $\overline{CIUC}$ tests on East Windsor samples with different inclinations at OCR=4 (the in situ OCR) were performed along with UUC tests on East Windsor samples with different inclinations.

The three factors that control the anisotropic strength behavior of varved clays are: (1) the layered nature of the soil (material anisotropy); (2) the prior K_o consolidation (structural anisotropy); and (3) when $K_o \neq 1$, the stress system induced anisotropy. An attempt was made to separate the influence on undrained shear strength (q_f) of material anisotropy from the effects due to structural and stress system induced anisotropy.

The anisotropy in $q_f/\bar{\sigma}_{vm}$ is due to the anisotropy in pore pressure response and in the normalized effective stress envelope (NESE) at q_f, the plot of $q_f/\bar{\sigma}_{vm}$ versus $\bar{p}_f/\bar{\sigma}_{vm}$. A major finding is the fact the anisotropy in NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ is only due to material anisotropy. Thus, there are three possible NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$,

namely: NESE for failure across the varves, that for failure within a clay layer, and that for failure within a silt layer. The first two NESE are shown to be basic material properties.

Test data on a block sample from East Windsor show that disturbance is mostly due to separation of the varves and is therefore directional dependent. With block samples, the SHANSEP method of consolidation is thought to yield reasonable estimates of undrained shear strength but the stress-strain characteristics are less brittle and less sensitive compared to the in situ soil.

Chapter 8 presents: recommended normalized soil parameters for the design of structures on Connecticut Valley varved clays; type of tests for checking these parameters; recommended testing procedures for developing design strengths for other varved clay deposits; and suitable methods of stability analysis wherein anisotropic strengths are taken into consideration.

Thesis Supervisor — Charles C. Ladd
Title — Professor of Civil Engineering

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TABLE OF CONTENTS

Page

TITLE PAGE. i

ABSTRACT ii

ACKNOWLEDGEMENTS iv

TABLE OF CONTENTS vi

LIST OF TABLES. x

LIST OF FIGURES xiv

1 INTRODUCTION 1

1.1 Background 1
1.2 Objectives 2
1.3 Scope 3

2 BACKGROUND 6

2.1 Introduction 6
2.2 Background 6
2.2.1 Anisotropy in Undrained Shear Strength . . 6
2.2.2 Measurement of Undrained Shear Strength Anisotropy 9
2.2.3 The Normalized Soil Properties (NSP) Concept 13
2.3 Literature Review 17
2.3.1 Analytical Approach 18
2.3.1(a) Curve Fitting Method 18
2.3.1(b) Theoretical Prediction of Anisotropic Behavior 19
2.3.1(c) Factors Affecting the Anisotropic Shear Strength Behavior of Varved Clay 21
2.3.2 Experimental Approach 23
2.3.2(a) Evidence from $\overline{UUC}$ Tests 23
2.3.2(b) Evidence from $\overline{CIUC}$ Tests 25
2.3.2(c) Evidence from $\overline{CK_oU}$ Tests 29
2.3.3 Summary and Conclusions 32
2.3.4 Effect of Certain Stress System Variables on Undrained Shear Strength Behavior. . . . 34
2.3.4(a) Effect of Intermediate Principal Stress 35
2.3.4(b) Effect of K_o Consolidation . . . 36

3 EXPERIMENTAL PROGRAM 60

Page

3.1 General Scope and Objectives 60
3.2 Experimental Program 60
3.3 Description of Varved Clays 64
3.3.1 East Windsor Varved Clay 66
3.3.2 Amherst Varved Clay 67
3.3.3 Northampton Varved Clay 68
3.3.4 Hackensack Valley Varved Clay 70
3.4 Summary of Soil Properties 71
3.5 General Testing Procedures 72
3.5.1 Consolidated Undrained Triaxial Tests . . . 73
3.5.2 Testing Problems with Consolidated Undrained Triaxial Tests. 77
3.5.3 The Direct Simple Shear Test 78
3.5.4 The Plane Strain Tests 79

4 COMPARISON OF NORMALIZED BEHAVIOR OF DIFFERENT VARVED CLAYS. 100

4.1 Introduction and Background. 100
4.2 Effect of Sample Stress History (MPPR) 102
4.3 Effect of Soil Location 105
4.3.1 Results from $\overline{CIU}$ Tests. 105
4.3.2 Results from $\overline{CK_oU}$ Tests 107
4.3.2(a) Data from $\overline{CK_oU}C$ Tests 107
4.3.2(b) Data from $\overline{CK_oU}$DSS Tests 108
4.3.2(c) Data from $\overline{CK_oU}E$ Tests 109
4.4 Discussion 111
4.5 Summary and Conclusions. 115

5. EFFECT OF STRESS SYSTEM ON UNDRAINED BEHAVIOR FROM SHANSEP SAMPLES 145

5.1 Introduction 145
5.2 Effect of K_o Versus Isotropic Consolidation. . . . 148
5.2.1 Method of Investigation 148
5.2.2 Volume Change Data from $\overline{CIU}$ and $\overline{CK_oU}$ Tests 149
5.2.3 Results from $\overline{CIU}C$ and $\overline{CK_oU}C$ Tests 150
5.2.4 Results from $\overline{CIU}E$ and $\overline{CK_oU}E$ Tests 154
5.2.5 Summary and Discussion. 157
5.3 Effect of Intermediate Principal Stress. 160
5.3.1 Method of Investigation 160
5.3.2 Results from $\overline{CIU}$ Tests 161
5.3.3 Results from K_o Consolidated Plane Strain and Triaxial Tests 165
5.3.4 Interpretation and Results from Special DSS Tests 170
5.3.5 Summary and Discussion. 181
5.4 Effect of Testing Method for Shearing Parallel to Varves 182
5.4.1 Introduction and Method of Investigation 182

Page

5.4.2 Results from CDDS and $\overline{CIUC}$ Tests 184
5.4.3 Results from $\overline{CK_oUDSS}$ Tests and Their Interpretation 186
5.4.4 Comparison of Results from $\overline{CIUC}$ (θ=45°) and $\overline{CK_oUDSS}$ Tests 188
5.5 General Discussion 190
5.6 Summary and Conclusions. 191

6 CONSIDERATION OF SAMPLE DISTURBANCE 251

6.1 Introduction and Background 251
6.1.1 Purpose 251
6.1.2 Types and Causes of Sample Disturbance . . 251
6.1.3 General Effect of Sample Disturbance . . . 252
6.1.4 Methods for Minimizing Sample Disturbance
6.1.5 Experimental Program 256
6.2 Effect of Methods for Minimizing Disturbance on Undrained Shear Strength Behavior 256
6.2.1 Behavior from Vertical Loading. 257
6.2.2 Behavior from Horizontal Loading. 260
6.2.3 Behavior from Inclined Loading. 262
6.3 Discussion 263
6.4 Summary and Conclusions. 266

7 INTERPRETATION OF ANISOTROPIC BEHAVIOR OF CONNECTICUT VALLEY VARVED CLAYS 292

7.1 Introduction 292
7.2 Method of Investigation 294
7.3 Interpretation of "Material" Anisotropic Behavior 296
7.4 Effect of Structural Aniostropy Component 299
7.5 Interpretation of Combined Anisotropy 301
7.6 Summary and Conclusions. 305

8 EVALUATION OF UNDRAINED STRESS-STRAIN-STRENGTH PROPERITES OF CVVC FOR USE IN DESIGN 322

8.1 Objectives and Scope 322
8.2 Development of Undrained Strength Parameters of CVVC for Total Stress Stability Analyses Using SHANSEP. 323
8.2.1 Criteria for Developing Design Undrained Strength Parameter 324
8.2.2 Anisotropic S_u Parameters Developed by Ladd (1975) 327
8.2.3 Design Parameters Developed by the Author
8.3 Comparison of the Use of SHANSEP Anisotropic S_u Versus Corrected FV Strength Data 331
8.3.1 Introduction 331
8.3.2 Recommended Correction Factor (μ) for Connecticut Valley Varved Clay. 332

Page

8.3.3 Compatibility of Using SHANSEP Anisotropic S_u with M-P Analyses 334
8.4 Evaluation of Undrained Shear Strength for Total Stress Analyses in Practice 335
8.4.1 Initial Undrained Shear Strength for Total Stress Analyses (Undrained Case) . . 335
8.4.2 Undrained Shear Strength for Total Stress Analysis in Partially Drained Case 336
8.5 Recommended Parameters for Undrained Analyses of Pore Pressure and Deformation. 339
8.5.1 Parameters for Predictions Using Elastic Theory 339
8.5.2 Parameters for Finite Element Program FEECON 343
8.6 Recommended Procedures for Checking CVVC NSP and For Developing Normalized Shear Strength Parameters for Other Varved Clays 345
8.6.1 Recommended Method for Checking Soil Parameters 346
8.6.2 Recommended Laboratory Testing Procedures for Developing Normalized Soil Parameters for Other Varved Clay Deposits 346

9 SUMMARY AND CONCLUSIONS 369

10 REFERENCES 381

BIBLIOGRAPHY 387

APPENDIX A - NOTATION 388

APPENDIX B - BEST ESTIMATE OF STRESS-STRAIN-STRENGTH DATA OF CONNECTICUT VALLEY VARVED CLAYS. . . . 395

APPENDIX C - SUMMARY OF TEST RESULTS FROM CONNECTICUT VALLEY AND HACHENSACK VALLEY VARVED CLAYS 406

APPENDIX D - TABULATION OF TEST RESULTS FROM AMHERST AND EAST WINDSOR VARVED CLAYS 431

LIST OF TABLES

Table No.		Page
2.1	General Problems with Testing Methods for Investigating Direction Variation of Undrained Shear Strength (S_u)	38
2.2	Prediction of Anisotropy in S_u from Hansen and Gibson's Equation for Plane Strain Loading	39
2.3	Influence of Inherent Anisotropy on Measured Undrained Shear Strengths of Homogeneous Clay from UUC Tests	40
2.4	Influence of Inherent Aniostropy on Measured Undrained Shear Strength of Non-Homogeneous Clay from UUC Tests	41
2.5	Anisotropy in S_u Measured from $\overline{CK_oU}$PSC, $\overline{CK_oU}$DSS, $\overline{CK_oU}$PSE Tests (Option 4) of Several NC Cohesive Soils	42
2.6	Anisotropy in S_u Measured from $\overline{CK_oU}$C, $\overline{CK_oU}$E, and $\overline{CK_oU}$DSS Tests (Option 3) of Several Soft Clays	43
2.7	Summary of Anisotropic Data from Several NC Cohesive Soils	44
2.8	Effect of Stress History on Anisotropic Shear Strength Behavior of Boston Blue Clay Measured from Plane Strain Compression and Extension Tests	45
2.9	Effect of σ_2 on Vertical and Horizontal Undrained Shear Strength Behavior of Several NC Cohesive Soils	46
3.1	Summary of the Experimental Program	83
3.2	Summary of Index Properties and Engineering Properties of East Windsor Varved Clay	85
3.3	Laboratory Testing Program of East Windsor Varved Clay	86
3.4	Laboratory of Testing Program of Amherst Varved Clay	87
3.5	Laboratory Testing Program of Northampton Varved Clay	88

Table No.		Page
3.6	Laboratory Testing Program of Hackensack Valley Varved Clay	89
3.7	Summary of the Index Properties of Varved Clay Samples from Amherst, East Windsor, Northampton and Hackensack Valley	90
3.8	Comparison of the Undrained Shear Strengths from the $\overline{CK_oUE}$ Tests from Different Testing Procedures	91
4.1	Experimental Program for Investigating the Effects of Soil Samples' Stress History and Soil Location on NSP	118
4.2	Effect of Sample Stress History on the NSP of Amherst Varved Clay	119
4.3	Effect of Stress History on the NSP of Northampton Varved Clay	120
4.4	Effect of Sample Stress History on the NSP of East Windsor and Hackensack Valley Varved Clay	121
4.5	Effect of Soil Location on NSP from $\overline{CIUC}$ Tests on Normally Consolidated Samples.	122
4.6	Effect of Soil Location on NSP from $\overline{CK_oUC}$ Tests	123
4.7	Effect of Soil Location on the NSP from $\overline{CK_oUDSS}$ Tests	124
4.8	Undrained Strength Data on Samples from Two Depths of Matagami Varved Clay	125
5.1	Experimental Program for Investigating the Effect of Stress System on SHANSEP NSP	196
5.2	Comparison of NSP from $\overline{CIUC}$ and $\overline{CK_oUC}$ Tests on Connecticut Valley Varved Clays	197
5.3	Comparison of NSP from $\overline{CIUE}$ and $\overline{CK_oUE}$ Tests on Connecticut Valley Varved Clays	198
5.4	Comparison of NSP from $\overline{CIUE}$ (θ=90°) and $\overline{CIUC}$ (θ=0°) Tests on Connecticut Valley Varved clays (Vertical Loading)	199
5.5	Comaprison of NSP from $\overline{CIUE}$ (θ=0°) and $\overline{CIUC}$ (θ=90°) Tests on Connecticut Valley Varved clays (Horizontal Loading)	200

Table No.		Page
5.6	Comparison of NSP from $\overline{CK_oU}$ Triaxial, Plane Strain, and Special DSS Tests on Normally Consolidated Samples	201
5.7	Load Increments in $\overline{CK_oU}$DSS(C) and $\overline{CK_oU}$DSS(E) Tests on Normally Consolidated Inclined Samples	202
5.8	Estimation of A_f from DSS Test Sample (θ=45°) Using $\overline{NESE}$ Assumption	203
5.9	Summary of the Effect of $\Delta\sigma_2$ on Vertical Loading	204
5.10	Summary of the Effect of $\Delta\sigma_2$ on Horizontal Loading	205
5.11	Summary of NSP from $\overline{CIUC}$ Tests on Inclined Samples (θ=45°)	206
5.12	Results from Drained Direct Shear Tests on Northampton Varved Clay for Failure Within a Clay Layer	207
5.13	Summary of the Test Results from $\overline{CK_oU}$DSS Tests	208
5.14	Comparison of Typical Values of $q_f/\bar{\sigma}_{vc}$, $\tau_{ff}/\bar{\sigma}_{vc}$ at $(\bar{\sigma}_1/\sigma_3)_{max}$ and A_f from $\overline{CIUC}$ Tests on Inclined Samples and $\overline{CK_oU}$DSS Tests	209
6.1	Experimental Program for Investigating the Effect of Method for Minimizing Disturbance on Undrained Shear Strength Behavior	271
6.2	Comparison of Test Results from $\overline{CK_oU}$ Tests on SHANSEP and RECOMPRESSION Samples from Connecticut Valley	272
6.3	Comparison of Test Results from $\overline{CIUC}$ Tests on SHANSEP and RECOMPRESSION Samples	273
7.1	Summary of Test Data from $\overline{CIUC}$ Tests for the Investigation of "Material" Anisotropy	308
7.2	Summary of Measured and Estimated Data from $\overline{CK_oU}$PSC, $\overline{CK_oU}$DSS, and $\overline{CK_oU}$PSE Tests for the Study of Combined Anisotropy	309
8.1	Types of Stability Analyses and Recommendations for Embankments on Connecticut Valley Varved Clays	349

Table No.		Page
8.2	Input Parameters for Connecticut Valley Varved Clays for Undrained Deformation and Pore Pressure Analyses Using FEECON	350
8.3	Soil Parameters for the Estimation of Initial Settlement and Pore Pressure	351
8.4	Determination of S_u at OCR = 1.0 from Plane Strain Loading Using Strain Compatibility Method	352
8.5	Determination of S_u at OCR = 1.0 from Triaxial Loading Using Strain Compatibility Method	353
8.6(a)	Determination of S_u by Using Strain Compatibility Method for Triaxial Loading at OCR=2	354
8.6(b)	Determination of S_u by Using Strain Compatibility Method for Triaxial Loading at OCR=4	355
8.7	Determination of S_u by the Author's Approach	356
8.8	Comparison of S_u values from Ladd (1975), Author, and Corrected Field Vane Strengths	356

LIST OF FIGURES

Fig. No.		Page
1.1	Locations of Soil Samples of Varved Clays from Connecticut Valley	5
2.1	Definition of Undrained Shear Strength Anisotropy	47
2.2(a)	In Situ and Applied Stresses in Field Situation of Elements A, B, and C.	48
2.2(b)	Testing Methods for Investigating the Undrained Shear Strength Anisotropy of Varved Clays in the Laboratory	49
2.3	Davis-Christian (1971) Elliptical Anisotropic Strength Relationship	50
2.4	Modified Mohr-Coulomb Failure Criteria for Cross-Anisotropic Soil (After Merkle, 1971)	51
2.5	Summary of Results from UUC Tests on Connecticut Valley Varved Clays (Data from Healy, 1967; Ladd and Wissa, 1970; and Long, 1975)	52
2.6	Anisotropic Behavior of London Clay at the Site of the Ashford Common Shaft Measured from $\overline{CIUC}$ Tests on Vertical and Horizontal Samples (Data from Bishop et al., 1965)	53
2.7	Test Data from $\overline{CIUC}$ Tests on OC Welland Clay Samples with Different Inclinations (From Lo, 1965; and Lo, 1966)	54
2.8	Anisotropy in q_f and A_f of Fulford Soft Clayey Silt Measured from $\overline{CIUC}$ tests on Samples with Different Inclinations (Data from Perry and Nadarajah, 1974)	55
2.9	Test Data from $\overline{CIUC}$ Tests on Laboratory Prepared Vertical and Horizontal Samples of OC Kaolin (Duncan and Seed, 1966)	56
2.10	Test Data from $\overline{CIUC}$ Tests on Horizontal and Vertical Samples of a Laboratory Prepared Lightly Overconsolidated Kaolin (Data from Mitchell, R.J., 1972)	57
2.11	Effect of $\bar{\sigma}_c$ on the Measurement of Undrained Shear Strength Anisotropy of Connecticut Valley Varved Clay (After Long, 1975)	58

Fig. No.		Page
2.12	Anisotropy in S_u ($S_u = \tau_{\alpha f_{max}}$) from $\overline{CK_oUDSS}$ Tests on Samples with Different Inclinations (Option 5) of Three Soft Clays (Data from Soydemir, 1972)	58
2.13	Anisotropic Undrained Strength Ratio Versus Plasticity Index	59
3.1	Results of Consolidation Tests on East Windsor Varved Clay	92
3.2	Soil Profile, Atterberg Limits, Recompression Ratio, and Virgin Compression Ratio of Amherst Varved Clay	93
3.3	Soil Profile, Stress History and Undrained Strength Data at Amherst, Massachusetts	94
3.4	Summary of Atterberg Limits and Compressibility of Northampton Varved Clay (From Ladd and Wissa, 1970)	95
3.5	Summary of Stress History, Undrained Shear Strength of Northampton Varved Clay (From Ladd and Wissa, 1970)	96
3.6	Summary of Consolidation Data at Boring 65 and 65A, Secaucus, New Jersey (from Saxena, et al., 1974)	97
3.7	Schematic Diagram of Essential Elements in Triaxial Extension Test	98
3.8	Diagramatic Drawing of Plane Strain Sample and Loading Systems	99
4.1	Normalized Stress-Strain Curves from $\overline{CK_oUC}$ Tests on Overconsolidated Amherst Varved Clay at Two $\bar{\sigma}_{vm}(L)/\bar{\sigma}_{vm}$ Ratios	126
4.2	Normalized Stress-Strain Curves from $\overline{CK_oUDSS}$ Tests on Normally Consolidated Northampton Varved Clay	127
4.3	Effect of MPPR on Values of $3G/\bar{\sigma}_{vc}$ from $\overline{CK_oUDSS}$ Tests on Northampton, Amherst, East Windsor and Hackensack Valley Varved Clays	128
4.4	Comparison of Stress-Strain Curves from $\overline{CIUC}$ and $\overline{CIUE}$ Tests on Normally Consolidated Vertical Samples from Amherst, East Windsor and Hackensack Valley	129

Fig. No.		Page
4.5	$\bar{\sigma}_1/\bar{\sigma}_3$ - ε Curves from $\overline{CIUC}$ and $\overline{CIUE}$ Tests on Normally Consolidated Vertical Samples from Amherst, East Windsor and Hackensack Valley	130
4.6	Comparison of Stress-Strain Curves from $\overline{CIUC}$ Tests on Inclined Samples (45°) of Normally Consolidated Amherst and East Windsor Varved Clays	131
4.7	Comparison of Stress-Strain Curves from $\overline{CIUC}$ Tests on Horizontal Samples of Normally Consolidated Amherst and East Windsor Varved Clays	132
4.8	Normalized Stress-Strain Curves from $\overline{CK_oUC}$ Tests on Normally Consolidated Vertical Samples from Connecticut Valley	133
4.9	Normalized Effective Stress Paths from $\overline{CK_oUC}$ Tests on Normally Consolidated Vertical Samples from Connecticut Valley	134
4.10	p_f Versus q_f at Maximum Shear Stress from $\overline{CK_oUC}$ Tests on Normally Consolidated Vertical Samples from Connecticut Valley	135
4.11	Comparison of Normalized Stress-Strain Curves from $\overline{CK_oUC}$ Tests on Overconsolidated Amherst and East Windsor Varved Clays	136
4.12	A_f and ε_f Versus OCR from $\overline{CK_oUC}$ Tests on Amherst and East Windsor Varved Clays	137
4.13	Effects of Soil Location on Normalized Effective Stress Envelope at Maximum Shear Stress	138
4.14	Normalized Undrained Shear Strength of Connecticut Valley and Hackensack Valley Varved Clays	139
4.15	Normalized Stress-Strain Curves from $\overline{CK_oUDSS}$ Tests on Normally Consolidated Northampton, Amherst, East Windsor and Hackensack Valley Varved Clays	140
4.16	Normalized $\tau_{h_{max}}/\bar{\sigma}_{vm}$ Versus $\bar{\sigma}_v/\bar{\sigma}_{vm}$ of Connecticut Valley and Hackensack Valley Varved Clays from $\overline{CK_oUDSS}$ Tests	141
4.17	Normalized Stress-Obliquity-Strain from $\overline{CK_oUE}$ Tests on Overconsolidated Amherst and East Windsor Varved Clays	142

Fig. No.		Page
4.18	A_f and ε_f Versus OCR from $\overline{CK_oUE}$ Tests of East Windsor and Amherst Varved Clays	143
4.19	Comparison of the Normalized Undrained Shear Strengths of Connecticut Valley, Matagami and New Liskeard Varved Clays	144
5.1	Volume Change Data from $\overline{CIU}$ and $\overline{CK_oU}$ Tests with Horizontal Varved (θ=0°) Samples from East Windsor	210
5.2	K_o Versus OCR for Connecticut Valley Varved Clay	211
5.3	Normalized Stress-Strain Data from $\overline{CIUC}$ and $\overline{CK_oUC}$ Tests on Normally Consolidated Amherst Varved Clay	212
5.4	Normalized Stress-Strain Curves from $\overline{CIUC}$ and $\overline{CK_oUC}$ Tests on Overconsolidated (OCR=4) East Windsor Varved Clay	213
5.5	A_f Versus OCR from $\overline{CIUC}$, $\overline{CK_oUC}$, $\overline{CIUE}$ and $\overline{CK_oUE}$ Tests on Connecticut Valley Varved Clays	214
5.6	ε_f Versus OCR from $\overline{CIUC}$, $\overline{CK_oUC}$, $\overline{CIUE}$ and $\overline{CK_oUE}$ Tests on Connecticut Valley Varved Clay	215
5.7	Typical Normalized Effective Stress Paths for $\overline{CIU}$ and $\overline{CK_oU}$ Triaxial Compression and Extension Tests on Normally Consolidated and Overconsolidated Connecticut Valley Varved Clays	216
5.8	Normalized Effective Stress Envelopes from $\overline{CIUC}$ and $\overline{CK_oUC}$ Tests on Connecticut Valley Varved Clays	217
5.9	E_u/q_f Versus $\Delta q/\Delta q_f$ from $\overline{CIUC}$ and $\overline{CK_oUC}$ Tests on Samples from Connecticut Valley	218
5.10	$E_u/\bar{\sigma}_{vc}$ at ε = 0.2% Versus OCR from $\overline{CIUC}$, $\overline{CK_oUC}$, $\overline{CIUE}$, and $\overline{CK_oUE}$ Tests on Samples from East Windsor	219
5.11	Normalized Stress-Strain Curves from $\overline{CIUE}$ and $\overline{CK_oUE}$ Tests on Normally Consolidated East Windsor Varved Clay	220
5.12	Normalized Stress-Strain Curves from $\overline{CIUE}$ and $\overline{CK_oUE}$ Tests on Overconsolidated (OCR=4) Connecticut Valley Varved Clays	221

Fig. No.		Page
5.13	E_u/S_u Versus $\Delta q/\Delta q_f$ from $\overline{CIUE}$ and $\overline{CK_oUE}$ Tests on Samples from East Windsor	222
5.14	States of Stress for $\overline{CIUC}$ (θ=0°), $\overline{CIUE}$ (θ=0°), $\overline{CIUE}$ (θ=90°) and $\overline{CIUC}$ (θ = 90°) Test Samples	223
5.15(a)	Normalized Stress-Strain Curves for Normally Consolidated $\overline{CIUE}$ (θ=90°) and $\overline{CIUC}$ (θ=0°) Test Samples (Vertical Loading)	224
5.15(b)	Normalized Stress-Strain Curves for Overconsolidated $\overline{CIUE}$ (θ=90°) and $\overline{CIUE}$ (θ=0°) Test Samples (OCR=4)	225
5.16(a)	Normalized Stress-Strain Curves for Normally Consolidated $\overline{CIUE}$ (θ=0°) and $\overline{CIUC}$ (θ=90°) Test Samples (Horizontal Loading)	226
5.16(b)	Normalized Stress-Strain Curves for Overconsolidated $\overline{CIUE}$ (θ=0°) and $\overline{CIUC}$ (θ=0°) Test Samples (OCR = 4)	227
5.17	Normalized Effective Stress Envelopes from $\overline{CIUC}$ and $\overline{CIUE}$ Tests on Vertical and Horizontal Samples of Connecticut Valley Varved Clays	228
5.18	Effect of Intermediate Principal Stress on Normalized Octahedral Stress Paths	229
5.19	Effects of σ_2: Normalized Effective Stress Paths from $\overline{CIUC}$ and $\overline{CIUE}$ Tests on Vertical and Horizontal Samples of East Windsor Varved Clay	230
5.20	Normalized Stress-Strain Curves of Normally Consolidated Samples from $\overline{CK_oUC}$ and $\overline{CK_oUPSC}$ Tests	231
5.21	Normalized Stress-Strain Curves of Normally Consolidated Samples from $\overline{CK_oUE}$ and $\overline{CK_oUPSE}$ Tests	232
5.22	Normalized Effective Stress Paths for Plane Strain and Triaxial Shear or Normally Consolidated Connecticut Valley Varved Clays	233
5.23	Comparison of $E_u/\bar{\sigma}_{vc}$ from $\overline{CK_oU}$ Plane Strain, Triaxial and Special DSS Tests	234
5.24	Estimation of $q_f/\bar{\sigma}_{vc}$ of Overconsolidated Plane Strain Horizontal and Vertical Loading Samples from Connecticut Valley Varved Clays	235

Fig. No.		Page
5.25	Assumed State of Stress (Pure Shear) for Measuring S_u(V) and S_u(H) of Inclined (45°) Normally Consolidated Samples from $\overline{CK_oU}$DSS(C) and $\overline{CK_oU}$DSS(E) Tests	236
5.26	NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ for Failure Across the Varves	237
5.27	Normalized Stress-Strain Curves of Normally Consolidated Inclined Samples from $\overline{CK_oU}$DSS(C) and $\overline{CK_oU}$DSS(E) Tests	238
5.28	Comparison of $q_f/\bar{\sigma}_{vc}$ from $\overline{CK_oU}$ Triaxial Tests and $\overline{CK_oU}$DSS Test on Inclined Samples from East Windsor (Using $\overline{NESE}$ Assumption)	239
5.29	Comparison of $q_f/\bar{\sigma}_{vc}$ Values from $\overline{NESE}$ Assumption and the Pure Shear Assumption	240
5.30	NESE at q_f and $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ from $\overline{CIUE}$ Tests on Inclined Samples and CDDS Tests	241
5.31	NESE at $(\sigma_1/\sigma_3)_{max}$ for Failure Within the Clay Layer	242
5.32	Typical Normalized Stress-Strain Curves from $\overline{CK_oU}$DSS Tests on NC and OC Connecticut Valley Varved Clays	243
5.33	Method of Computing q_{max} from $\overline{CK_oU}$DSS Test on Normally Consolidated Amherst Varved Clay (Assumed NESE at q_f from $\overline{CIUC}$ Tests on Inclined Samples)	244
5.34	Method of Computing q_f from $\overline{CK_oU}$DSS Test on Over-Consolidated (OCR=2) Northampton Varved Clay at $\tau_{h_{max}}$ (Assumed NESE at q_f from $\overline{CIUC}$ Tests on Inclined Samples)	245
5.35	Method of Computing q_f from $\overline{CK_oU}$DSS Tests on Over-Consolidated (OCR=4) East Windsor Varved Clay at $(\tau_h)_{max}$ (Assumed NESE at q_f from $\overline{CIUC}$ Tests on Inclined Samples)	246
5.36	Determinations of τ_{ff} at $(\tau_h/\bar{\sigma}_v)_{max}$ from $\overline{CK_oU}$DSS Tests	247
5.37	Failure Strains at $\tau_{h_{max}}$ and $(\tau_h/\bar{\sigma}_v)_{max}$ from $\overline{CK_oU}$DSS Tests on Connecticut Varved Clays	248
5.38	Computation of Relative Magnitudes of $q_f/\bar{\sigma}_{vc}$, $\tau_{ff}/\bar{\sigma}_{vc}$ and $\tau_{h_{max}}/\bar{\sigma}_{vc}$ by Assuming Two Locations of $\overline{NESE}$	249

Fig. No.		Page
5.39	Comparison of the Effect of Stress System on $q_f/\bar{\sigma}_{vc}$ Versus OCR for Three Different Directions of Loading	250
6.1	General Effects of Disturbance on UUC Test, RECOMPRESSION and SHANSEP samples.	274
6.2	Effect of Method of Consolidation on Vertical Undrained σ-ε- Strength Behavior from Triaxial Compression Tests on East Windsor Varved Clay OCR = 4	275
6.3	Effect of Method of Consolidation on Vertical Undrained σ-ε- Strength Behavior from Triaxial Compression Tests on East Windsor Varved Clay at OCR=2	276
6.4	Normalized Effective Stress Envelopes from $\overline{CU}$ Tests on SHANSEP and RECOMPRESSION Varved Clay Samples from Connecticut Valley (Data from Both Amherst and East Windsor for SHANSEP Samples)	277
6.5	Normalized Stress Paths from $\overline{CK_oU}$ Tests on SHANSEP and RECOMPRESSION Samples from Connecticut Valley	278
6.6	Comparison of $q_f(V)/\bar{\sigma}_{vc}$ from $\overline{CK_oUC}$ Test SHANSEP-Samples and UU Test Samples from Amherst and Northampton	279
6.7	Comparison of $q_f(V)/\bar{\sigma}_{vc}$ Versus OCR from $\overline{CK_oUC}$, $\overline{CIUC}$ and UU(V) Tests on SHANSEP and RECOMPRESSION Samples of Amherst and East Windsor Varved Clays	280
6.8	Effect of Method of Consolidation on Horizontal Undrained σ-ε- Strength Behavior from Triaxial Tests of East Windsor Varved Clay at OCR = 4	281
6.9	Effect of Method of Consolidation on Horizontal Undrained σ-ε- Strength Behavior from Triaxial Tests of Connecticut Varved Clays at OCR = 2	282
6.10	$E_u/\bar{\sigma}_{vc}$ at 1/2 $\Delta q/\Delta q_f$ from $\overline{CK_oUC}$ and $\overline{CK_oUE}$ Tests on SHANSEP and RECOMPRESSION Samples from Connecticut Valley	283
6.11	Comparison of $q_f(H)/\bar{\sigma}_{vc}$ Versus OCR from $\overline{CK_oUE}$ $\overline{CIUC}$ (θ=90°) and UU ($\theta \cong 90°$) Tests on SHANSEP and RECOMPRESSION Samples of Amherst and East Windsor Varved Clays	284

Fig. No.		Page
6.12	Normalized Stress-Strain Curves from $\overline{CIUC}$ (θ=90°) Tests on SHANSEP and RECOMPRESSION Samples (OCR= 4) from East Windsor	285
6.13	Normalized Stress Strain Curves from $\overline{CIUC}$ (θ=90°) Tests on SHANSEP and RECOMPRESSION Samples (OCR= 2) from East Windsor	286
6.14	Normalized Stress-Strain Curves from $\overline{CIUC}$ (θ=45°) Test on SHANSEP and RECOMPRESSION Samples (OCR= 4) from East Windsor	287
6.15	NESE from $\overline{CIUC}$ (θ=45°) Tests on SHANSEP and RECOMPRESSION Samples	288
6.16	Comparison of $q_f/\bar{\sigma}_{vc}$ Versus OCR from $\overline{CIUC}$ (θ=45°), $\overline{CK_oUDSS}$, and UU (45°) Test on SHANSEP and RECOMPRESSION Samples from Amherst and East Windsor	289
6.17	Comparison of $G/\bar{\sigma}_{vc}$ at γ = 1.0% from $\overline{CK_oUDSS}$ Tests on SHANSEP and RECOMPRESSION Samples	290
6.18	Comparison of $q_f/\bar{\sigma}_{vc}$ from $\overline{CIUC}$ and $\overline{CK_oUC}$ Tests on SHANSEP and RECOMPRESSION Samples	291
7.1	Effect of Sample Orientation on Stress Versus Strain from $\overline{CIUC}$ Tests on Samples of Normally Consolidated East Windsor Varved Clay	310
7.2	Effect of Sample Orientation on Stress Versus Strain from $\overline{CIUC}$ Tests on Samples of Overconsolidated (OCR=4) East Windsor Varved Clay	311
7.3	Anisotropy in $q_f/\bar{\sigma}_{vc}$ and A_f of Connecticut Valley Varved Clay Resulting from "Material" Anisotropy Component	312
7.4	Normalized Effective Stress Paths and Envelopes from Connecticut Valley Varved Clay at Different Varved Inclinations	313
7.5	"Material" Anisotropy in Pore Pressure and Pore Pressure Parameter A at ε = 1.0%	314
7.6	Anisotropy in Undrained Modulus from $\overline{CIUC}$ Tests	315
7.7	Comparison of Anisotropy in q_f Measured from Plane Strain and $\overline{CIUC}$ Tests	316
7.8	Effect of Structural Anisotropy Component on Anisotropy in q_f	317

Fig. No.		Page
7.9	Combined Anisotropy in q_f of Connecticut Valley Varved Clay Measured from $\overline{CK_oU}$ Plane Strain Tests	318
7.10	Estimated Combined Anisotropy in NESE from Plane Strain Loading	319
7.11	Anisotropy with Respect to $G/\bar{\sigma}_{vc}$ of Normally Consolidated East Windsor Varved Clay at Various Applied Shear Stress Levels	320
7.12	Anisotropy with Respect to $G/\bar{\sigma}_{vc}$ of Normally Consolidated East Windsor Varved Clay at $\gamma = 0.1\%$	321
8.1	Consideration of Anisotropy in Undrained Shear Strength in Morgenstern-Price Analyses (After Ladd, 1975)	357
8.2	Recommended $S_u/\bar{\sigma}_{vc}$ for Total Stress Analyses by Ladd (1975)	358
8.3	Comparison of Simplified Bishop Circular Arc and Morgenstern-Price Analyses (from Ladd and Foott, 1977)	359
8.4	Comparison of Normalized Undrained Shear Strength Data from UU_v, FV and Thesis's Recommended Undrained Shear Strength Parameters for Design	360
8.5	SHANSEP E_u/S_u Versus OCR for Connecticut Valley Varved Clays	361
8.6	Recommended Initial Stress Ratio Versus OCR for Connecticut Valley Varved Clays for Plane Strain Loading	362
8.7	Settlement Ratio Versus Applied Stress Ratio for Load on Isotropic Homogeneous Foundation (from D'Appolonia, Poulos and Ladd, 1971)	363
8.8	Recommended Pore Pressure Parameter A Versus OCR for Predictions of Pore Pressure in Connecticut Valley Varved Clays for Plane Strain Loading	364

Fig. No.		Page
8.9	Recommended Undrained Hyperbolic Stress-Strain Parameters for Use in Finite Element Analyses with Connecticut Valley Varved Clays	365
8.10	Recommended Vertical Undrained Strength for Use in Finite Element Analyses with Connecticut Valley Varved Clays in Plane Strain Loading	366
8.11	Recommended Elliptical Anisotropic Undrained Strength Parameters for Use in Finite Element Analyses with Connecticut Valley Varved Clays	367
8.12	Recommended Pore Pressure Parameter a Versus OCR for Use in Finite Element Analyses with Connecticut Valley Varved Clays	368
9.1	Material and Combined Anisotropy in q_f of Connecticut Valley Varved Clays at OCR = 4	377
9.2	Combined Anisotropy in q_f and in NESE at q_f at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$	378
9.3	Recommended Anisotropic Undrained Strength Parameters for Use in Total Stress Morgenstern-Price Stability Analyses with Connecticut Valley Varved Clays	379
9.4	Effect of Method of Consolidation on Vertical and Horizontal σ-ε- Strength Behavior from $\overline{CK_oUC}$ and $\overline{CK_oUE}$ Tests on Connecticut Valley Varved Clays at OCR = 2	380

1. INTRODUCTION

1.1 BACKGROUND

A varved clay is a rythmite or a rythmically layered sediment comprised of two layers and a couplet. Varved clays are generally glacial lake deposits. Each varve represents the deposition during a year, and is composed of a lower part of coarser grained material deposited in the summer and an upper finer grained part deposited in the winter. The composition of varved deposits can vary from alternating layers of highly plastic clay and clayey silt (typical of varved clays in northern US and Canada) to alternating layers of silt and medium to fine sand. The thickness of the varves also varies considerably, although 1/4 to 2 1/4 in. is most typical. There is generally a gradual transition of the soil composition within a varve, but there is an abrupt change between varves resulting from the rapid influx of sediments during the runoff season. The strength, deformation and permeability characteristics of varved clays are highly anisotropic as a result of this layering.

A three-year research program entitled, "The Behavior of Varved Clays in Civil Engineering Structures," sponsored by the Massachusetts' Department of Public Works in cooperation with the US Department of Transportation Federal Highway Administration, was conducted by the Constructed Facilities Division of the Department of Civil Engineering at MIT. The final objective

of the program was development of field and laboratory procedures that can be used with confidence in conjunction with realistic theoretical models for the design of civil engineering structures, and especially embankments, on varved clays encountered in the Connecticut Valley. In order to achieve this objective, it was necessary to:

1. Develop a better understanding of the fundamentals controlling the anisotropic engineering properties of varved clay;
2. Develop an experimental approach and theoretical model for predicting the engineering performance of varved clays;
3. Evaluate the reliability of these procedures based on case studies of the actual performance of structures on varved clays.

At the conclusion of the above research project, Ladd (1975) presented detailed recommendations for the foundation design of embankments constructed on Connecticut Valley varved clays. This report had to be prepared before research on the fundamentals controlling the undrained stress-strain-strength properties of varved clay, the topic of this thesis, was completed.

1.2 OBJECTIVES

The thesis concentrates on a study of the fundamentals of the undrained shear strength anisotropy of varved clays. The

objectives are:

1. To contribute to a fundamental understanding of the nature of undrained shear strength anisotropy of varved clays;
2. To present best estimates of normalized stress-strain strength parameters for undrained total stress analyses and for predictions of pore pressures and undrained deformations used in the design of structures on Connecticut Valley varved clays, with background information showing how these parameters were developed;
3. To recommend suitable testing procedures for developing normalized soil parameters for other varved clay deposits.

1.3 SCOPE

In order to achieve these objectives, it is necessary to study:

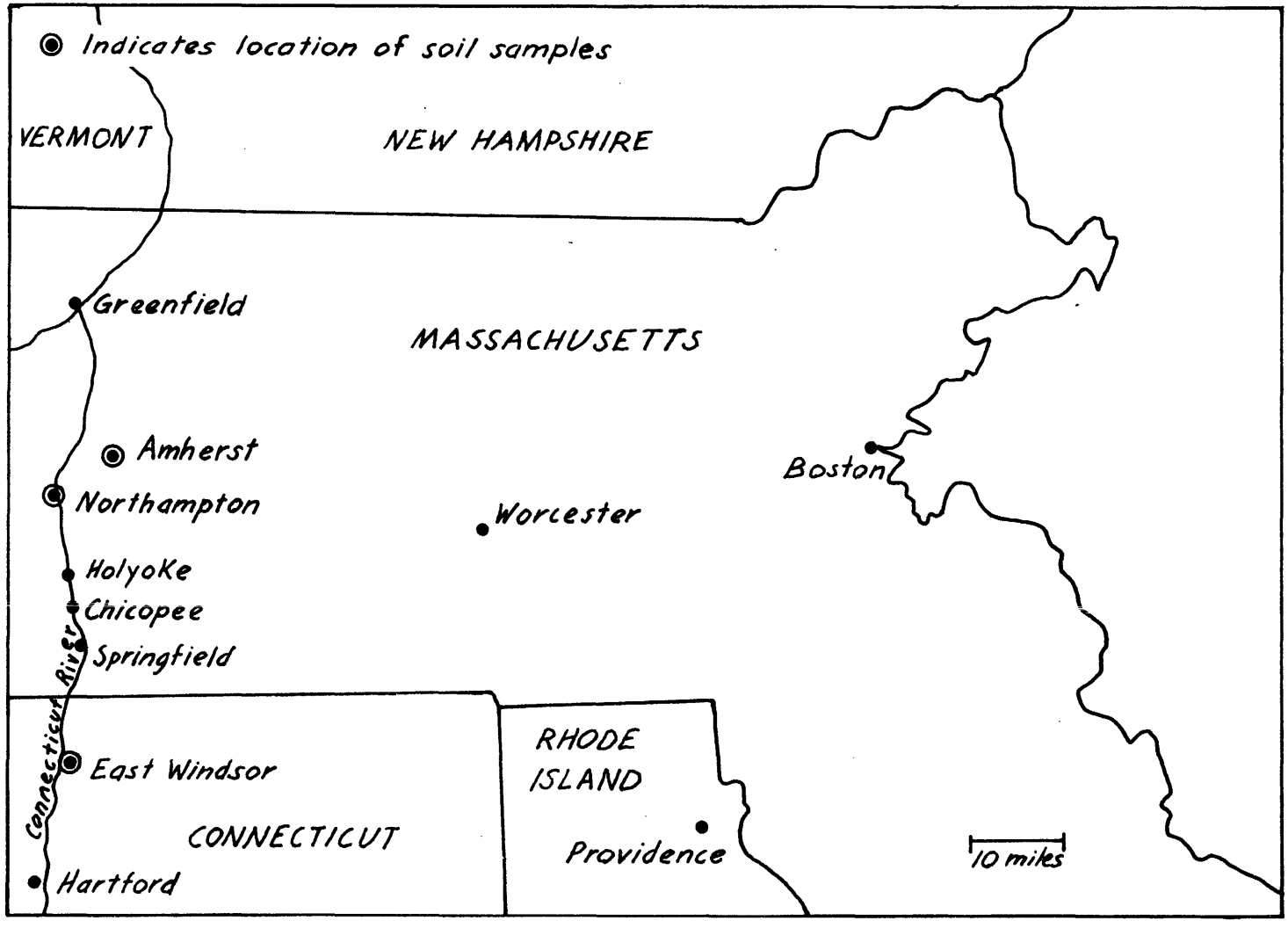
1. The normalized behavior of several varved clays using consolidated undrained shear tests;
2. The effects of stress system* and sample disturbance on the normalized stress-strain-strength parameters of varved clays;
3. Suitable laboratory methods for measuring the undrained shear behavior of varved clays;

*This denotes the applied stress conditions during consolidation and shear.

4. The fundamental factors controlling the anisotropic behavior of varved clays.

A laboratory testing program was conducted at MIT on four varved clays. These were located in the Connecticut Valley at Amherst and Northampton in Massachusetts and at East Windsor, Connecticut and in the Hackensack Valley at Secaucus, New Jersey. The geological formation of the latter soil is similar to that of the Connecticut Valley. Fig. 1.1 shows the location of the three Connecticut Valley varved clays.

The basic background and literature review of the anisotropic behavior of cohesive soils are presented in Chapter 2. Chapter 3 presents the general testing program and description of the varved clays, while Chapter 4 investigates their normalized soil properties (NSP). Chapters 5 through 7 study the fundamental factors influencing the anisotropic behavior of varved clays. The recommended NSP are presented in Chapter 8 with the necessary background showing how these parameters were developed.



LOCATIONS OF SOIL SAMPLES OF VARVED CLAYS FROM CONNECTICUT VALLEY

FIGURE 1.1

2. BACKGROUND

2.1 INTRODUCTION

The object of this chapter is to provide basic background on the subject of undrained shear strength anisotropy. Since the study of this subject yielded rather conflicting results, an attempt is made to include a fairly detailed summary of available information. The normalized soil properties (NSP) concept of Ladd and Foott (1974) will also be presented because it is used throughout the thesis. Experimental data on undrained shear strength anisotropy are also reviewed. Appendix A lists the notation used in this thesis.

2.2 BACKGROUND

2.2.1 Ansiotropy in Undrained Shear Strength

Definitions: The term anisotropy means that the properties vary with direction of the measurement. Anisotropy of undrained shear strength denotes variation in the undrained shear strength with the direction of loading. The convention that will be used to define direction is shown in Fig. 2.1. The angle β is the angle between the major principal stress at failure (σ_{1f}) and the vertical direction in the ground. Anisotropy of the undrained shear strength, therefore, implies a variation in undrained shear strength with β measured in the laboratory using tests with different directions of σ_{1f} and with controlled magnitudes of the applied intermediate principal

stress ($\Delta\sigma_2$).

Causes and Types of Anisotropy: The anisotropic undrained strength behavior of cohesive soil is intimately connected with "soil structure" (Lambe, 1958) which in turn depends on the environmental conditions during soil deposition, environmental and stress changes subsequent to deposition, and the applied stress system during shear. Stress changes subsequent to the deposition most frequently result from changes in the overburden pressure and in the ground water table elevation, including desiccation. It is convenient, according to Ladd et al. (1977), to divide undrained shear strength anisotropy, as measured in the laboratory, into three components, as follows:

(1) Inherent Anisotropy -- This component of strength anisotropy is a direct result of the in situ soil structure. Soil structure is affected by the arrangement of the soil particles, which in turn depends upon the environmental conditions during deposition and the geological stress history. The one-dimensional consolidation which prevails during the geological stress history of most sedimentary clays causes the plately-shaped soil particles to have a tendency to align themselves at right angles to the direction of the major principal stress (e.g., Martin and Ladd, 1975). As a result, when a clay is loaded in a direction such that the failure surface is horizontal or nearly horizontal, it may have the lowest shear strength (Perry, 1971; Merkle, 1971). This type of inherent anisotropy is therefore termed the "structural" anisotropy. Inherent

anisotropy also occurs if the material is non-homogeneous. The prime example of this type of anisotropy is with varved clays due to the presence of clay and silt layers, each of which has different stress-strain-strength characteristics. This will be called "material" anisotropy. In addition, each layer may also exhibit structural anisotropy.

(2) Stress System Induced Anisotropy -- This component of undrained strength anisotropy results from the fact that different increments of shear stress are required to produce failure as the major principal stress at failure, σ_{1f}, varies between the vertical and the horizontal direction, when the coefficient of earth pressure at rest (K_o) is not equal to unity. Hansen and Gibson (1949) first discussed the existence of a stress system induced anisotropy and predicted large variations in undrained shear strength with direction of loading for isotropic soils (i.e., no inherent anisotropy) with constant values of the effective strength parameters ($\bar{c}$ and $\bar{\phi}$) and of the equivalent of Skempton's (1954) pore pressure parameter, A_f. In essence, stress system induced anisotropy results in changes in effective stress as σ_{1f} varies in direction during shear.

(3) Combined Anisotropy -- "Combined" anisotropy (used in place of the term "in situ" anisotropy because anisotropic behavior is generally measured via laboratory tests) includes the total effects of inherent and stress system induced anisotropy. K_o consolidated undrained ($\overline{CK_oU}$) plane strain tests*

*i.e., plane strain compression (PSC) and plane strain extension (PSE).

run on vertical samples should measure the combined effect of inherent and stress system induced aniostropy for plane strain loading conditions in the vertical and horizontal direction because the anisotropic fabric is oriented in the same direction as in situ, and because shear starts from a K_o consolidation. However, with $\overline{CK_oU}$ triaxial tests, the directional variations in strength shown by comparing data from triaxial compression (TC) tests ($\beta = 0$) and triaxial extension (TE) tests ($\beta = 90°$) result both from the total effect of anisotropy (inherent and stress system induced) and the differences in the magnitudes of the intermediate principal stress ($\Delta\sigma_2$). [$\Delta\sigma_1 = \Delta\sigma_2 > \Delta\sigma_3$ in TE tests while $\Delta\sigma_1 > \Delta\sigma_2 = \Delta\sigma_3$ in TC tests.]

Section 2.2.2 will discuss the problems associated with various types of laboratory strength tests for predicting in situ shear strength anisotropy.

2.2.2 Measurement of Undrained Shear Strength Anisotropy

Figure 2.2(a) shows states of stress for three typical soil elements along a potential sliding surface beneath an embankment. For simplicity, straight lines are used to represent the failure surface. The stress components shown reflect both the initial in situ stress on the failure surface (i.e., $\bar{\sigma}_{\alpha o}$, $\tau_{\alpha o}$) and the incremental (decremental) effects ($\Delta\tau_\alpha$) due to subsequent undrained shearing caused by construction of the embankment. At Point A, the major principal stress at failure is in the vertical direction (i.e., $\beta = 0°$) while it acts in the horizontal direction at Point C ($\beta = 90°$). At an intermediate

location such as Point B, the major principal stress at failure is inclined. Thus, unless K_o is unity, the principal stresses are rotated during shear at Points B and either A or C.

Types of laboratory consolidated-undrained shear tests that might be used to measure the undrained shear strength of Points A, B, and C are shown in Fig. 2.2(b). For Point A, a vertical sample can be sheared in triaxial or plane strain compression (TC or PSC), after K_o consolidation for Method A_1. An inclined sample can also be tested in a direct simple shear (DSS) test for Method A_2 (Bjerrum, 1973). At Point B, K_o consolidated undrained direct simple shear ($\overline{CK_oU}$DSS) tests run on vertical samples, or isotropically consolidated undrained triaxial compression ($\overline{CIU}$C) tests performed on inclined samples might be used, Method B_1 and B_2, respectively. K_o consolidated undrained ($\overline{CK_oU}$) triaxial and plane strain extension (TE and PE) tests on vertical samples and isotropically consolidated undrained ($\overline{CIU}$) triaxial compression tests on horizontal* samples could be used to simulate the state of stresses at Point C for Methods C_1 and C_2. A $\overline{CK_oU}$DSS test on an inclined sample, varved inclination $\theta = 45°$, which is sheared by reversing the direction of the applied shear stress can be used to measure the strength at Point C for Method C_3. Figure 2.2(b) shows the states of stress acting on the soil samples during consolidation and subsequent shear for each method. It should be noted, however,

*Horizontal sample means that the axis of the sample is cut from the in situ horizontal direction (i.e., $\theta° = 90$ for varved clay; and the specimen varves are vertical).

that only the stress components that can be measured are shown for methods involving the direct simple shear test.

Both conventional and sophisticated methods of investigating undrained strength anisotropy are summarized in Table 2.1. The problems associated with these approaches are also outlined. Many investigators (e.g., Bjerrum and Landva, 1966; Beere, 1969,; Bjerrum, 1973; and Ladd et al.; 1977) are convinced that undrained shear strength and its anisotropy can only be reliably measured in the laboratory provided: the samples are consolidated anisotropically and sheared with the same stress system as in the field. These researchers have developed their "standard" procedures for measuring "anisotropy." They include K_o consolidated triaxial compression and extension tests (or less frequently plane strain compression and extension tests) for vertical and horizontal loadings respectively (Points A and C) and a direct simple shear test for inclined loadings (Point B). Since the $\overline{CK_oU}$DSS test can cause a rotation of principal planes during shear, Bjerrum (1973) also suggested Methods A_2 and C_3 in Fig. 2.2, i.e., Option 5 in Table 2.1.

Table 2.1 also lists problems associated with each option. Unconsolidated undrained triaxial compression (UUC) tests, Option 1, can at best yield only crude estimates of anisotropy because of sample disturbance. Furthermore, shear starts from an isotropic state of stress, and therefore neglects the stress system induced component. With $\overline{CIUC}$ tests, wherein samples

are usually consolidated to the effective vertical overburden stress ($\bar{\sigma}_{vo}$) or less, Option 2, the effect of sample disturbance may be somewhat reduced, but isotropic consolidation will undoubtedly cause some change in the initial structure anisotropy. Also, as with Option 1, there is no stress system induced anisotropy during the actual shearing process.

Option 3 uses K_o consolidated triaxial compression and extension tests and the direct simple shear test. This involves three different values of the intermediate principal stress ($\sigma_2 = \sigma_3$ in triaxial compression, $\sigma_1 > \sigma_2 > \sigma_3$ in direct simple shear, and $\sigma_1 = \sigma_2$ in triaxial extension), and thus, can not be directly used to obtain the in situ directional variation of undrained shear strength unless variations in the intermediate principal stress have little effect on the undrained shear strength (or the data are corrected for estimated effect of this factor). The DSS test also has problems with interpretation of the results, since the state of stress at failure is unknown. Consequently, the effective shear strength parameters, $\bar{c}$ and $\bar{\phi}$, and Skempton's pore pressure parameter, A_f, can not be directly measured (Ladd and Edgers, 1972). Furthermore, it is not certain what the measured maximum horizontal shear stress (τ_{hmax}) represents. It is generally less than the maximum shear stress (q_f) and may be, even less than the shear stress on the failure plane at failure, τ_{ff} (see Section 5.4). Because of the presence of the cylindrical wire reinforced rubber membrane, the non-uniform stresses and possibly

progressive action (especially at the edges of the sample) exist in the DSS test.

In Option 4, use of plane strain equipment often presents problems due to friction forces that are generated along the loading piston and along the vertical boundaries of the samples. Unless these friction forces are measured, one is never sure of the reliability of the data. In Option 5, the direct simple shear test has the same problems associated with interpretation of test results, as discussed above. In addition, there are two problems with tests simulating Points A and C. The first is that K_o must be known or estimated in order to apply the correct value of $\bar{\sigma}_{\alpha o}$ and $\tau_{\alpha o}$ (see Fig. 2.2(b)). The second is that the consolidation stresses perpendicular to this plane can not be controlled.

Thus, based on the above discussion, all options involve problems of a different nature and magnitude. Option 4 is theoretically the best if reliable plane strain equipment is available; if not, Option 3 or 5 offer the most attractive alternatives.

2.2.3 The Normalized Soil Properties (NSP) Concept

The normalized soil properties (NSP) concept is used to study the undrained shear strength behavior of varved clays throughout this thesis. In this section, the basic ideas of the NSP concept are presented. For further information on this subject, the reader is referred to Ladd (1971), Ladd and Foot

(1974) and Ladd et al. (1977).

The NSP concept was developed from the following empirical observation that applies to many clays: the results of consolidated undrained (CU) laboratory shear tests performed on samples with the same overconsolidation ratio (OCR), but different consolidation stresses and therefore, different maximum past pressures ($\bar{\sigma}_{vm}$), exhibit very similar strength and stress-strain characteristics when normalized with respect to the consolidation stress. Thus, it is possible to run laboratory tests at various OCR values and develop an unique normalized plot of soil parameters versus OCR. Examples of normalized soil parameters that can be used in such plots are the normalized maximum shear stress at failure ($q_f/\bar{\sigma}_{vc}$), the normalized undrained Young's modulus ($E_u/\bar{\sigma}_{vc}$), Skempton's pore pressure parameter (A_f), and the coefficient of earth pressure at rest (K_o). With a knowledge of the in situ stress history, pertinent soil properties can then be computed.

Measurement of NSP

Laboratory measurement of engineering properties always involves effects of sample disturbance. In cohesive soils, sample disturbance reduces the undrained shear strength (q_f) as measured in UUC tests (e.g., Ladd and Lambe, 1963; Davis and Poulos, 1967). But observations of compression curves from consolidation tests (Schmertmann, 1955) indicate that "disturbed" samples have approximately the same virgin compression characteristics as those of undisturbed samples provided that the

consolidation stress is beyond the maximum past pressure (and the degree of disturbance is not excessive). Thus, the effect of sample disturbance can be reduced if the soil sample is consolidated beyond the maximum past pressure into the virgin compression range. Based on this observation, Ladd and Foott (1974) suggest that for soils exhibiting normalized behavior, undrained strength parameters should be obtained from samples which are consolidated according to the SHANSEP approach. This involves consolidation of soil samples to stresses in excess of the in situ maximum past pressure in order to minimize sample disturbance effects, and subsequently reducing the effective stress to obtain samples at the required OCR values. Thus, this method not only reduces the effect of sample disturbance, but it also provides a sample of known OCR. For obtaining the appropriate SHANSEP NSP, the samples must be K_o consolidated to at least 1.5 to 2 times the in situ $\bar{\sigma}_{vm}$ and sheared with stress systems similar to the in situ conditions. Also, use of SHANSEP NSP (obtained from the above testing procedure) for estimating the in situ soil properties (q_f, E_u etc.) requires a knowledge of the in situ stress history ($\bar{\sigma}_{vo}$ and $\bar{\sigma}_{vm}$), since NSP are related to OCR.

Limitation of SHANSEP Method of Consolidation

With highly structured soils (e.g., "quick" clays and naturally cemented soils), consolidation above the in situ stresses will cause radical changes in soil behavior. The SHANSEP method of consolidation, therefore, does not apply to

such soil. With these materials, the in situ stresses should be used for consolidation (Berre and Bjerrum, 1973) although this requires samples of exceptionally high quality if meaningful data are to be obtained. Recent work by Sangrey (1975) illustrated that the NSP concept could be used in cemented soils provided the soil parameters were normalized with respect to yield stress, defined as the maximum stress to which the samples can be consolidated without changing its basic behavior.

In theory, if a soil truely exhibits normalized behavior, NSP from the SHANSEP method of consolidation should represent the in situ soil properties. In reality, almost all natural clays will undergo some change in structure when they are consolidated above the in situ $\bar{\sigma}_{vm}$. As a result, NSP from the SHANSEP samples, especially those of overconsolidated (OC) soils, may be different from those of the in situ soil. However, this does not imply that the SHANSEP method of consolidation necessarily yields unsatisfactory NSP, especially when compared to those obtained from other methods (e.g., UU tests or CU with the RECOMPRESSION* Method). Since sample disturbance also causes changes in soil structure, data obtained by the other methods also yield soil properties which are different from those existing in situ. In fact, for most NC soils, SHANSEP samples ($\bar{\sigma}_{vc} \geq$ (1.5 to 2) $\bar{\sigma}_{vm}$) should have a structure

*RECOMPRESSION: the method of consolidation whereby the sample is consolidated at the desired OCR with a vertical consolidation stress is less than the in situ maximum past pressure.

similar to in situ normally consolidated clay and hence yield reasonable NSP (Ladd et al. 1977).

Chapter 4 will present the experimental evidence showing that for the stress range used in the study (which is rather small), the varved clays under investigation do indeed exhibit normalized behavior, at least with regard to certain properties. Chapter 6 then compares properties obtained via the SHANSEP method with those measured using other procedures.

2.3 LITERATURE REVIEW

Two areas of research for the study of the undrained shear strength anisotropy are considered. The first, the analytical approach, concentrates on the analytical presentation of undrained shear strength anisotropy and includes (1) curve fitting methods to describe anisotropy using empirical equations and semi-theoretical relations; and (2) theoretical predictions wherein, after hypothesizing possible causes of shear strength anisotropy, "theoretical" anisotropic shear strength relations are derived. The second approach is experimental, in which possible causes and the factors influencing undrained shear strength anisotropy are investigated.

Section 2.3.1 will review the analytical approach, including both curve fitting methods (Section 2.3.1(a)) and theoretical predictions (Section 2.3.1(b)). Possible factors affecting the aniostropic shear strength behavior of varved clay, using isotropic Mohr-Coulomb failure theory, are presented in Section

2.3.1(c). Section 2.3.2 summarizes the experimental results from: UUC and $\overline{CIUC}$ tests on samples with different inclinations; $\overline{CK_oU}$ triaxial, direct simple shear (DSS), and plane strain tests on vertical samples; and special $\overline{CK_oU}$DSS tests on samples with different inclinations. Since the effect of stress system on NSP is studied in Chapter 5, a brief review of the influence of the intermediate principal stress and the K_o consolidation on the strength behavior is presented in Section 2.3.3.

2.3.1 Analytical Approach

2.3.1(a) Curve Fitting Method

The first curve fitting method, proposed by Casagrande and Carillo (1944), for presenting anisotropic shear strength behavior is the equation:

$$S_u(\beta) = S_u(H) + \left(S_u(V) - S_u(H)\right)\cos^2\beta \qquad \text{(Eq. 2.1)}$$

Their relationship was suggested originally as a working hypothesis without experimental justification. Davis and Christian (1971) later showed that experimental data from UUC tests on several relatively homogeneous soft clays (e.g., Welland clay, San Francisco Bay mud, and Kaolinite) fitted this relationship reasonably well. Bishop (1966) proposed a modification of Casagrande-Carillo equation:

$$S_u(\beta) = S_u(V)\ (1 - m \sin^2\beta)\ (1 - n \sin^2 2\beta) \qquad \text{(Eq. 2.2)}$$

to fit UUC test data on London clay in which the parameters $S_u(V)$, m, and n are determined from laboratory tests (Bishop suggested using $S_u(V)$, $S_u(45)$ and $S_u(H)$ as a minimum in establishing these parameters).

Davis and Christian (1971) proposed an elliptical anisotropic strength model to use in bearing capacity analyses. They modified the anisotropic yield criteria for metals (Hill, 1950 and Scott, 1963) to apply to cohesive soil as follows:

$$\frac{(\sigma_y - \sigma_x)}{2} - \frac{S_u(V)-S_u(H)}{2}^2 + \tau^2_{xy} \frac{a^2}{b^2} = a^2 \qquad \text{(Eq. 2.3)}$$

$S_u(V)$ and $S_u(H)$ = the apparent undrained shear strength in compression in the vertical and horizontal directions.

a and b = constants to be found experimentally.

$$\frac{S_u(V)+S_u(H)}{2} = a, \text{ and } b/a = \frac{S_u(45)}{\sqrt{S_u(V)S_u(H)}}$$

In the polar plot such as shown in Fig. 2.3, the variation in undrained shear strength with angle 2β is elliptical. This equation was found to fit most existing UUC test data (Davis and Christian, 1971).

2.3.1(b) Theoretical Prediction of Anisotropic Behavior

Hanson and Gibson (1949) recognized that K_o consolidation could cause anisotropy with respect to undrained shear strength even in soil having isotropic Hvorslev (1960) effective shear strength parameters and Skempton's (1948) λ parameter (the ratio of the slope of a swelling curve to the slope of the virgin consolidation curve). They first predicted the existence of stress system induced anisotropy (Ladd et al.

1977). Their expression for $S_u(=q_f)$ for any orientation of the failure plane was in terms of the initial shear stress , the Hvorslev shear strength parameters, the angle between the failure plane and the horizontal plane (α), and the change in pore pressure due to changes in total stress based on Skempton's (1948) "λ theory." Duncan and Seed (1966(a)) reviewed their work and modified the Hanson and Gibson equation to use isotropic Mohr-Coulomb effective stress strength parameters $\bar{\phi}$ and $\bar{c}$ and Skempton's pore pressure parameter A_f. The equation in terms of $\bar{c}$, $\bar{\phi}$ and A_f for plane strain condition is

$$\frac{S_u(\alpha)}{\bar{\sigma}_{vc}} = \frac{\bar{c}}{\bar{\sigma}_{vc}} \cos\bar{\phi} + 1/2\ (1+K_o)\sin\bar{\phi} - \sin\bar{\phi}(2A_f-1)$$
$$\left[\left(\frac{S_u(\alpha)}{\bar{\sigma}_{vc}}\right)^2 - \frac{S_u(\alpha)}{\bar{\sigma}_{vc}}\ (1-K_o)\cos 2(45 + \bar{\phi}/2 - \alpha)\right.$$
$$\left. + \left(\frac{1-K_o}{2}\right)^2\right]^{1/2} \qquad \text{(Eq. 2.4)}$$

Although the theory is simplistic in that A_f, $\bar{c}$ and $\bar{\phi}$ are assumed constant, and therefore independent of the direction of loading, Eq. 2.4 predicts trends (shown in Table 2.2) which are amazingly realistic compared to test data on homogeneous clays reported in Tables 2.5 and 2.6.

Baker and Krizek (1970) and Markle (1971) considered directional variations in $\bar{\phi}$ and $\bar{c}$ with failure plane inclinations (α). They independently developed equations for calculating the shear strength (q_f) and the failure plane inclination in a

cross-anisotropic soil, given the values of β and $\bar{p}_f$ (Markle, 1971), or β and $\bar{\sigma}_{3f}$ (Baker and Krizek, 1970) provided the variations of $\bar{\phi}$ and $\bar{c}$ with α are known. The governing equations (Eqs. 2.5 to 2.7) from Markle's method (1971) are presented in Figs. 2.4(a) and 2.4(b). Based on the modified Mohr-Coulomb (M-C) failure criteria (i.e., $\bar{\phi}$ and $\bar{c}$ vary with failure plane inclinations (α)), for a given value of β, failure in the soil occurs when the point representing the state of stress at failure (τ_{ff} and $\bar{\sigma}_{ff}$) on the failure plane at orientation δ lies on the corresponding failure envelope (only one envelope exists for given values of β and $\bar{p}_f$). As a result, the failure envelope can in the general case, intersect or be tangent to the Mohr circle. It is seen that the resulting equations are difficult to use (since the variations of $\bar{\phi}$ and $\bar{c}$ with inclinations of failure plane (α) must be known) and they require knowledge of $\bar{p}_f$ for computing q_f and the failure plane inclination (α). As a result, they have little practical value for predicting the variation in q_f with β, though the basic causes of anisotropy in q_f are hypothesized.

2.3.1(c) Factors Affecting the Anisotropic Shear Strength Behavior of Varved Clay

The variables that could affect $\bar{\phi}$ and $\bar{c}$ of varved clays were studied by Townsend et al. (1965). They presented the following general expression indicating the possible variables in the case where failure planes cut across the varves.

$$\tan \bar{\phi} = (1-n_c) \tan \bar{\phi}_s + n_c \tan \bar{\phi}_c \qquad \text{(Eq. 2.8(a))}$$

and $\bar{c} = (1 - n_c)\ \bar{c}_s + n_c\ \bar{c}_s$ (Eq. 2.8(b))

where $\bar{\phi}$ and $\bar{c}$ = M-C effective stress strength parameters of bulk material

$\bar{\phi}_s$ and c_s = M-C effective stress strength parameters of silt portion

$\bar{\phi}_c$ and $\bar{c}_c$ = M-C effective stress strength parameters of clay portion

n_c = % of clay layer in the failure plane

The following are the assumptions used for developing Eq. 2.8:

1. The failure surface is a plane that intersects clay and silt layers at the same inclination.
2. Both silt and clay layer exhibit isotropy with respect to the effective stress strength parameters $\bar{\phi}$ and $\bar{c}$ (i.e., $\bar{\phi}_c$, $\bar{\phi}_s$, $\bar{c}_c$ and $\bar{c}_s$ are independent of failure plane inclinations).
3. The same $\bar{\sigma}_{ff}$ acts in both clay and silt layers.

The above equation suggests that in the case where n_c is constant, values of $\bar{\phi}$ and $\bar{c}$ of varved clays will be constant. Thus, for homogeneous varved clay, there are three possible values of $\bar{\phi}$ and $\bar{c}$. These are: (1) for failure within the clay layer (i.e., $n_c = 1.0$); (2) for failure within the silt layer (i.e., $n_c = 0$); and (3) for failure across the varves.

Data in Chapter 5 will show, provided samples have the same failure mechanism (i.e., failure across the varves or failure within the clay layer), that values of $\bar{\phi}$ and $\bar{c}$ at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ of OC samples are constant and independent of failure

plane inclination. But for other conditions, i.e., $\bar{\phi}$ ($\bar{c}$=0) at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ of NC soil or $\bar{\phi}$ and $\bar{c}$ at q_f (when they are not equal to those at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$), it is not true.

2.3.2 Experimental Approach

The purpose of the literature review in this section is to synthesize experimental evidence from several cohesive soils regarding: (1) the possible causes of anisotropy in S_u, and the factors influencing anisotropic behavior measured in laboratory tests. The presentation of results will be organized according to the types of laboratory tests, namely: UUC* and $\overline{CIUC}$ tests on samples with different inclinations; and $\overline{CK_oU}$ tests such as those described in Table 2.1. Experimental results from homogeneous clays and from varved clays are included.

2.3.2.(a) Evidence from UUC Tests

Table 2.3 summarizes the data from UUC tests on differently inclined homogeneous samples (presumably measuring the inherent anisotropy) of six clays. The results illustrate the possible trends that may be encountered from UUC tests on homogeneous samples, as follow:

(1) The isotropic trend: Jacobson (1955) reported results of 34 unconfined compression tests performed at various inclinations on samples of post-glacial marine clay from Stockholm. As shown in Table 2.3, the variation in q_f is within the limit

*The unconfined compression test is treated as a UUC test where σ_c is equal to zero.

of the mean error. This apparent isotropic trend could be the result of sample disturbance which tends to mask undrained strength anisotropy of homogeneous clay (Ladd et al., 1977).

(2) The anisotropic trend. As shown in Table 2.3, with the exception of test results from Kaolin (Mitchell, 1967; and Duncan and Seed (1966(b)) and from Little Belt clay (Hvorslev, 1960), the horizontal strength is the weakest. Therefore, these data suggest, for homogeneous clays, as shown for UUC test data, that the lowest strength will be either in the inclined or horizontal loading direction.

(ii) Non-Homogeneous Clay

Non-homogeneous clays include stratified soils as well as varved clays. The London clay which has laminations and fissures is also included in this category. Table 2.4 and Fig. 2.5 summarizes data on non-homogeneous clays from several locations. These data show that samples inclined with β equal to 45 to 60 degrees had the lowest strength. The UUC results for Connecticut Valley varved clays, summarized in Fig. 2.5, are of particular interest. The main points worthy for mention are:

1. The q_f (45)/q_f(V) ratio ranges from 0.3 to 0.5
2. The anisotropic effect is more pronounced in samples at shallow depth ($W_N \tilde{\gtrsim} 50\%$) compared to those at greater depth.
3. $\frac{q_f(H)}{q_f(V)}$ ratio is greater than one for varved clay located near the surface, and less than one for sample

located at greater depth. This difference suggests that the stress history influences anisotropic behavior, i.e., the sample at shallow depth probably has a higher OCR.

Sample disturbance in non-homogeneous clay may amplify the inherent anisotropic effect as measured with UUC tests. Ward et al. (1965) concluded that opening of the laminations in London clay (caused by disturbance during sampling) was likely to have little effect on shear strength unless the failure surface was nearly horizontal (generally observed in inclined loading) in which case the shear strength could be appreciably reduced. Disturbance due to separation of clay and silt layers in varved clay during sampling and trimming may also cause a significant reduction in $q_f(45°)$ while $q_f(V)$ is slightly affected (see Chapter 6 and Section 2.3.2(b)).

2.3.2(b) Evidence from $\overline{CIUC}$ Tests

The anisotropic behavior from $\overline{CIUC}$ tests on samples with different inclinations was measured for both undisturbed samples (Bishop et al., 1965; Lo, 1966, and Perry and Nadarajah, 1974) and laboratory prepared Kaolin samples (Duncan and Seed (1966(a)) and Mitchell (1972). Figures 2.6 to 2.10 summarize their results. Test data from Connecticut Valley varved clays (Long, 1975) are presented in Fig. 2.11. For the laboratory prepared Kaolin samples (Duncan and Seed, 1966(a); and Mitchell, 1972), the test specimens were anisotropically consolidated and then extruded so that the effective stress

immediately after sampling was due to a negative pore pressure and was therefore isotropic. The specimens were then isotropically consolidated to stresses less than the $\bar{\sigma}_{vc}$ of the block sample before they were sheared, with the exception of two samples (vertical and horizontal) reported by Mitchell (1972) where $\bar{\sigma}_c$ was greater than $\bar{\sigma}_{vc}$ of the block sample. The undisturbed soils are the OC Welland clay from Ontario (L0, 1966); the heavily overconsolidated London clay at Ashford Common Shaft (Bishop et al., 1965); an overconsolidated soft clayey silt from Fulford, England; and the Connecticut Valley varved clays. The available index properties and the UUC test data (only from Welland Clay (Lo, 1965) and Connecticut Valley varved clay Long (1975)) are also presented.

These $\overline{CIUC}$ data, mostly for OC vertical and horizontal samples, show the following:

1. Data from London clay (Fig. 2.6) and laboratory prepared Kaolin (Figs. 2.9 and 2.10) show that the failure envelope in terms of effective stress (i.e., the effective stress strength parameters $\bar{\phi}$ and $\bar{c}$) was essentially the same for vertical and horizontal samples. The differences in q_f are therefore the result of the differences in A_f and hence pore pressure developed during shear. Data from the soft clayey silt from Fulford (Fig. 2.8) show only slight differences in failure envelopes from vertical,

inclined, and horizontal samples (practically insignificant considering the difficulty in determining $\bar{\phi}$ and $\bar{c}$ from the scatter of laboratory data). For the East Windsor varved clay, the anisotropy in q_f is due to both the anisotropy in effective stress strength parameters and the pore pressure (Fig. 2.11).

2. The data from UUC and $\overline{CIUC}$ tests on Welland clay (Fig. 2.7) and the East Windsor varved clay (Fig. 2.11) show different trends with respect to anisotropy in q_f. It is believed, however, that sample disturbance affected the UUC test results so as to decrease the anisotropy in q_f for the Welland clay and to increase it for the East Windsor varved clay (see Chapter 6).
3. Based on test data from Welland clay (Fig. 2.7) and laboratory prepared Kaolin (Fig. 2.10), the excess pore pressure versus axial strain relation is unique for samples tested with various orientations, though Δu_f of these samples is different (since ε_f is different).
4. For both homogeneous and non-homogeneous soils (Figs. 2.6 to 2.11), the vertical strength ($q_f(V)$) is larger than the horizontal strength ($q_f(H)$).

5. An increase in the magnitude of isotropic consolidation stress ($\bar{\sigma}_c$) in $\overline{CIUC}$ tests decreases the anisotropic shear strength behavior. Data from Figs. 2.6 and 2.8 to 2.11 indicate that $\frac{q_f(H)}{q_f(V)}$ and $\frac{A_f(H)}{A_f(V)}$ approach unity as $\bar{\sigma}_c$ reaches $\bar{\sigma}_{vm}$ for undisturbed soil or approach or exceeds $\bar{\sigma}_{vc}$ of the block for laboratory prepared Kaolin samples. Test data from prior K_o consolidated laboratory prepared Kaolin (Mitchell, 1972; in Fig. 2.10) in fact, showed that when $\bar{\sigma}_c$ exceeds $\bar{\sigma}_{vc}$ of the block, the Kaolin behaved like an isotropic material. The stress-strain-pore pressure (σ-ε-u) behavior and thus the stress paths (plots of q versus $\frac{\bar{\sigma}_1 + 2\bar{\sigma}_3}{3}$), of horizontal and vertical samples (shown as the dashed line in Fig. 2.9) are essentially identical. Therefore, the data in Figs. 2.6 and 2.8 to 2.11 (especially those in Fig. 2.9) suggest that the structural anisotropy component resulting from prior K_o consolidation which causes anisotropic behavior is reduced (or may even be completely eliminated if $\bar{\sigma}_c$ exceeds $\bar{\sigma}_{vm}$) by high isotropic consolidation stresses.

2.3.2(c) Evidence from $\overline{CK_oU}$ Tests

The total effect of the combined anisotropy (that resulting from the inherent and the stress system induced components) and the difference in the applied magnitudes of σ_2 (only when comparing data from TC and TE tests) on variations of S_u with β, is measured with $\overline{CK_oU}$ tests. This section will present the NC and OC anisotropic shear strength behavior data obtained from Options 3, 4, and 5 of Table 2.1. The variations of S_u with loading direction, of several soft clays from these options are presented in Tables 2.5, 2.6, and 2.7 and Fig. 2.12. The data in Table 2.6 and Fig. 2.12 were obtained from the RECOMPRESSION ($\bar{\sigma}_{vc} = \bar{\sigma}_{vo}$) method of consolidation used by NGI. The SHANSEP method of consolidation ($\bar{\sigma}_{vc} \geq$ (1.5 to 2.0) $\bar{\sigma}_{vm}$) was used to obtain test data in Tables 2.5 and 2.7 for $\overline{CK_oU}$ plane strain and traixial test results. The strength ratios $\frac{S_u(DSS)^*}{S_u(V)}$ and $\frac{q_f(H)}{q_f(V)}$ from Options 3 and 4 (excluding Option 5 because of problems in the interpretation of test data) are presented in Figs. 2.13(a) and 2.13(b). The data from Tables 2.5, 2.6, and 2.7 and Figs. 2.13(a) and (b) show that:

1. The anisotropy in S_u is more pronounced in the lean sensitive clay (having lower PI) than in the plastic insensitive clays.
2. For both plane strain and triaxial loading ,

*See the definition in Tables 2.5 and 2.6.

$S_u(H)$ of homogeneous clays is lowest, partly because of the effect of the rotation of principal planes and, for the triaxial tests, also due to the difference in the magnitude of σ_2.

3. $\frac{q_f(H)}{q_f(V)}$ ratio is affected by the type of test (i.e., plane strain versus triaxial) and (perhaps) the method of consolidation. Compared at same PI (Fig. 2.13(b)), SHANSEP $\overline{CK_oU}$ plane strain (Option 4) tests yield higher $\frac{q_f(H)}{q_f(V)}$ ratios than those from RECOMPRESSION ($\bar{\sigma}_{vc}=\bar{\sigma}_{vo}$) $\overline{CK_oU}$ triaxial (Option 3) tests, and to a lesser extent, the SHANSEP $\overline{CK_oU}$ triaxial samples.

4. $\frac{S_u(DSS)}{S_u(V)}$ ratio is apparently affected little by the type of test and the method of consolidation.

The test data from $\overline{CK_oU}$DSS tests (Option 5, Fig. 2.12) give the variation of q_f with sample inclination rather than the loading direction. It is seen that the maximum S_u from $\overline{CK_oU}$DSS (C) tests occurs between sample inclinations of 40 to 45 degrees, and the minimum S_u from $\overline{CK_oU}$DSS (E) tests occurs between sample inclinations of 50 to 60 degrees. Thus, the use of sample inclinations of 45^o (see Chapter 5) should yield S_u values close to $S_{u\,max}$ from $\overline{CK_oU}$DSS (C) tests and somewhat larger than $S_{u\,min}$ from $\overline{CK_oU}$DSS (E) tests.

Table 2.7 also presents a summary of effective stress strength data from $\overline{CK_oUPSC}$, $\overline{CK_oUPSE}$, $\overline{CK_oUC}$ and $\overline{CK_oUE}$ tests on SHANSEP normally consolidated samples of several soft clays. It is seen that the direction of loading can cause differences in both $\bar{\phi}$ and A_f. It should be noted that, at the same $\bar{\sigma}_{vc}$, a smaller A_f in $\overline{CK_oUPSE}$ tests compared to in $\overline{CK_oUPSC}$ tests, such as occurred with the San Francisco Bay Mud and AGS CH clay, does not mean that the extension tests had smaller pore pressure. Because of the larger magnitude of Δq required to cause failure, $\bar{p}_f/\bar{\sigma}_{vc}$ value from the $\overline{CK_oUPSE}$ tests (when $K_o < 1$) can be lower than that from $\overline{CK_oUPSC}$ tests (i.e., the effect of stress system induced anisotropy). To illustrate this point, $\bar{p}_f/\bar{\sigma}_{vc}$ values from San Francisco Bay Mud (Duncan and Seed, 1966(b)) are given below:

Type of Test	A_f	$q_f/\bar{\sigma}_{vc}$	$\bar{p}_f/\bar{\sigma}_{vc}$	$\bar{\phi}^o$
$\overline{CK_oUPSC}$	1.12	0.12	0.60	38^o
$\overline{CK_oUPSE}$	0.70	0.57	0.49	35^o

Based on test data in Table 2.7, the differences in A_f, γ_f* and $\bar{\phi}$ from vertically and horizontally loaded samples are more pronounced in lean sensitive clays than in plastic insensitive clays.

The stress history can also affect the combined

*Defined in Table 2.7, based on linear isotropic elastic theory.

anisotropic behavior. Table 2.8 shows the effect of OCR on the q_f, A_f and ε_f ratios from plane strain compression and extension tests on Boston Blue Clay. It is seen that the anisotropy in q_f and ε_f decrease, but A_f increases with increasing OCR. The fact that the effect of the rotation of principal planes is more pronounced in NC samples than in OC samples (since K_o increases with increasing OCR) partly explains such trends.

2.3.3. Summary and Conclusions

Based on the review test data from UUC, $\overline{CIU}$ and $\overline{CK_oU}$ tests, the possible factors affecting the anisotropy in S_u measured in the laboratory are:

1. Sample disturbance: For homogeneous clay, sample disturbance tends to mask the anisotropic effect measured with the UUC tests. However, for non-homogeneous clay, sample disturbance may increase the anisotropic effect.
2. The Magnitude of $\bar{\sigma}_c$: The use of high $\bar{\sigma}_c$, especially using $\bar{\sigma}_c > \bar{\sigma}_{vm}$ in $\overline{CIUC}$ tests on samples with different inclinations tends to minimize, if not completely eliminate, the structural anisotropy resulting from prior in situ K_o consolidation. As a result, unless K_o is equal to 1, it is not possible

to truly separate the effect of stress system induced anisotropy from the combined anisotropy by comparing test data from $\overline{CIUC}$ tests ($\bar{\sigma}_{vc} = \bar{\sigma}_{vo}$) on samples with different inclinations (presumably measuring the inherent anisotropy), and from $\overline{CK_oU}$ tests on vertical samples (measuring combined anisotropy).

Anisotropy is found to be more pronounced in the lean sensitive soils than in highly plastic clays. Based on limited data, the anisotropic behavior of cohesive soil is also dependent upon the in situ stress history.

The magnitude of the σ_2 used in the test can affect the $\frac{q_f(H)}{q_f(V)}$ ratio. Test data from $\overline{CK_oU}$ plane strain and triaxial tests show that the $\frac{q_f(H)}{q_f(V)}$ ratios from plane strain tests are somewhat larger than those from triaxial tests. Such differences are thought to be due to the differences in the magnitudes of σ_2 between triaxial and plane strain loading.

Data from both plane strain and triaxial tests on NC sample show that the directional variation of $S_u(q_f)$ with β is due to the directional variations of both the effective stress strength parameter $\bar{\phi}$ at q_f and the pore pressure. The directional variation in pore pressure is in turn the result of the directional variation of A_f, and the difference in the magnitudes of Δq_f. The larger magnitude of Δq_f in a horizontally loaded sample, compared to

that in the vertically loaded sample, can cause larger pore pressures and a lower effective stress at failure, even though the A_f for horizontally loaded sample is lower.

For OC samples, based on data from $\overline{CIUC}$ tests on RECOMPRESSION samples of several homogeneous cohesive soils with different inclinations, the anisotropy in q_f is due only to the anisotropy in pore pressure. The effective stress strength parameters $\bar{\phi}$ and $\bar{c}$ at q_f are practically unaffected by sample inclination. However, one should realize that $\overline{CIUC}$ test do not cause a rotation of principal planes during shear and thus, the directional variations in $\bar{\phi}$ and $\bar{c}$ could occur in the combined anisotropic behavior of OC soil.

The interpretation of the anisotropic behavior of both NC and OC Connecticut Valley varved clay measured using SHANSEP method of consolidation will be presented in Chapter 7.

2.3.4 Effect of Certain Stress System Variables on Undrained Shear Strength Behavior

This section summarizes experimental data showing the general effects of intermediate principal stress and K_o consolidation on undrained shear strength behavior. The effect of the intermediate principal stress is shown by comparing test data from $\overline{CK_oU}$ triaxial and plane strain tests. The results from NC $\overline{CK_oU}$ PSC tests ($\Delta\sigma_1 > \Delta\sigma_2 > \Delta\sigma_3$) and $\overline{CK_oUC}$ ($\Delta\sigma_1 > \Delta\sigma_2 = \Delta\sigma_3$) tests are compared for the intermediate principal stress effect in vertical loading. For

horizontal loading, results from $\overline{CK_oUPSE}$ tests ($\Delta\sigma_1>\Delta\sigma_2>\Delta\sigma_3$) and $\overline{CK_oUE}$ tests ($\Delta\sigma_1=\Delta\sigma_2>\Delta\sigma_3$) are compared. The influence of K_o consolidation (versus isotropic consolidation) is shown by comparing test data from $\overline{CIUC}$ and $\overline{CK_oUC}$ tests, and from $\overline{CIUE}$ and $\overline{CK_oUE}$ tests for vertical and horizontal loading respectively.

2.3.4(a) Effect of Intermediate Principal Stress

Tables 2.9(a) and 2.9(b) summarize data from $\overline{CK_oUPSC}$ versus $\overline{CK_oUC}$ tests and from $\overline{CK_oUPSE}$ versus $\overline{CK_oUE}$ tests on several normally consolidated undisturbed and remolded clays. For San Francisco Bay Mud, Atchfalaya Clay, and remolded Weald Clay, only data for vertical loading are available. These data show that shearing in plane strain compared to that in triaxial:

1. Increases q_f for both vertical and horizontal loadings. Such increases are larger for horizontal loading (up to 25%) than for the vertical loading (an increase of about 10 to 15%).
2. Generally decreases γ_f* for both vertical and horizontal loadings. Such decreases are, however, larger for vertical loading than for horizontal loading.
3. Significantly increases $\bar{\phi}$ at q_f for vertical loading, though $\bar{\phi}$ at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ is only

*Defined according to Table 2.9 for triaxial and plane strain tests based upon linear isotropic elasticity theory.

slightly increased. For horizontal loading, the difference in $\bar{\Phi}$ at q_f between TE and PSE test is somewhat smaller.

4. Does not consistently affect A_f. This is not to say, however, that increase in σ_2 will not increase Δu.

For an ideal isotropic soil, the intermediate principal stress will not affect the stress path in terms of octahedral stress (τ_{oct} versus $\bar{\sigma}_{oct}$). For such a material, difference in the magnitude of σ_2 does not change the pore pressure parameter "a" defined below:

$$\Delta u = \Delta\sigma_{oct} + a\,\Delta\tau_{oct} \qquad \text{(Eq. 2.9)}$$

For vertical loading, test data from $\overline{CK_oU}C$ and $\overline{CK_oU}PSC$ tests on remolded Weald clay and natural Hanney clay show unique stress path in terms of octahedral stress. For horizontal loading, however, (i.e., the data from $\overline{CK_oU}PSE$ and $\overline{CK_oU}E$ test), the octahedral stress paths are far from unique.

2.3.4(b) Effect of K_o Consolidation

Ladd (1971) summarized the effect of isotropic versus K_o consolidation undrained ($\overline{CU}$) triaxial compression tests on several normally consolidated clays. According to Ladd (1971), the results from NC clays generally show that K_o consolidation relative to the isotropic consolidation:

1. Has a variable effect on $q_f/\bar{\sigma}_{vc}$, the change

being $\pm$ 10 to 15%;

2. Generally decreases $\bar{\phi}$ and A_f at q_f;
3. Causes a substantial change in stress strain behavior, with a much smaller strain at failure, and more strain softening; and
4. Little change in $\bar{\phi}$ at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$.

For horizontal loading, NC test data on Boston Blue clay (Ladd and Varallyay, 1965) indicate a significant decrease in q_f and A_f and a slight increase in $\bar{\phi}$ with K_o consolidation.

For OC samples, test data from $\overline{CIUC}$, $\overline{CK_oUC}$, $\overline{CIUE}$ and $\overline{CK_oUE}$ tests on Kaolin (Amerasinghe and Perry, 1975) indicate essentially the same trends as reported by Ladd (1971) and Ladd and Varallyay (1965) with the exception that for the same loading direction (i.e., TC or TE tests), the isotropically consolidated specimens had the same failure envelope (i.e., the same $\bar{\phi}$ and $\bar{c}$).

GENERAL PROBLEMS WITH TESTING METHODS FOR INVESTIGATION DIRECTION VARIATION OF UNDRAINED SHEAR STRENGTH, (S_u)

Option	Type of Test	(1) Stress System	General Problems	Specific Problems		
				Element A	Element B	Element C
1	UUC	$\bar{A}_1$, B_2, C_2	Not plane strain Sample+disturb. + No initial shear stress	--	No rotation of principal planes during shear	Can fail by splitting
2	$\overline{CIUC}$	A_1, B_2, C_2	Not plane strain No initial shear stress	--	No rotation of principal planes during shear	Can fail by splitting
3	$\overline{CK_oU}$ (TC; DSS; TE)	A_1, B_1, C_1	Mixture of tri-axial and plane strain tests	σ_2 not plane strain	Interpretation of results	σ_2 not plane strain
4	$\overline{CK_oU}$ (PSC; DSS; PSE)	A_1, B_1, C_1	Requires specialized equipment	Apparatus friction	Interpretation of results	Apparatus friction
5	$\overline{CK_oU}$DSS	A_2, B_2, C_3	Interpretation of results	Unknown consolidation stress on other planes	Interpretation of results	Unknown consolidation stresses on other planes

NOTE (1) shown in Figure 2.2

Table 2.1

PREDICTION OF ANISOTROPY IN S_u FROM HANSEN AND GIBSON'S EQUATION FOR PLANE STRAIN LOADING (DUNCAN AND SEED, 1966(a))

Assuming $A_f = 1.0$; $K_o = 0.5$; $\bar{\phi} = 38°$; $\bar{c} = 0$

Loading Direction	α	$S_u(\alpha)/\bar{\sigma}_{vc}$	$\frac{S_u(\alpha)}{S_u(\alpha=64°)}$
Vertical loading[1]	64	0.37	1.0
Inclined loading[2]	0	0.22	0.59
Horizontal loading[3]	-26	0.20	0.54

Notes: (1) i.e., Data from PSC test

(2) i.e., Data from DSS (θ=0) test

(3) i.e., Data from PSE test

Table 2.2

INFLUENCE OF INHERENT ANISOTROPY ON MEASURED UNDRAINED SHEAR STRENGTHS OF HOMOGENEOUS CLAY FROM UUC TESTS

References	Clay	Relative Undrained Shear Strength		
		Vertical	45°	Horizontal
Jakobson (1955)	post glacial marine clay	1.0	1.13± .02	1.05± .05
Hvorslev (1960)	Vienna	1.0	0.92	0.87
Hvorslev (1960)	Little Belt	1.0	1.08	1.20
Duncan and Seed (1966(b))	S.F. Bay Mud	1.0	0.86	0.81
Duncan and Seed (1966(a))	Kaolin	1.0	0.75	0.87
Mitchell (1967)	Kaolin	1.0	0.87	1.06
D'Appolonia (1968)	Resediment Boston Blue Clay	1.0	0.9 ±0.03	0.82±0.03

Table 2.3

INFLUENCE OF INHERENT ANISOTROPY ON MEASURED UNDRAINED SHEAR STRENGTHS OF NON HOMOGENEOUS CLAY FROM UUC TESTS

Investigators	Clay	Relative Undrained Shear Strength			
		Vertical	45°	60°	Horizontal
Lo and Milligan (1967)	Welland (Block F)	1.00	0.63	0.56	0.99
Ward et al. (1965)	Heavily OC London Clay	1.00	0.60	--	1.23
Jakobson (1952)	Varved clay from Surte, Sweden	1.00	0.82	--	0.93
Eden (1955)	Varved clay from Step Rock Lake	1.00	0.51	0.53	1.18

Table 2.4

ANISOTROPY IN S_u MEASURED FROM $\overline{CK_oU}$PSC, $\overline{CK_oU}$DSS, AND $\overline{CK_oU}$PSE TEST (OPTION 4) OF SEVERAL NC COHESIVE SOILS

Type of Soil	Index Properties			$\overline{CK_oU}$ $S_u/\bar{\sigma}_{vc}$ (2)			$\frac{S_u(DSS)}{S_u(PSC)}$	$\frac{S_u(PSE)}{S_u(PSC)}$	Reference
	W_L	PI	LI	PSC	DSS	PSE			
Portmouth Clay	35	15	1.8	0.295	0.20	0.13	0.68	0.44	Ladd and Edgers (1972)
Haney Sensitive Clay	44	18	0.75	0.27	--	0.175	--	0.65	Vaid and Campanella (1974)
Boston Blue Clay	41	21	0.81	0.30	0.20	0.145	0.67	0.48	Ladd and Edgers (1972)
San Fran. Bay Mud	88	45	1.04	0.295	0.25	0.23	0.88	0.78	Duncan and Dunlop (1969)
AGS CH Clay	71	40	--	0.295	0.25	0.18	0.85	0.61	MIT and University of British Col.
Atchfalaya Clay	95	75	0.80	0.28	0.24 (1)	--	0.93	--	Ladd and Edgers (1972)

Notes: (1) Ave value

(2) $S_u = q_f \cos\bar{\phi}$ for PSC and PSE tests (data for q_f in Table 2.7)

$S_u = \tau_{h\ max}$ for DSS test.

Table 2.5

ANISOTROPY IN S_u MEASURED FROM $\overline{CK_oUC}$, $\overline{CK_oUE}$, AND $\overline{CK_oUDSS}$ TESTS (OPTION 3) OF SEVERAL SOFT CLAYS (FROM BEERE AND BJERRUM, 1973)

Type of Soil	Index Properties			$\overline{CK_cU}$ $S_u/\bar{\sigma}_{vc}$			$\frac{S_u(DSS)}{S_u(TC)}$	$\frac{S_u(TE)}{S_u(TC)}$
	W_L	PI	LI	TC	DSS	TE		
Bangkok Clay	150	85	.88	0.70	0.41	0.40	0.59	0.57
Drammen Plastic Clay	61	29	.69	0.40	0.30	0.15	0.75	0.38
Drammen Lean Clay	33	11	.73	0.34	0.22	0.09	0.65	0.26
Vaterland Clay, Oslo	42	16	.56	0.32	0.26	0.09	0.81	0.28
Studenter-lunden, Oslo	43	18	.33	0.31	0.19	0.10	0.61	0.32
Kimola	62	31	--	0.46	0.36	--	0.78	--
Sundland	58	28	--	0.40	0.30	--	0.75	--

Notes: (1) $p_c/\bar{\sigma}_{vo}$ ranges from 1.1 to 1.8: p_c = quasi-preconsolidation pressure.

(2) $S_u = q_f$ for $\overline{CK_oUC}$ and $\overline{CK_oUE}$ test and $S_u = \tau_{h\ max}$ from $\overline{CK_oUDSS}$ test

(3) Consolidated with $\bar{\sigma}_{vc} = \bar{\sigma}_{vo}$

Table 2.6

SUMMARY OF ANISOTROPIC DATA FROM SEVERAL NC COHESIVE SOILS

(a) Plane Strain Loading

Type of Soil	Index Properties				PSC					PSE					Reference
					$(\sigma_1 - \sigma_3)_{max}$				$\bar{\phi}^o$ at MO	$(\sigma_1 - \sigma_3)_{max}$				$\bar{\phi}^o$ at MO	
	W_L	W_P	PI	LI	$q_f/\bar{\sigma}_{vc}$	A_f	γ_f	$\bar{\phi}^o$		$q_f/\bar{\sigma}_{vc}$	A_f	γ_f	$\bar{\phi}^o$		
Portmouth Clay	35	20	15	1.8	0.35	0.94	0.6	31.8	--	0.155	1.01	11.8	31.5	--	Ladd and Edger (1972)
Haney Sensitive Clay	44	26	18	0.75	0.296	--	0.8	35.2	31.6	0.211	--	21.0	34.3	34.3	Vaid and Campanella (1974)
Boston Blue Clay	41	20	21	0.81	0.34	0.8	0.8	29	34	0.19	1.02	8.6	≤ 40	≤ 40	Ladd and Edger (1972)
San Fran. Bay Mud	88	43	45	1.04	0.37	1.12	7.2	38	--	0.28	0.70	20.4	35	--	Duncan and Seed (1966(b))
AGS CH Clay	71	31	40	--	0.37	0.83	2.0	36.5	40.5	0.22	0.70	16.4	36.5	37	MIT & Univ. of British Col.

(b) Triaxial Loading

Type of Soil	Index Properties				TC					TE					Reference
					$(\sigma_1 - \sigma_3)_{max}$				$\bar{\phi}^o$ at MO	$(\sigma_1 - \sigma_3)_{max}$				$\bar{\phi}^o$ at MO	
	W_L	W_P	PI	LI	$q_f/\bar{\sigma}_{vc}$	A_f	γ_f	$\bar{\phi}^o$		$q_f/\bar{\sigma}_{vc}$	A_f	γ_f	$\bar{\phi}^o$		
Atchafalaya Clay	95	20	75	0.80	0.24	1.4	4.5	21	25	0.22	0.9	22.5	24	25	Ladd and Edger (1972)
Haney Sensitive Clay	46	26	18	0.75	0.268	--	18	21.4	29.8	0.168	--	19.5	33.8	33.8	Vaid and Campanella (1974)
Boston Blue Clay	41	20	21	0.81	0.33	0.6	0.45	26.5	33	0.155	1.17	15	38	38	Ladd and Edger (1972)

NOTES: (1) $\gamma_t = 1.5\ \varepsilon_f$ for Triaxial Tests and $\gamma_f = 2\varepsilon_f$ for Plane Strain tests.

(2) $\bar{\sigma}_{vc} = 1.5$ to $2.0\ \bar{\sigma}_{vm}$

Table 2.7

EFFECT OF STRESS HISTORY ON ANISOTROPIC SHEAR STRENGTH BEHAVIOR OF BOSTON BLUE CLAY MEASURED FROM PLANE STRAIN COMPRESSION AND EXTENSION TESTS (FROM LADD ET AL., 1971)

OCR(1)	K_o	$\frac{S_u(PSE)}{S_u(PSC)}$	$\frac{A_f(PSE)}{A_f(PSC)}$	$\frac{\varepsilon_f(PSE)}{\varepsilon_f(PSC)}$
1	0.51	0.56	1.3	10.8
2	0.71	0.65	2.4	4.2
3	0.87	0.71	3.5	3.9

Note: (1) Using SHANSEP Method of consolidation.

Table 2.8

EFFECT OF σ_2 ON VERTICAL AND HORIZONTAL UNDRAINED SHEAR STRENGTH BEHAVIOR OF SEVERAL NC COHESIVE SOILS

(a) Vertical Loading

Type of Soil	Index Properties				Data at q_f								$\bar{\phi}^o$ at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$		Reference
					$q_f/\bar{\sigma}_{vc}$		A_f		γ_f		$\bar{\phi}^o$				
	W_L	W_p	PI	LI	PSC	TC	PSC	TC	PSC	TC	PSC	TC	PSC	TC	
Remolded Weald Clay	46	20	26	0.54	0.28	0.26	1.7	2.0	4.0	9.0	27.1	25.9	27.4	26.2	Henkel and Wade (1966)
Boston Blue Clay	41	20	21	0.81	0.34	0.33	0.8	0.6	0.8	0.45	29	26.5	34.0	33.0	Ladd and Edgers (1972)
Haney Sensitive Clay	44	26	18	0.75	0.30	0.27	--	--	0.8	.50	25.2	21.4	31.6	29.8	Vaid and Campanella (1974)
San Fran Bay Mud	88	43	45	1.04	0.37	0.35	1.12	1.11	7.2	9.3	38	34.3	--	--	Duncan and Seed (1966)
Atchfalaya Clay	95	20	75	0.80	0.31	0.24	0.8	1.4	1.4	4.5	24	21	28	25	Ladd and Edgers (1972)

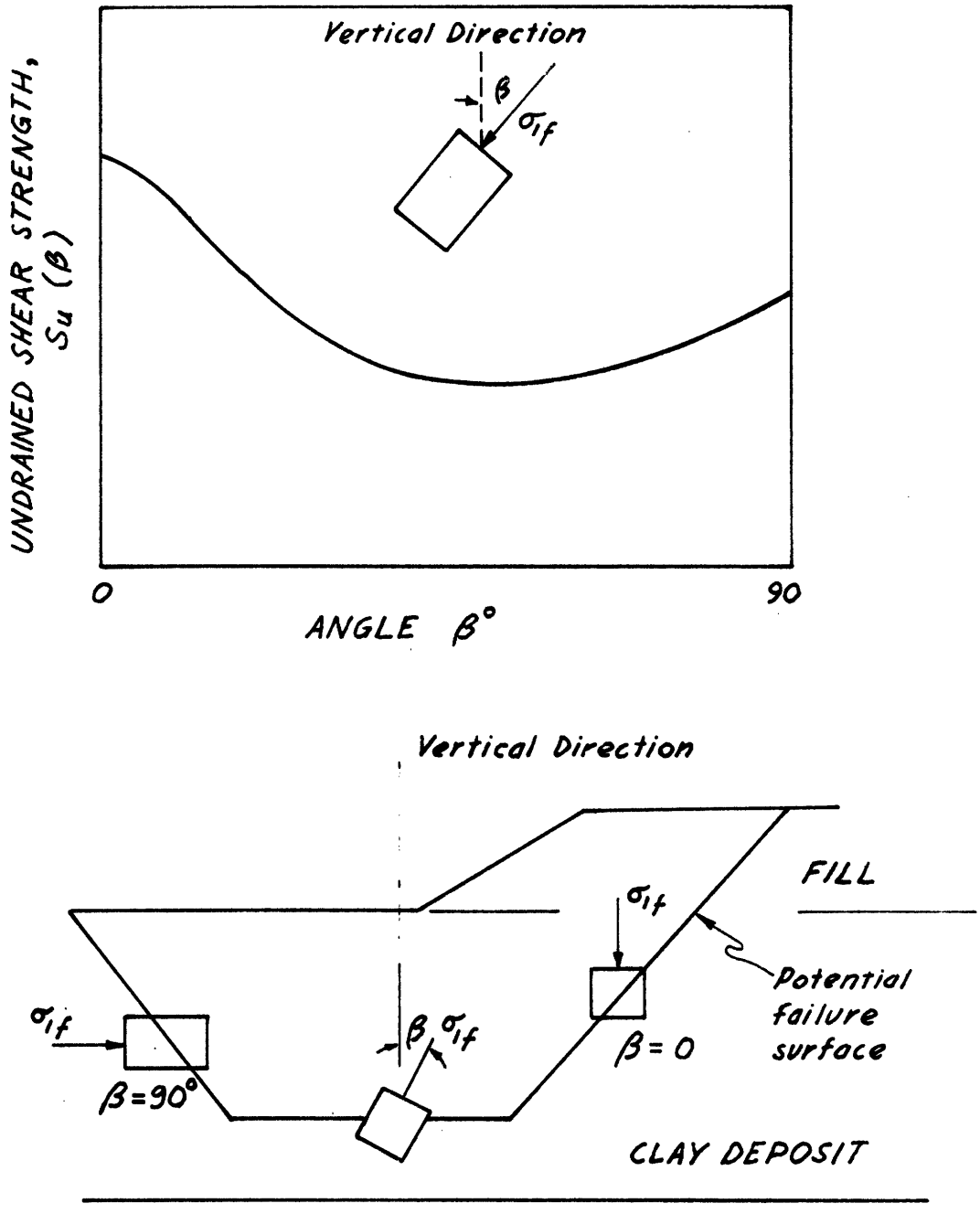
(b) Horizontal Loading

Type of Soil	Index Properties				Data at q_f								$\bar{\phi}^o$ at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$		Reference
					$q_f/\bar{\sigma}_{vc}$		A_f		γ_f		$\bar{\phi}^o$				
	W_L	W_p	PI	LI	PSE	TE	PSE	TE	PSE	TE	PSE	TE	PSE	TE	
Boston Blue Clay	41	20	21	0.81	0.19	0.16	1.01	1.17	8.6	15.0	40	38	40	38	Ladd and Edgers (1972)
Haney Sensitive Clay	44	26	18	0.75	0.21	0.17	--	--	21.0	19.5	34.3	33.8	34.3	33.8	Vaid and Campanella (1974)

NOTES: (1) $\bar{\sigma}_{vc} \geq 1.5$ to 2.0 $\bar{\sigma}_{vm}$

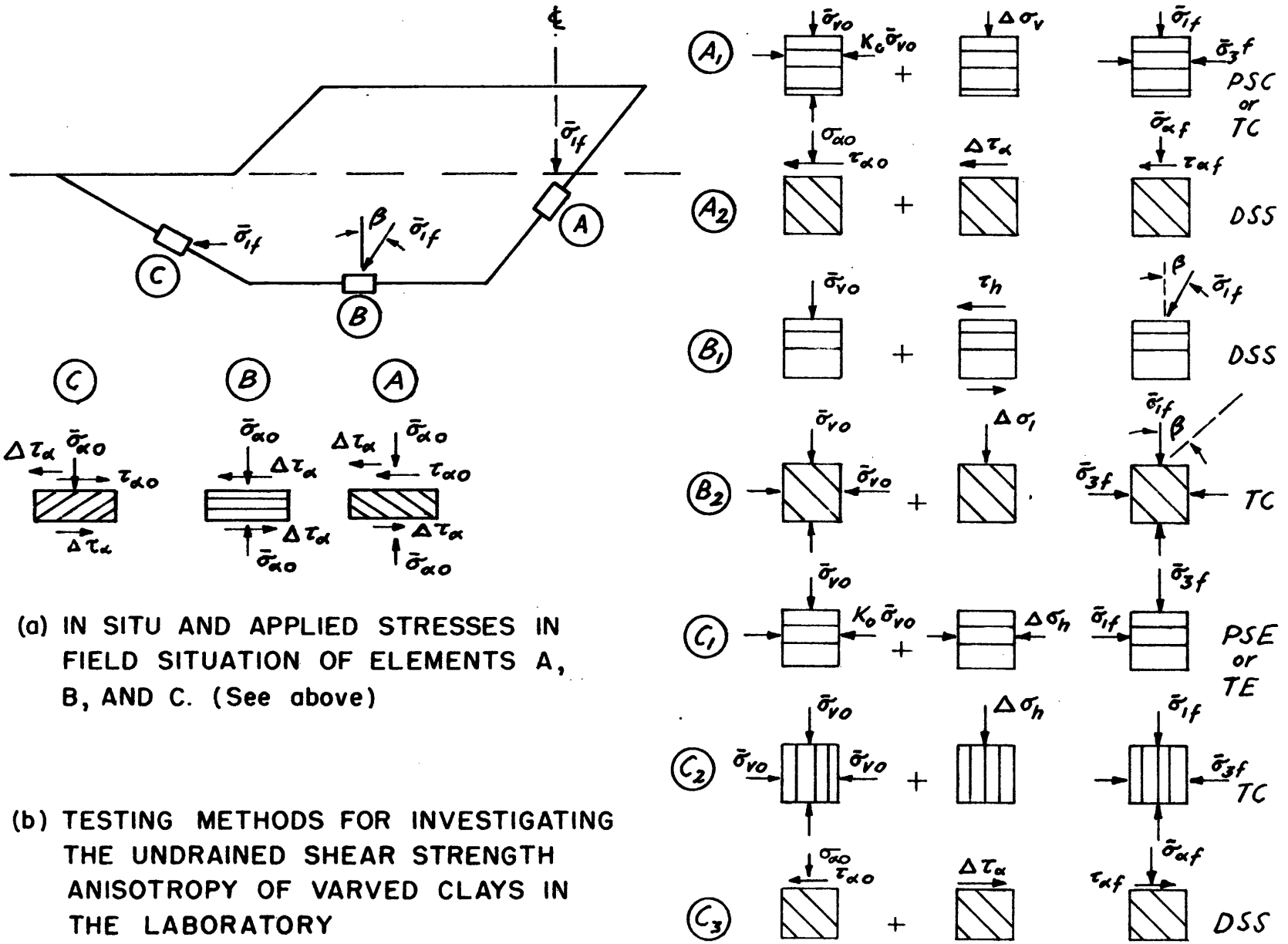
(2) $\gamma_f = 1.5\ \gamma_f$ for Triaxial test and $\epsilon_f = 2.0\ \epsilon_f$ for Plane Strain test.

Table 2.9



DEFINITION OF UNDRAINED SHEAR STRENGTH ANISOTROPY

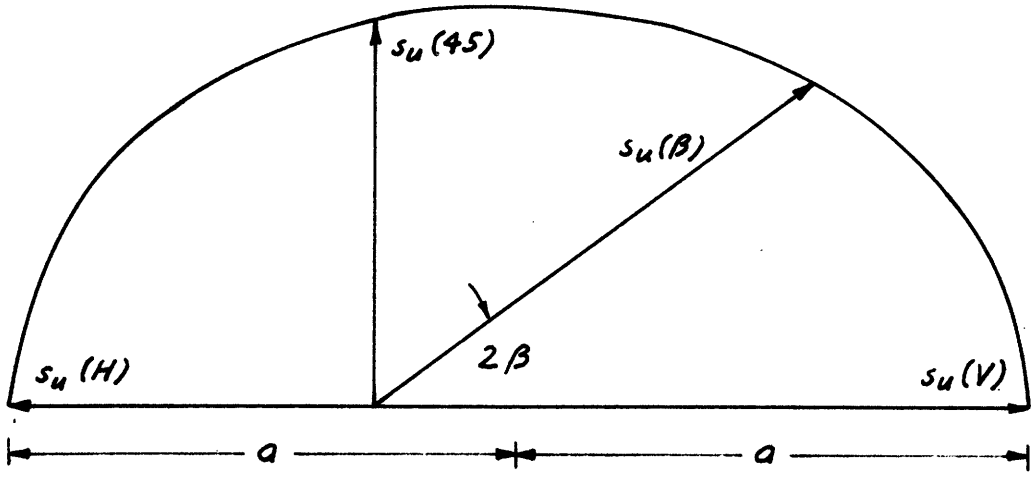
FIGURE 2.1



(a) IN SITU AND APPLIED STRESSES IN FIELD SITUATION OF ELEMENTS A, B, AND C. (See above)

(b) TESTING METHODS FOR INVESTIGATING THE UNDRAINED SHEAR STRENGTH ANISOTROPY OF VARVED CLAYS IN THE LABORATORY

FIGURE 2.2



$$a = 0.5\left[s_u(H) + s_u(V)\right] \qquad \frac{b}{a} = \frac{s_u(45)}{\sqrt{s_u(V)\,s_u(H)}}$$

$$s_u(\beta) = \frac{M + \sqrt{M^2 + LN}}{L}$$

where $M = \cos 2\beta \sqrt{a^2 - N}$

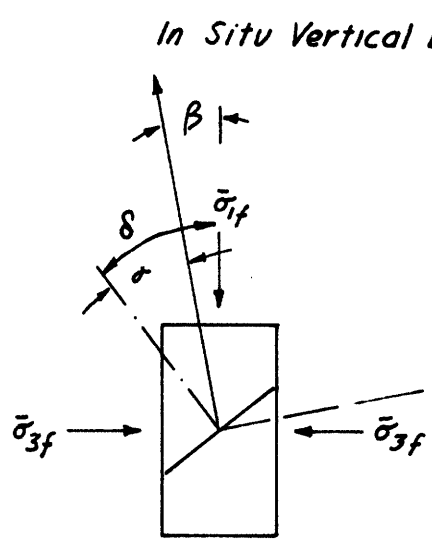
$N = s_u(V)\,s_u(H)$

$L = \cos^2 2\beta + \left(\frac{a}{b}\right)^2 \sin^2 2\beta$

β = angle between vertical and $\bar{\sigma}_{1f}$

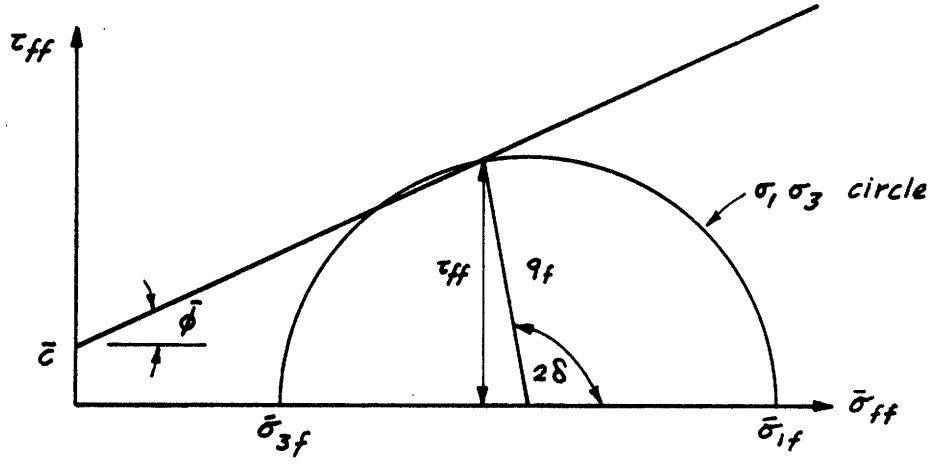
DAVIS-CHRISTIAN (1971) ELLIPTICAL ANISOTROPIC STRENGTH RELATIONSHIP

FIGURE 2.3



$$\alpha = \delta - \beta \qquad \text{Eq. 2.5}$$

a) DEFINITION OF FAILURE PLANE ORIENTATION δ



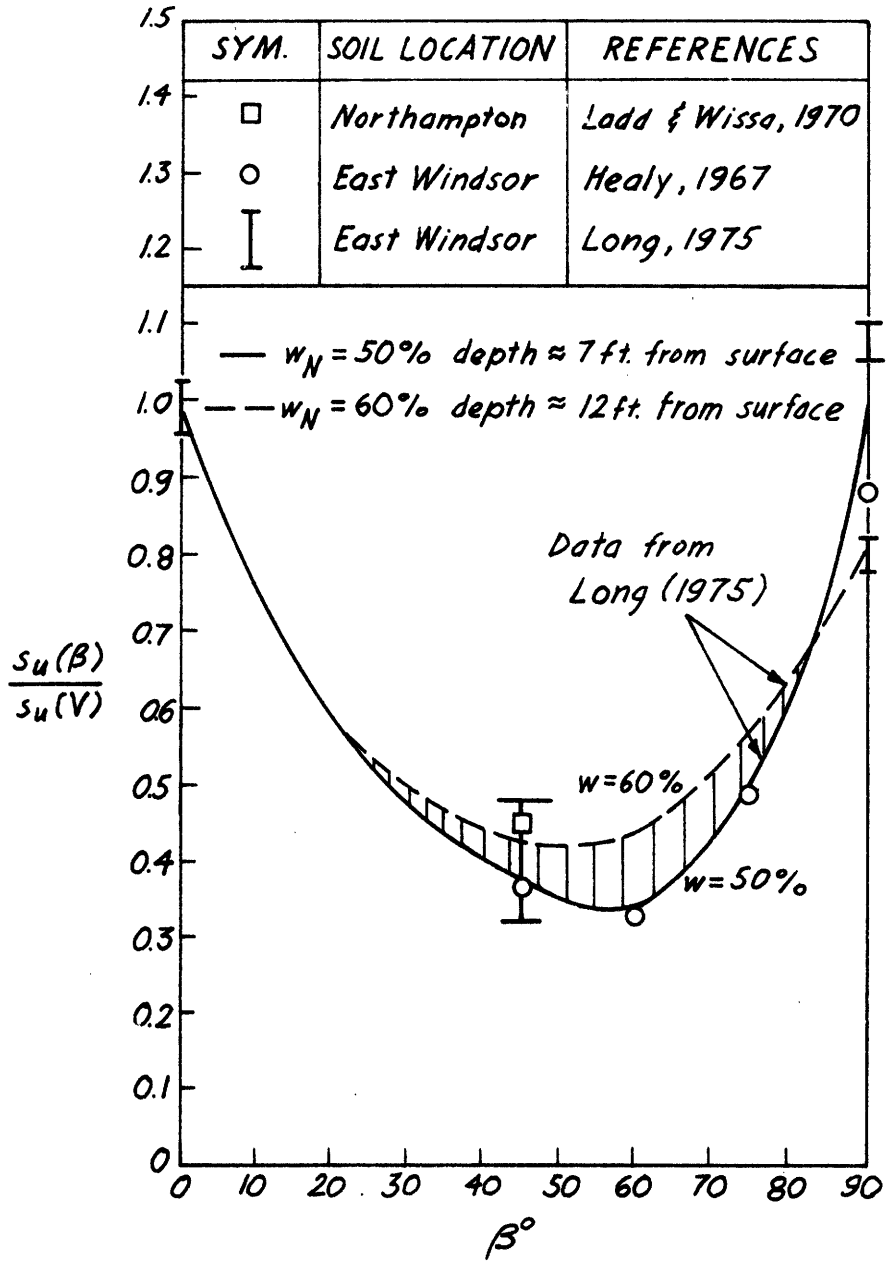
From Geometry

$$q_f = \frac{\bar{c}\cos\bar{\phi} + \bar{P}_f \sin\bar{\phi}}{\sin(2\delta - \phi)} \qquad \text{Eq. 2.6}$$

$$\bar{c} = \bar{c}(\alpha)$$
$$\bar{\phi} = \bar{\phi}(\alpha) \qquad \text{Eq. 2.7}$$

b) STATE OF STRESS AT FAILURE IN AN ANISOTROPIC CLAY

MODIFIED MOHR-COULOMB FAILURE CRITERIA FOR CROSS ANISOTROPIC SOIL (After Merkle, 1971) FIGURE 2.4



SUMMARY OF RESULTS FROM UUC TESTS ON CONNECTICUT VALLEY VARVED CLAYS (Data from Healy, 1967; Ladd and Wissa, 1970; and Long, 1975)

FIGURE 2.5

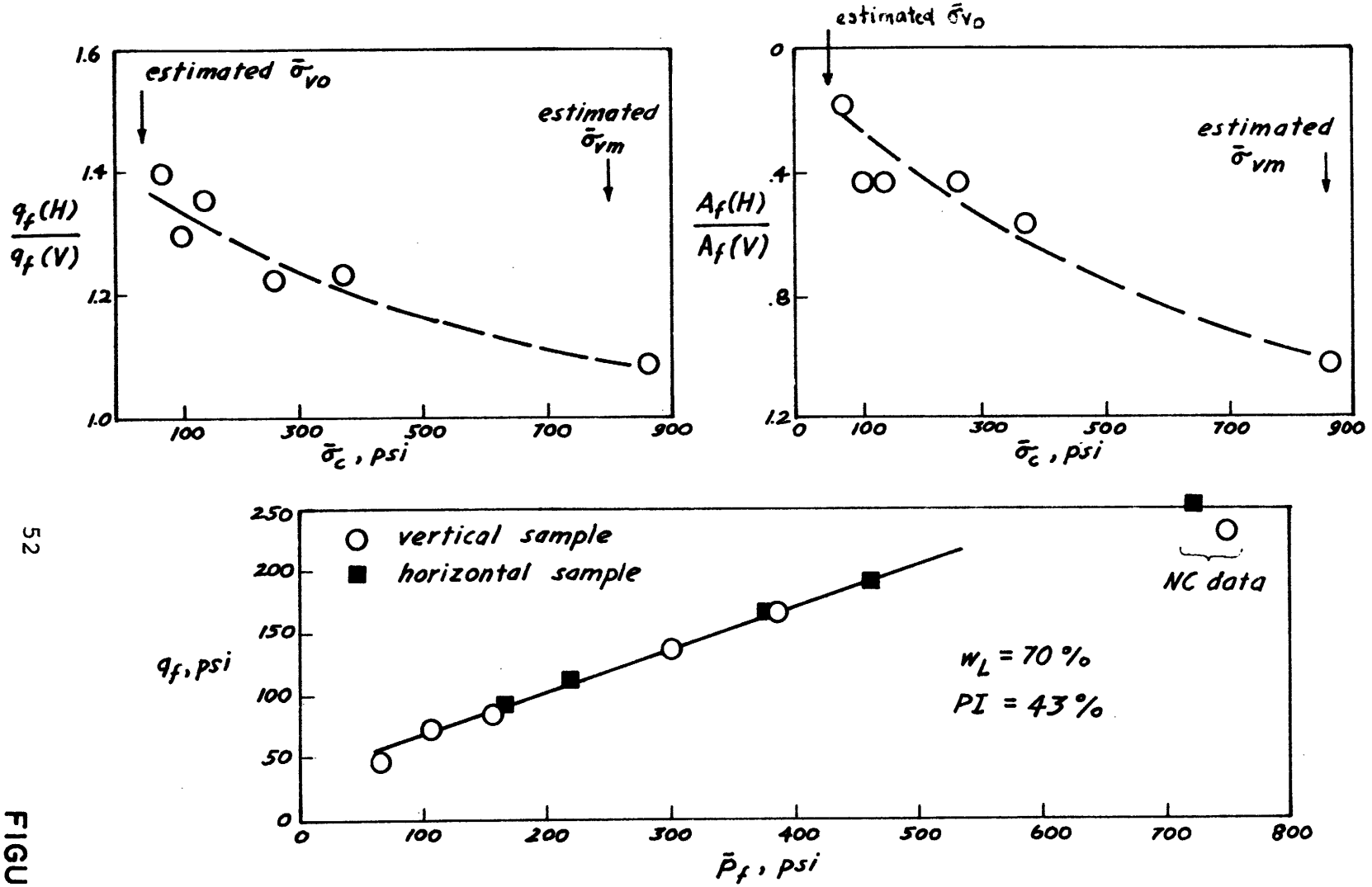


FIGURE 2.6 ANISOTROPIC BEHAVIOR OF LONDON CLAY AT THE SITE OF THE ASHFORD COMMON SHAFT MEASURED FROM $\overline{CIUC}$ TESTS ON VERTICAL AND HORIZONTAL SAMPLES (Data from Bishop et al, 1965)

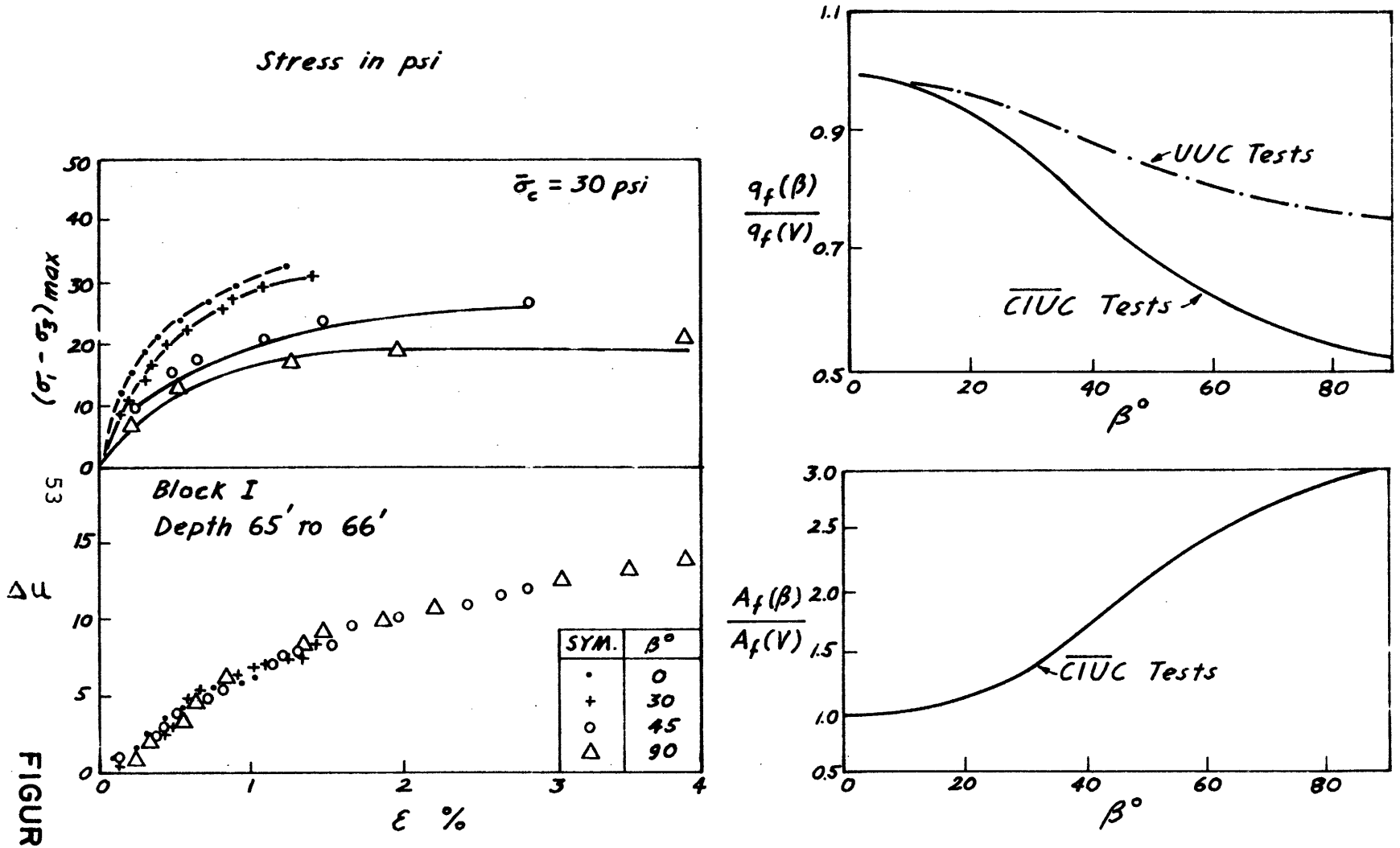
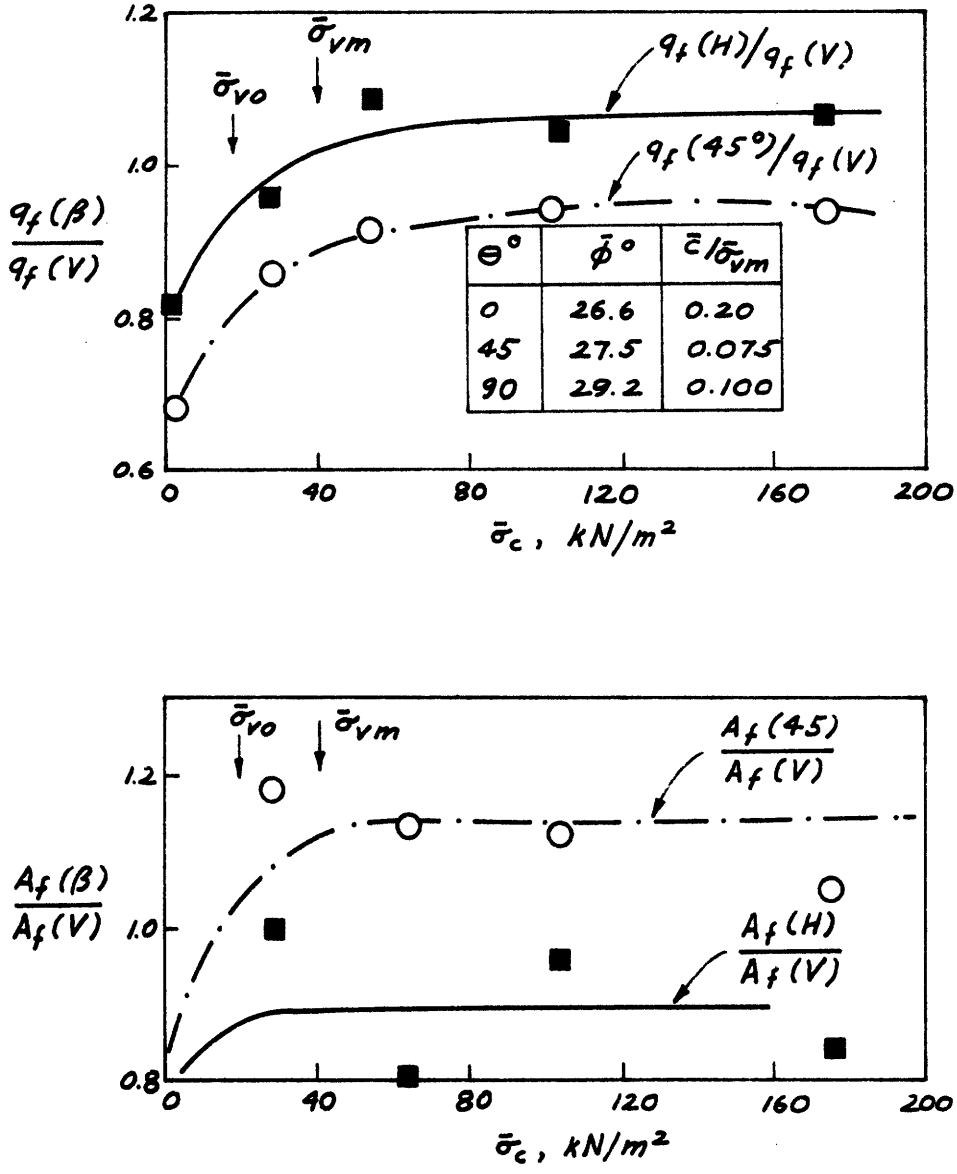


FIGURE 2.7

TEST DATA FROM $\overline{CIUC}$ TESTS ON OC WELLAND CLAY SAMPLES WITH DIFFERENT INCLINATIONS (From Lo, 1965; & Lo, 1966)



ANISOTROPY IN q_f AND A_f OF FULFORD SOFT CLAYEY SILT MEASURED FROM CIUC TESTS ON SAMPLES WITH DIFFERENT INCLINATIONS (Data from Perry & Nadarajah, 1974) FIGURE 2.8

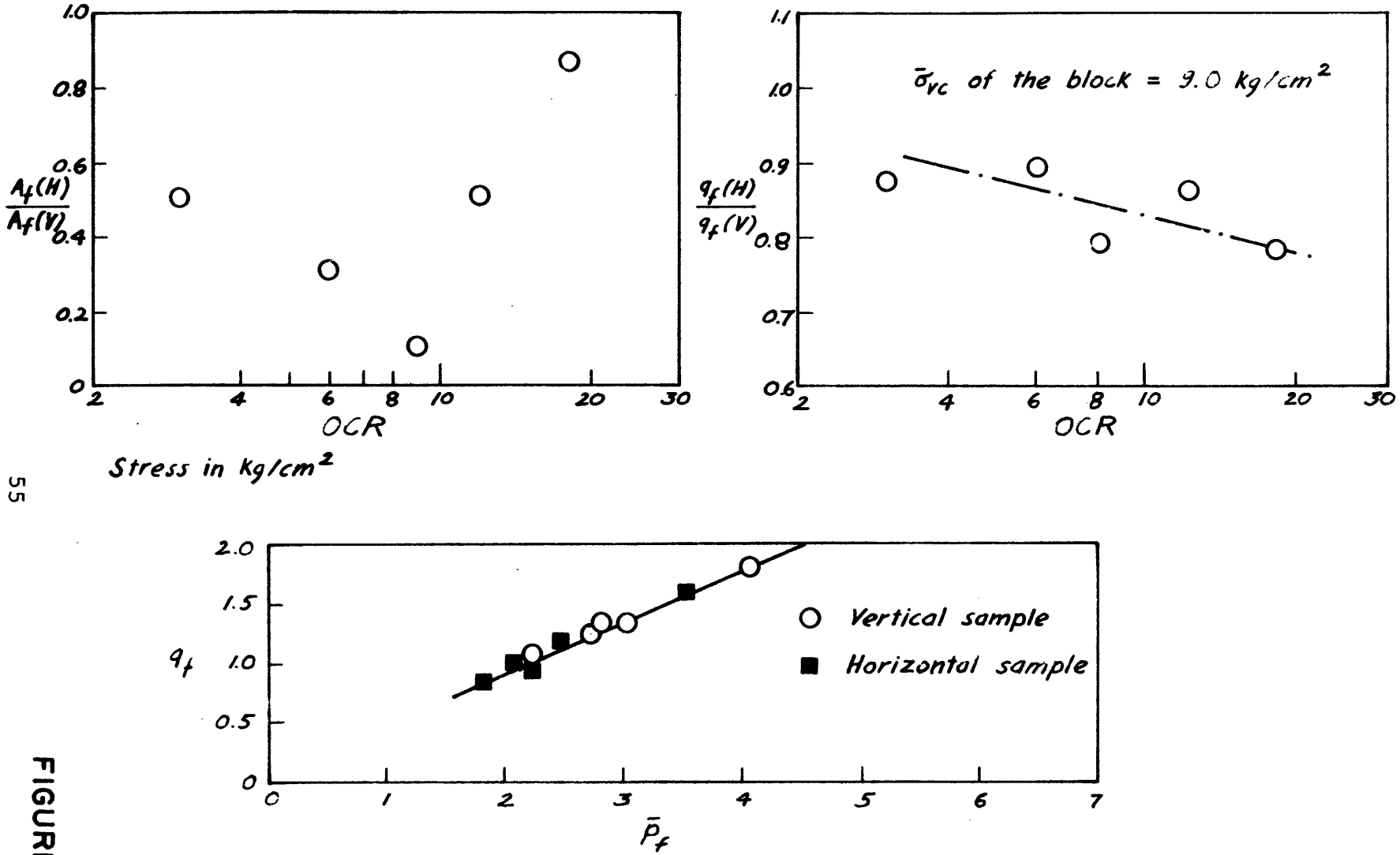


FIGURE 2.9 TEST DATA FROM $\overline{CIUC}$ TESTS ON LABORATORY PREPARED VERTICAL AND HORIZONTAL SAMPLES OF OC KAOLIN (Duncan & Seed, 1966 a)

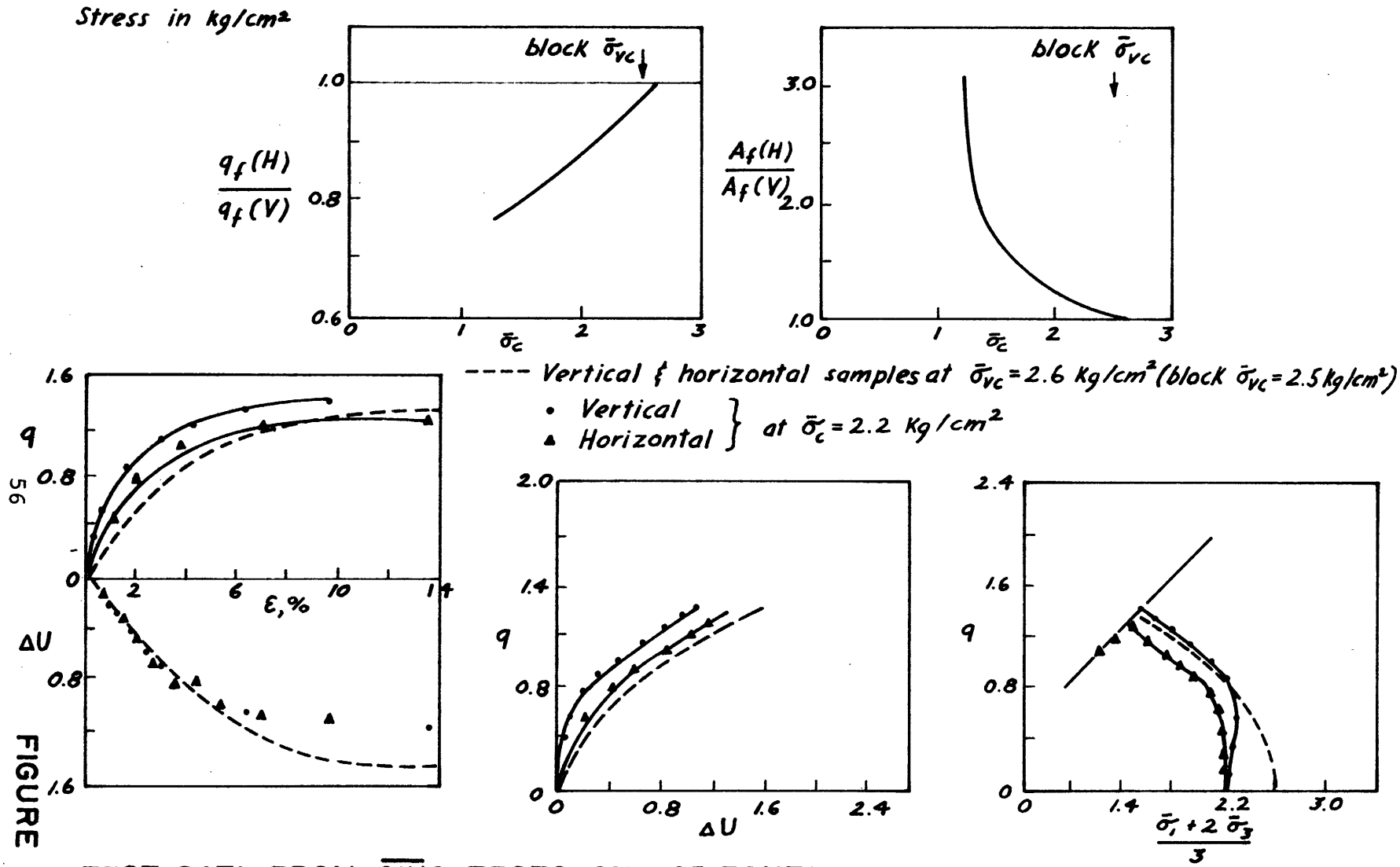
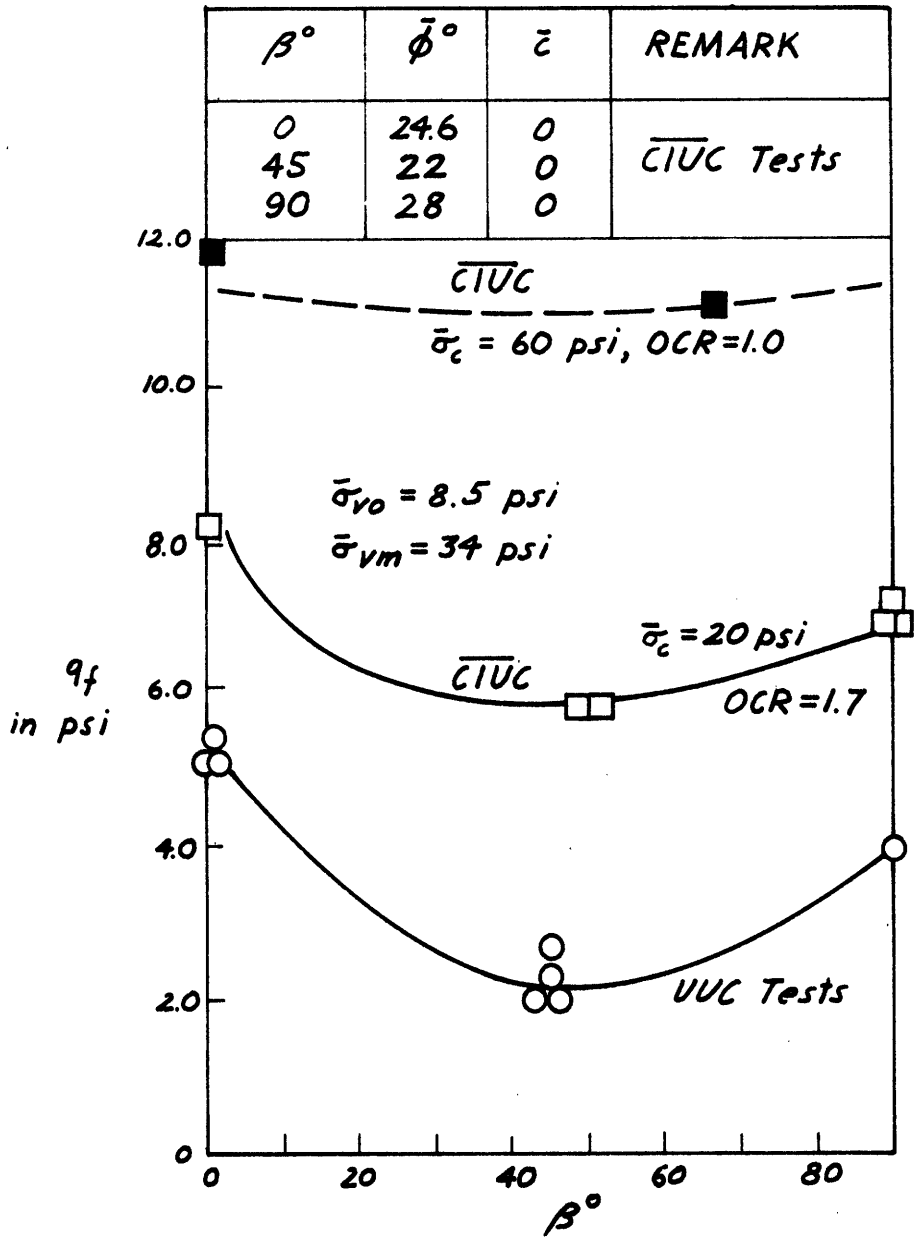


FIGURE 2.10 TEST DATA FROM $\overline{CIUC}$ TESTS ON HORIZONTAL AND VERTICAL SAMPLES OF A LABORATORY PREPARED LIGHTLY OVERCONSOLIDATED KAOLIN
(Data from Mitchell R. J., 1972)



EFFECT OF $\bar{\sigma}_c$ ON THE MEASUREMENT OF UNDRAINED SHEAR STRENGTH ANISOTROPY OF CONNECTICUT VALLEY VARVED CLAY (After Long, 1975)

FIGURE 2.11

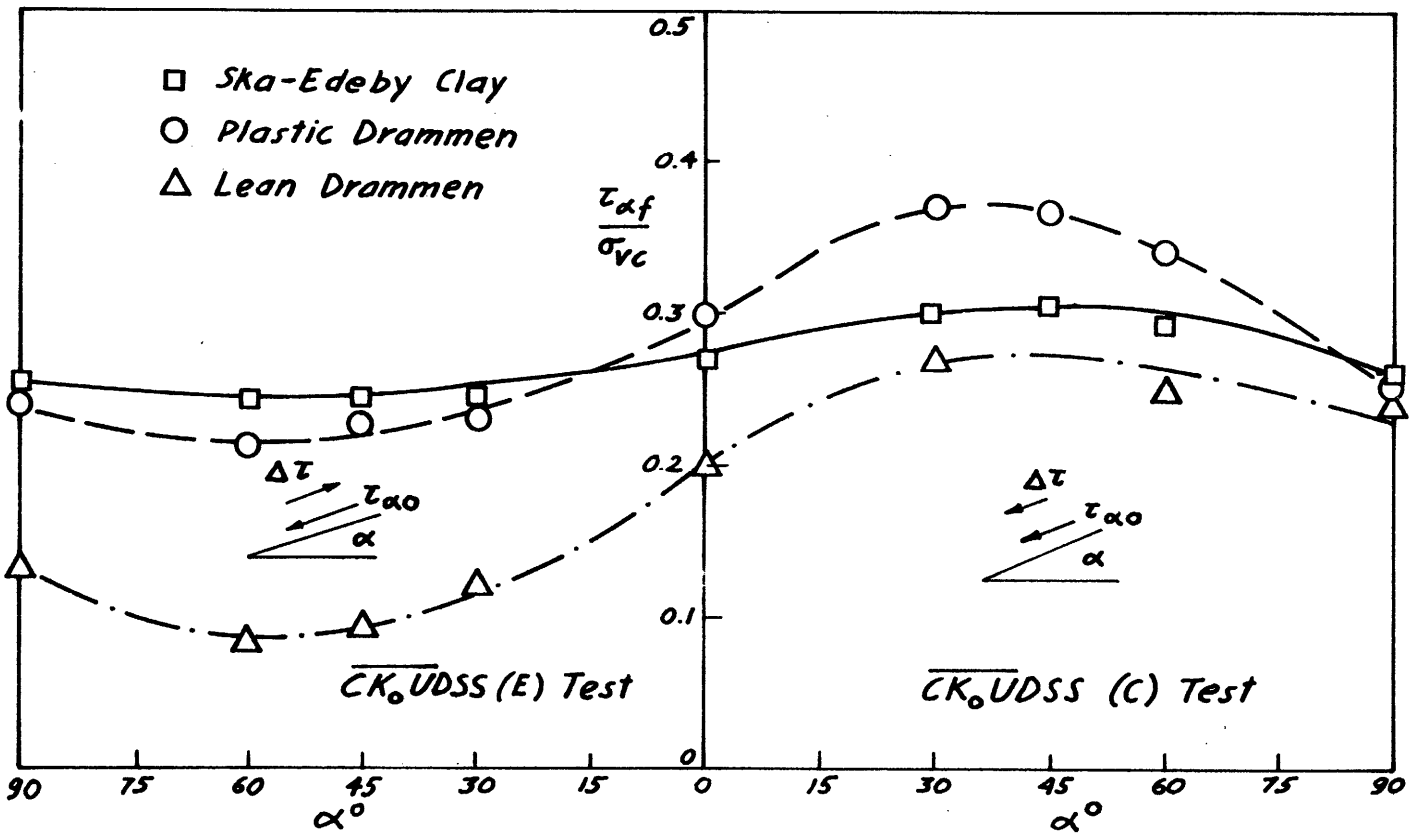
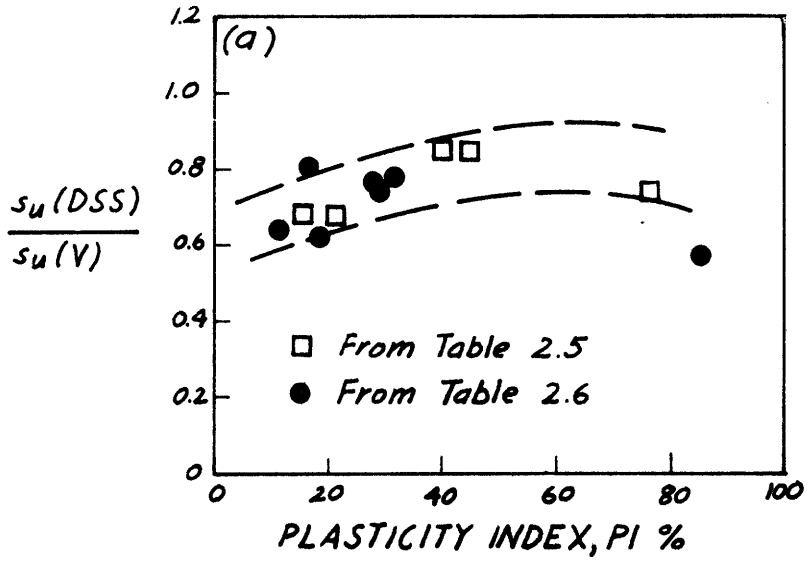
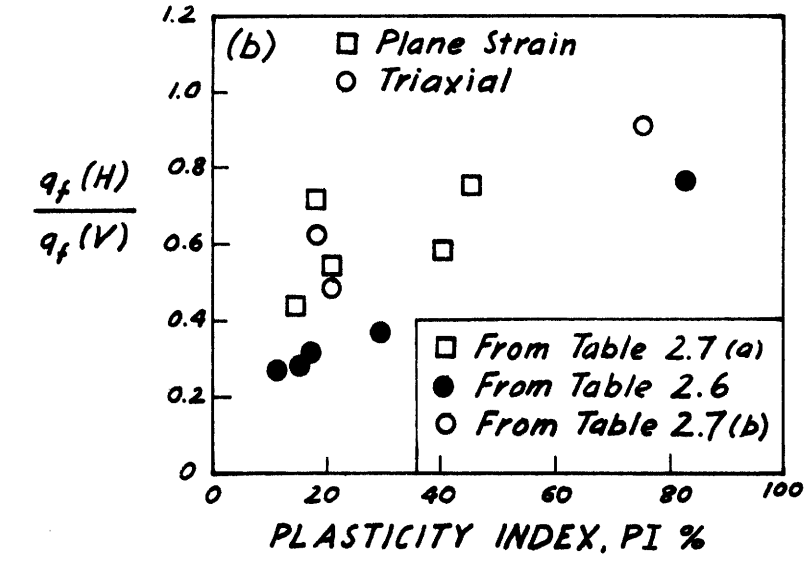


FIGURE 2.12

ANISOTROPY IN s_u ($s_u = \tau_{\alpha f max}$) FROM $\overline{CK_oUDSS}$ TESTS ON SAMPLES WITH DIFFERENT INCLINATIONS (OPTION 5) OF THREE SOFT CLAYS(Data from Soydemir, 1972)





ANISOTROPIC UNDRAINED STRENGTH RATIO VS PLASTICITY INDEX

FIGURE 2.13

3. EXPERIMENTAL PROGRAM

3.1 GENERAL SCOPE AND OBJECTIVES

This chapter provides a general description of varved clays from four locations, namely: East Windsor in Connecticut, Amherst and Northampton in Massachusetts, and Hackensack Valley in New Jersey. The similarity in the general properties of these soils is discussed. The experimental program, and the details of the testing conditions used for the soil samples at each location are presented. The last portion of the chapter describes the standard testing procedures used for triaxial compression, triaxial extension, direct simple shear, plane strain compression and plane strain extension tests.

3.2 EXPERIMENTAL PROGRAM

The plan of the laboratory testing program was to make a comprehensive study of the anisotropic behavior of the four varved clays. Additional test results for the Northampton and and Hackensack Valley varved clays are reported in Lacasse et al. (1972) and Saxena et al. (1974). Tables 3.1(a) and 3.1 (b) present a summary of the experimental program. It consisted of:

1. A series of unconsolidated-undrained triaxial

compression (UUC) tests run with different orientations of the varves;

2. A series of isotropically consolidated-undrained triaxial compression ($\overline{CIUC}$) tests with pore pressure measurement run with different orientations of varves and at varying OCR;
3. A series of K_o consolidated-undrained triaxial compression ($\overline{CK_oUC}$) tests with pore pressure measurement at varying OCR;
4. A series of isotropically consolidated-undrained triaxial extension ($\overline{CIUE}$) tests with pore pressure measurement run on vertical and horizontal samples at varying OCR;
5. A series of K_o consolidated-undrained triaxial extension ($\overline{CK_oUE}$) tests with pore pressure measurement at varying OCR;
6. A series of K_o consolidated-undrained direct simple shear ($\overline{CK_oUDSS}$) tests at varying OCR;
7. One special K_o consolidated-undrained direct simple shear test run on a normally consolidated sample where the normal and shear stresses were applied during consolidation before the sample was sheared by increasing the horizontal shear stress to simulate plane strain compression ($\overline{CK_oUDSS}$ (C) test);
8. One special K_o consolidated-undrained direct

simple shear test run on a normally consolidated sample where the normal and shear stresses were applied during consolidation before the sample was sheared by reversing the direction of the horizontal shear stress to simulate plane strain extension ($\overline{CK_oU}$DSS (E) test);

9. One K_o consolidated plane strain compression ($\overline{CK_oU}$PSC) test run on a normally consolidated sample from Northampton;
10. Two K_o consolidated undrained plane strain ($\overline{CK_oU}$PSE) tests run on a normally consolidated sample from Northampton;
11. A series of consolidated drained direct shear (CDDS) tests at different OCR on Northampton varved clay;
12. Two isotropically consolidated drained triaxial unloading (CID (U)) tests on Northampton varved clay.

Both SHANSEP and RECOMPRESSION* methods of consolidation were used for items 3 and 5. The locations and the testing conditions of soil samples are presented in Section 3.3.

Chapter 4 will present the investigation of the normalized behavior of these soils, and a comparison of their NSP.

*RECOMPRESSION--the method of consolidation whereby the sample is consolidated at the desired OCR with a vertical consolidation stress that is less than the in situ maximum past pressure.

The testing program included $\overline{CK_oUC}$, $\overline{CK_oUE}$, and $\overline{CK_oUDSS}$ tests on varved clays from East Windsor, Amherst, Northampton, and Hackensack Valley. NC $\overline{CIUE}$ test vertical samples and NC $\overline{CIUC}$ test samples with different inclinations, of East Windsor and Amherst varved clays, were also tested to compare their NSP. All samples were consolidated according to the SHANSEP approach.

The effect of stress system on the NSP, obtained via the SHANSEP approach, of Connecticut Valley varved clays is presented in Chapter 5. This includes:

1. The effect of initial shear stress (i.e., isotropic versus K_o consolidation);
2. The effect of the intermediate principal stress;
3. The effect of testing method for shearing parallel to the varves.

$\overline{CK_oUC}$ and $\overline{CIUC}$ tests on vertical samples which had OCR's of 1,2, and 4 were run to study the effect of initial shear stress on NSP for vertical loading. For horizontal loading, $\overline{CK_oUE}$ and $\overline{CIUE}$ tests were run on vertical samples with OCR's of 1, 2, and 4. $\overline{CIUC}$ tests on vertical samples, and $\overline{CIUE}$ tests on horizontal samples, at OCR's of 1, 2, and 4, were run to study the effect of the magnitude of the intermediate principal stress for vertical loading. Similarly, $\overline{CIUC}$ tests on horizontal samples and $\overline{CIUE}$ tests on vertical samples at OCR's of 1,2, and 4, were performed to study the effect of σ_2 on NSP for horizontal loading. Results from $\overline{CK_oUPSC}$ and $\overline{CK_oUPSE}$ tests (Lacasse et al., 1972) were also used to

compare with data from $\overline{CK_oUC}$ and $\overline{CK_oUE}$ tests for the investigation of the effect of σ_2. The special $\overline{CK_oU}$DSS (C) and $\overline{CK_oU}$DSS (E) tests on inclined samples from East Windsor also served this purpose. $\overline{CIUC}$ tests on inclined samples (θ=45), and $\overline{CK_oU}$DSS tests on vertical samples at varying OCR, were performed to study the effect of testing method for shearing parallel to the varves. Test results from drained direct shear tests on samples from Northampton (Lacasse et al., 1972) were also used for this purpose.

Chapter 6 will study the effect of "sample disturbance" on NSP. UUC tests on samples with different inclinations, $\overline{CIUC}$ tests on samples with different inclinations consolidated by the RECOMPRESSION method and $\overline{CK_oUC}$ and $\overline{CK_oUE}$ tests on vertical samples consolidated by the RECOMPRESSION method were performed for this study. The NSP from these tests are then compared with the NSP from SHANSEP samples which have the same OCR.

Interpretation of the anisotropic undrained strength behavior of varved clays, which is based upon the results from Chapter 5, is presented in Chapter 7, while Chapter 8 provides the "best estimates" of soil parameters for use in the design of structures on varved clays.

3.3 DESCRIPTION OF VARIOUS CLAYS

This section provides a general description of the soil samples and their index properties. Details of the geological

formation of the Connecticut Valley varved clays are presented in Ladd and Wissa (1970); the geological formation of the Hackensack Valley varved clay is discussed in Saxena et al., (1974).

Briefly, the Connecticut Valley varved clays were deposited in glacial Lake Hitchcock during the late Pleistocene time. The bedrock of this area was covered by a widespread but discontinuous blanket of till. After deposition of the varved clays, glacial Lake Hitchcock was drained when the natural dam at Rocky Hill, Connecticut breached. As a result, the varved clays along the Connecticut River were then deeply eroded. The eroded surface was subsequently covered by post glacial sand deposits. The upper present-day surface of the varved clay is thus not necessarily the same as its original surface under Lake Hitchcock. The varve thicknesses vary typically from 1/2 to 2 1/2 inch. The thickness often decreases with increasing elevation because of the disappearance of the glaciers with time.

The Hackensack Valley varved clay was obtained from the Hackensack Meadows. It is a tidal marsh occupying the area of a glacial lake formed by the obstruction of a preglacial valley with glacial drift in the vicinity of Perth Amboy. In recent times, extensive areas of the meadows have been covered with uncontrolled fill varying in thickness from 2 to 12 ft.

3.2.1 East Windsor Varved Clay

Two undisturbed one cubic foot block samples of East Windsor varved clay from the same depth, were provided by Prof. R.P. Long of the University of Connecticut. They were obtained from a clay pit located off Route 5 and owned by the Kelsey-Ferguson Brick Co. in East Windsor, Connecticut. The clay is greyish in color and the varves show a gradual transition in grain size from top to bottom; but upon drying, the "clay" and "silt" layer can be easily distinguished by their dark and light colors, respectively. The thickness of these layers generally varies from 1/16 in. to 3/16 in.

The block samples were taken from the clay pit at a depth of about 11 ft. Five ft of medium sand and six ft of brown varved clay overlie these samples. They have an estimated effective overburden pressure of 0.6 kg/cm^2. The samples generally had very uniform properties, based on torvane strengths and natural bulk water contents. Only portions of these samples which had the same torvane strength and moisture content were used for testing.

Table 3.2 shows the index and the engineering properties of the East Windsor varved clay. According to Casagrande's plasticity chart with the basis of limited data, the clay portion can be classified as CH material and the silt portion can be classified as ML material. Fig. 3.1 shows results from three standard consolidation tests on samples from these two blocks. The strains at the end of primary consolidation,

determined by the $\sqrt{t}$ method, are used in the plot of the vertical strain versus log $\bar{\sigma}_{vc}$. A small variation in the values of the maximum past pressure ($\bar{\sigma}_{vm}$) and the virgin compression ratio (CR) is noted in these tests. The average $\bar{\sigma}_{vm}$ of this clay is 2.4 kg/cm^2. The in situ OCR is therefore about four. Table 3.3 gives the details of the testing conditions and the types of tests performed on the samples.

3.2.2 Amherst Varved Clay

Under sponsorship of the Massachusetts Department of Public Works and the US Department of Transportation Federal Highway Administration for research on varved clays, the Department of Civil Engineering at MIT obtained 5 in. diameter undisturbed samples from a boring near the Route 116 by-pass and North Hadley Road at the University of Massachusetts, Amherst. These samples were obtained by using an Osterberg type fixed piston sampler. Field vane tests using the Geonor apparatus were also run adjacent to the boring.

The clay is generally grayish and brown in color, and the varve thickness varies from 1/2 in. to 1 in. The varve thicknesses of the Amherst varved clay are larger than those from East Windsor. Figure 3.2 shows the soil profile, Atterberg limits and compressibility data. The soil profile consists of a thick deposit of varved clay overlain by 3 ft of clayey sand. The bulk moisture content and plasticity index first increase and then decrease with depth (below El. 95),

but such changes are relatively small indicating a fairly uniform deposit of varved clay. The bulk moisture content is also larger than the bulk liquid limit, indicating that the soil may be somewhat sensitive.

Figure 3.3 shows the general soil profile, stress history, and undrained shear strength from triaxial UUC and field vane tests. The stress history shows that the soil is overconsolidated at all depths. It is presumed that the increasing values of the maximum past pressure ($\bar{\sigma}_{vm}$) above El. 110 is due to prior disiccation (Ladd, 1975). Below El. 110, the soil is slightly overconsolidated. This resulted from a lowering of the ground water table and/or erosion of the overburden. The Geonor field vane strengths below the crust are almost constant with depth at 700 $\pm$ 100 psf; the sensitivity from the field vane test is about 7 above El. 100, and 5 to 6 below that depth. Triaxial UUC tests yield strengths fairly close to those measured with the field vane in the upper portion of the clay, whereas below El. 110 they are significantly lower. Table 3.4 shows the details of the testing conditions and testing program.

3.2.3 Northampton Varved Clay

Four borings, B1 through B4, were made at several locations near the intersection of I91 and Route 9 in Northampton, Massachusetts (Ladd and Wissa, 1970). Three in. diameter undisturbed samples were obtained from borings B1 and B2 in

1967 and 5 in. diameter undisturbed Osterberg type samples were obtained from Borings B3 and B4 in 1970. Field vane tests using the Geonor apparatus were run adjacent to Boring B3 and B4. Soil samples from Borings B3 and B4 were used for consolidated - undrained strength tests. See Ladd and Wissa (1970) for further details.

Figure 3.4 shows the soil profile, Atterberg limits, and virgin compression ratios from the borings. The soil profile generally consists of a thick deposit of varved clay which is covered by medium sand. The clay is grayish or gray brown in color and has typical varve thicknesses of one-half to one inch. The Atterberg limits of the clay and silt portions of the Northampton varved clay show no significant difference from those of the Amherst and East Windsor varved clays. Values of the virgin compression ratio range from 0.2 to 0.4. The upper values are larger than those from Amherst and East Windsor. The varved clay deposit at Northampton is not as uniform as that at Amherst.

Figure 3.5 shows stress history and undrained shear strength data from triaxial UUC, unconfined compression, and field vane tests. The stress history from the "good quality" undisturbed sample at Boring B3 and B4 shows that soil is overconsolidated at all depths. The stress history from Borings B1 and B2 is not as reliable because the samples were more disturbed. The results from the Geonor field vane tests indicate that the clay is medium to stiff and that the strength

increases with depth from 1000 psf at El. 90 to 1700 psf at El. 50. Undrained strength data from unconfined compression and triaxial UUC tests on vertical samples are significantly lower, exhibit appreciable scatter and show no consistent trend with depth. This occurred in spite of the fact that the tests were run on samples from 5 in. diameter fixed piston samples. Table 3.5 shows the testing program of Northampton varved clay. Other experimental results were also reported in Lacasse et al. (1972).

3.3.4 Hackensack Valley Varved Clay

Five in. diameter Osterberg type samples of varved clay were obtained from the Hackensack Valley at Sacaucus, New Jersey in cooperation with the Port Authority of New York and New Jersey. The boring was located immediately to the southwest of the Palisades in the township of Secaucus (see Saxena et al., 1974). Figure 3.6 shows the general soil profile, Atterberg limits, the stress history, and the values of the virgin compression ratio. The soil profile consists of fill, peat, sand, and silty clay overlying about 35 ft of varved clay.

The varved clay is red-brown in color. The varve thicknesses vary widely. They are about 3/8 in. at shallow depths and about 1 in. below El. -40. Though the bulk natural moisture content is somewhat lower, the bulk Atterberg limits and the values of the virgin compression ratio are generally

similar to those for the Connecticut Valley varved clays, at least for samples between El. -35 to 50 ft which were used for the consolidated-undrained strength tests.

The stress history at this site shows that the clay is overconsolidated at all depths. The causes of the overconsolidation are mostly due to desiccation for soil above El. -40 and to the lowering of the ground water table for soil below El. -40.

Table 3.6 shows the experimental program for the Hackensack Valley varved clay.

3.4 SUMMARY OF SOIL PROPERTIES

The following is the summary of the soil properties of Northampton, Amherst, East Windsor, and Hackensack Valley varved clays.

1. The soil profiles of Amherst, Northampton and Hackensack Valley have thick deposits of varved clay, although the varve thicknesses varied.
2. At Amherst, the deposit was quite uniform, as were the thicknesses of the clay and silt layers. On the other hand, the deposits at Northampton and Hackensack Valley were more variable.
3. The varve thicknesses of samples from Amherst, Northampton, and East Windsor used in the testing program (Tables 3.3 to 3.6) are different.

Northampton and Amherst samples have about the same varve thicknesses. East Windsor samples have the smallest varve thickness. The varve thickness of Hackensack Valley samples are between those from Amherst and East Windsor.

4. Despite the difference in varve thicknesses, the samples used in the undrained strength tests from East Windsor, Amherst, Northampton, and Hackensack Valley have essnetially the same bulk Atterberg limits and virgin recompression ratio (see the summary in Table 3.7). The Atterberg limits of the individual clay and silt portions of the Connecticut Valley varved clays (no data from Hackensack Valley varved clay) are also essentially the same. However, the natural moisture contents (listed below) of the Hackensack Valley varved clay are significantly different from those of the Connecticut Valley varved clays.

Soil Location	W_N (%)	$\bar{\sigma}_{vm}$ (kg/cm^2)
Amherst	63±4	2.0±0.5
Northampton	59±3	3.0±0.5
East Windsor	57±2	2.4±0.2
Hackensack Valley	48±2	2.5±0.1

3.5 GENERAL TESTING PROCEDURE

All tests were performed at MIT using the general test

procedures described below.

3.5.1 Consolidated Undrained Triaxial Tests

Triaxial Compression Test

All triaxial compression tests were performed with Geo-measurement cells. Only bottom drainage was allowed during consolidation. One of the two drainage lines leading from the bottom pedestal at the base of the triaxial cell was attached to a burette for measuring the volume change. An electrical pressure transducer was attached to the other drainage line for the measurement of pore pressure during shear. The confining pressure, applied to the triaxial cell water, and the back pressure were obtained from adjustable mercury columns (Bishop and Henkel, 1962). After samples were cut to the desired varved inclination, they were tested according to the standard triaxial testing procedure presented in Ladd and Varallyay (1965).

The experimental program includes both isotropic and K_o consolidation. For K_o consolidation, the axial consolidation stress was applied to the samples with small load increments by using dead weight. The value of K_c, the ratio of the effective horizontal consolidation stress ($\bar{\sigma}_{hc}$) to the effective vertical consolidation stress ($\bar{\sigma}_{vc}$), was adjusted with each load increment to obtain essentially one-dimensional strain. Since an approximate relationship between K_o and OCR was known in advance, K_o consolidation was closely followed in most load steps. Vertical filter strips are used for

accelerating the drainage during consolidation and equalizing the pore pressure during shear. The samples consolidated via the SHANSEP approach were loaded in steps to a stress greater than about 1.2 to 2 times the in situ maximum past pressure ($\bar{\sigma}_{vm}$). For testing samples at an OCR greater than one, the samples were unloaded to the desired OCR. The RECOMPRESSION samples were loaded in steps to the vertical stress at the desired OCR ($\frac{\bar{\sigma}_{vm}}{\bar{\sigma}_{vc}}$). At the final consolidation stress, two days were used for normally consolidated SHANSEP samples to allow a reasonable amount of secondary compression, and one day was used for OC SHANSEP and RECOMPRESSION samples. A back pressure of 2.0 kg/cm^2 was used for all tests.

All but two of the overconsolidated SHANSEP $\overline{CK_oU}$ test samples were consolidated to a maximum vertical effective stress of 3.6 kg/cm^2 before they were unloaded. The vertical effective stress was increased in the following steps: 0.5 kg/cm^2 (isotropic), 0.95, 1.6, 2.0, 2.3, 2.75, 3.23 and 3.6. The vertical effective stress was then rebounded to 1.8 kg/cm^2 in two steps and 0.9 kg/cm^2 in one step to obtain samples which had OCR's of 2 and 4, respectively. The other two OC $\overline{CK_oUC}$ test samples (OCR of four) were consolidated to 4.0 kg/cm^2 before being unloaded to 1.0 kg/cm^2.

All NC SHANSEP samples used for $\overline{CIU}$ ($\overline{CIUC}$ and $\overline{CIUE}$) tests were consolidated to a maximum consolidation pressure of 4.0 kg/cm^2. The load steps were 0.3, 0.75, 1.5, 2.5 and 4.0 kg/cm^2.

RECOMPRESSION samples from East Windsor were isotropically consolidated to the overburden pressure of 0.6 kg/cm^2 in two steps to obtain samples with an OCR of four. Samples with an OCR of two were anisotropically consolidated to a vertical effective stress of 1.2 kg/cm^2 in three steps.

Wykeham-Ferrance loading frames were used to produce controlled rates of strain during shear. A strain rate of 0.7%/hr was used for SHANSEP samples, and 0.35%/hr for RECOMPRESSION samples. The samples were loaded (i.e., axial stress increased) in compression tests and a proving ring was used for measuring the load. Readings of elapsed time, pore pressure, sample length, and the vertical load from the proving ring were obtained from a data aquisition system with less frequent visual checks.

The piston friction correction was obtained by measuring the friction force required to move the piston through water in a triaxial cell at the strain rate and cell pressure corresponding to the testing condition. The correction was made by subtracting this friction force from the total measured axial force. A filter paper correction was applied in $\overline{CK_oUC}$ tests according to the procedure outlined in Ladd and Varallyay (1965).

Triaxial Extension Test

The testing procedure for TE test is different from that for TC test in two aspects, as follows:

1. During sample set-up: the filter strip paper is wrapped spirally around the sample instead of being placed vertically as in the TC test. Thus, there is no filter paper correction.

2. During shear: the samples were unloaded (i.e., axial stress decreased) in TE tests. Also, suitable methods for connecting the top cap to the loading piston and the loading piston to the proving ring are needed.

Figure 3.7 shows a schematic diagram of the essential elements in the TE test, including the connections between the loading piston and the top cap, and between the loading piston and the proving ring. Two piston pins (1) are inserted into the chamber (2) in the top cap (3) for connecting the loading piston (4) to the top cap. After lowering the loading piston until it touches the ball bearing (5), the connection between the top cap and the piston was then made by rotating the pin across the slot. As there is a small clearance between these pins and the top cap, this arrangement will not cause a torsional force to be applied to the top of the sample.

The proving ring and the loading piston were connected by two pins (6) and two cross bars (7). These two pins were first put through the holes located in the piston block (8) and the proving ring head before inserting the cross bars. Because the diameter of these pins is smaller than the holes in the cross bars, proving ring head, and the piston block, the connection is very flexible. Consequently, any small deviation in alignment of the proving ring, loading piston

and top cap is not very important.

For $\overline{CK_oUE}$ and $\overline{CIUE}$ tests, the load steps, the back pressure, the rate of shearing, and the time allowing for final load increment during consolidation were identical to those used for TC tests.

3.5.2 Testing Problems with Consolidated Undrained Triaxial Tests

Due to the layered nature of varved clays, disturbance due to trimming and setting up the sample, expecially with TE tests, can severly affect the undrained shear strength. Chapter 6 will discuss the effect of disturbance due to trimming which causes separation of silt and clay portions in horizontal and inclined samples. This section will present test data illustrating the importance at the details of testing on the undrained shear strength ($s_u=q_f$) when disturbance during setting up a TE test sample introduces a plane of weakness.

In the TE test, the process of connecting the loading piston to the top cap can introduce sample disturbance. Table 3.8 shows q_f data from NC and OC samples from Amherst, Northampton, and East Windsor, using two different testing procedures. Upon using the "new" testing procedure described in Section 3.5.1, $q_f/\bar{\sigma}_{vc}$ (Table 3.7 (a)) from OC (OCR=4) samples from Amherst and East Windsor are essentially identical (in fact, it will be shown in Chapter 4 that NSP from Amherst, Northampton, and East Windsor are identical). Thus, the larger

$q_f/\bar{\sigma}_{vc}$ (an increase of 30%) of NC East Windsor clay compared to those for NC Amherst and Northampton (using "previous" testing procedure) was due to the difference in the testing procedures. In the "previous" method, the tests (NC Amherst and Northampton samples) were performed in a cell where the loading piston was screwed into the top cap after the samples had been set up, and that probably caused a slight separation between one or more varves at the top of the specimen and hence introduced planes of weakness. This in turn probably caused the decrease in q_f for NC Amherst and Northampton samples. The fact that necking occurred at the top of these samples instead of at the middle of the sample as observed in the East Windsor specimen seems to support this hypothesis.

3.5.3 The Direct Simple Shear Test

All tests were performed with a Geonor Model 4 direct simple shear device. Description of the equipment and the step by step testing procedures for $\overline{CK_oU}$ tests are outlined in detail in Ladd and Edgers (1972) and Saxena et al. (1974).

In this test, a cylindrical sample 8 cm in diameter and approximately two cm high is confined in a wire-reinforced rubber membrane. This allows vertical deformations and horizontal shear movement with little or no change in diameter. During consolidation, there is very little lateral strain so that K_o consolidation is closely approximated. The consolidation process is the same as that of an ordinary consolidation test wherein a lever arm is used. The direct simple

shear device is capable of applying an instantaneous load which is achieved by locking the lever arm, putting on a load increment and then releasing the level arm. The SHANSEP samples were first loaded in steps to stresses greater than about 1.5 to 2. times the in situ maximum past pressure for the normally consolidated samples. For the overconsolidated samples, ths specimens were then unloaded to obtain the desired overconsolidation ratio. A period of two days was allowed at the last load increment for normally consolidated samples, and one day was allowed for overconsolidated samples.

Shearing was achieved by applying a horizontal shear force parallel to the top of the specimen. The sample height during shear was kept constant by a screw-controlled loading mechanism to cause shearing under constant volume conditions. This was done by regulating the vertical stress so that the dial gage reading for vertical deformation remains constant, after allowing for the compression or expansion of the porous stones and the top and bottom caps. The state of stress during shear can be considered to be close to a plane strain condition (Ladd and Edgers, 1972). The rate of shear for the tests was 5.5%/hr.

The special test procedures used for testing inclined samples are described in Section 5.3.4.

3.5.4 The Plane Strain Tests

This section provides a very brief description of the

MIT plane strain device and the testing procedure used with the Northampton varved clay. Details of the device and testing procedure are given in Bovee and Ladd (1970).

Two types of plane strain tests were performed, namely: K_o consolidated undrained plane strain compression ($\overline{CK_oU}$PSC) tests and plane strain extension ($\overline{CK_oU}$PSE) tests on vertical samples. The $\overline{CK_oU}$PSC test sample was sheared by increasing the vertical stress (i.e., $\bar{\sigma}_{1f} = \bar{\sigma}_{vf}$), while the $\overline{CK_oU}$PSE test sample was sheared by decreasing the vertical stress.

Figure 3.8 shows a simplified diagram of the specimen and the loading systems in the MIT plane strain device. The sample can be loaded both vertically and horizontally (the latter only used during consolidation for the strain controlled test). With the vertical loading system, during consolidation a vertical load is applied by a hanger placed with dead weights on the loading piston. During strained controlled shear, a load cell is placed above the hanger and a variable speed load frame is used to apply the vertical load.

For the horizontal loading system, a cell pressure (supplied by the self compensating mercury pot system) transmitted to the sample via a cell membrane is used for the purposes of:

(1) Maintaining K_o consolidation. Movable side platens located within water-filled chambers enclosed by the cell membranes and the side assembly plates are also used for restricting the horizontal deformation during consolidation.

(2) Applying σ_h increments to the sample during shear in stress controlled PS tests (not used for this study).

The pore pressures are measured from the transducer at the bottom of the specimen. The back end platen (Fig. 3.8) has a flush diaphram transducer for measuring the horizontal normal shear at that height.

Testing Procedure

The test specimen is 3.5 in by 3.5 in. by 1.4 in. After trimming, the entire specimen (after installing filter paper, overlapping for $\overline{CK_oU}$PSE test and continuous for $\overline{CK_oU}$PSC test) is enclosed in a 0.015 in thick membrane. A watertight seal is maintained by compression of latex membranes and an o-ring between the platens located at the top and bottom of the specimen and the piston block or the base plate.

Prior to each consolidation increment (dead weight for the vertical stress and cell pressure for the horizontal stress), the side platens are brought into contact with the end platens (Fig. 3.8) for restricting the horizontal deformation. During consolidation, adjustment of the cell pressure is required to insure that the volume change resulting from axial movement of the piston times the sample area equals the volume change recorded on the volume change device. At the later stages of consolidation in each load increment, the side platens are withdrawn. After final consolidation, the $\overline{CK_oU}$PSC test sample is vertically loaded by forcing the loading piston against

the load cell. The $\overline{CK_oU}$PSE sample is sheared by pulling up a loading piston and load cell.

The back pressure in the test was 2.0 kg/cm^2. The rate of shearing was 0.7%/hr. The corrections for filter paper (PSC only) and membrane resistance are given in Bovee and Ladd (1970).

SUMMARY OF THE EXPERIMENTAL PROGRAM

Type of Test	θ^{o} (1)	Soil (2) Location	OCR	Method (3) of Consolidation
UUC	0, 45, 90	E	4	--
$\overline{CIU}C$	0, 90	E	1,2,4,20	S
	45	E	1,4,20	S
	45	A	1	S
	0	H	1	S
	0,45, 90	E	4	R
$\overline{CK_oU}C$	0	E	1,2,4	S
	0	A	1,2,4	S
	0	H	1	S
	0	E	2,4	R
	0	A	1,6	R
$\overline{CIU}E$	0, 90	E	1,2,4	S
	0, 90	A	1	S
	90	E	4	R

Notes: (1) θ = angle between the varves and the in situ horizontal direction.
(2) E = East Windsor, A = Amherst, H = Hackensack.
(3) S = SHANSEP
R = RECOMPRESSION

Table 3.1(a)

SUMMARY OF THE EXPERIMENTAL PROGRAM

Type of Test	θ°	Soil[1] Location	OCR	Method[3] of Consolidation
$\overline{CK_oU}E$	0	E	1,4	S
		A	2,4	S
		E	2,4	R
		A	1.6	R
$\overline{CK_oU}DSS$	0	E	1,2,4	S
	0	A	1,3,6	S
	0	N	1,2,4,8	S
	0	H	1,2,4	S
	0	A	1.6	R
$\overline{CK_oU}DSS(C)$	45	E	1	S
$\overline{CK_oU}DSS(E)$	45	E	1	S
$\overline{CK_oU}PSC$	0	N	1	S
$\overline{CK_oU}PSE$	0	N	1	S
CDDS	0 0	N	1,1.8,3,5	S,R

Notes: (1) E = East Windsor; A = Amherst; N = Northampton; H = Hackensack

(2) S = SHANSEP; R = RECOMPRESSION

Table 3.1(b)

SUMMARY OF INDEX PROPERTIES AND ENGINEERING PROPERTIES OF EAST WINDSOR VARVED CLAY

Physical Properties:

Description	Bulk Sample	Clay Layer	Silt Layer
Natural Water Content (%)	55 - 60	63	45
Liquid Limit (%)	50.0	64.5	43.6
Plastic Limit (%)	28.9	31.4	30.5
Plasticity Index (%)	21.1	33.1	13.1
Specific Gravity	2.8	--	--
Total Density (pcf)	104 - 105	--	--

Engineering Properties:

$\bar{\sigma}_{vm} = 2.4 \pm 0.1$ kg/cm^2

CR = 0.20 ± 0.01

RR = 0.035 ± 0.003

q_f from UUC test = 0.37 TSF (vertical sample)

S_u from Torvane = 0.35 ± 0.02 TSF

Table 3.2

LABORATORY TESTING PROGRAM OF EAST WINDSOR VARVED CLAY

(θ = varve inclination)

Test No.	Type of Test	$\theta°$	OCR	Method of Consolidation
EUC-1 to EUC-3	UUC	0,45,90	4	--
EIC-1-1 to EIC-1-4	$\overline{CIUC}$	0	1,2,4,20	SHANSEP
EIC-2-1 to EIC-2-3		45	1,4,20	
EIC-3-1 to EIC-3-4		90	1,2,4,20	
EIE-1-1 to EIE-1-3	$\overline{CIUE}$	0	1,2,4	SHANSEP
EIE-3-1 to EIE-3-3		90	1,2,4	
EKC-1 to EKC-5	$\overline{CK_oUC}$	0	1,2,4	SHANSEP
EKE-1 to EKE-3	$\overline{CK_oUE}$	0	1,4	SHANSEP
EDS-1 to EDS-3	$\overline{CK_oUDSS}$	0	1,4,7.38	SHANSEP
ESDS-1	$\overline{CK_oUDSS(C)}$	45	1	SHANSEP
ESDS-2	$\overline{CK_oUDSS(E)}$	45	1	
ERIC-1-1	$\overline{CIUC}$	0	4	RECOMPRESSION
ERIC-2-1		45	4	
ERIC-3-1 to ERIC-3-2		90	2,4	
ERIE-1-1	$\overline{CIUE}$	0	4	
ERKC-1	$\overline{CK_oUC}$	0	2	
ERKE-1	$\overline{CK_oUE}$	0	2	

Table 3.3

LABORATORY TESTING PROGRAM OF AMHERST VARVED CLAY

Test No.	Type of Test	θ°	OCR	Method of Consolidation
ACK-1 to ACK-4(b)	$\overline{CK_oU}C$	0	1,2,4	SHANSEP
AKE-1 to AKE-2	$\overline{CK_oU}E$	0	2,4	SHANSEP
ADS-1 to ADS-4	$\overline{CK_oU}$DSS	0	1,3,6	SHANSEP
AIC-1-1 AIC-2-1 AIC-3-1	$\overline{CIU}C$	0 45 90	1	SHANSEP
AIE-1-1 AIE-3-1	$\overline{CIU}E$	0 90	1 1	SHANSEP
ARKC-1	$\overline{CK_oU}C$	0	1.6	RECOMPRESSION
ARDS-1	$\overline{CK_oU}$DSS	0	1.7	RECOMPRESSION

θ = varve inclination

Table 3.4

LABORATORY TESTING PROGRAM OF NORTHAMPTON VARVED CLAY

Test No.	Type of Test	θ°	OCR	Method of Consolidation
NKC-1 and NKC-2	$\overline{CK_oU}C$	0	1	SHANSEP
NDS-1 to NDS-8	$\overline{CK_oU}$DSS	0	1,2,4,8	SHANSEP
NCD-1 to NCD-6	CDSS	0	1,1.83,3	SHANSEP
NPC-1	$\overline{CK_oU}$PSC	0	1	SHANSEP
NPE-1 and NPE-2	$\overline{CK_oU}$PSE	0	1	SHANSEP

θ = varve inclination

Table 3.5

LABORATORY TESTING PROGRAM OF HACKENSACK VALLEY VARVED CLAY

Test No.	Type of Test	θ	OCR	Method of Consolidation
HKC-1	$\overline{CK_oUC}$	0	1	SHANSEP
HIC-1	$\overline{CIUC}$	0	1	SHANSEP
HDS-1 to HDS-4	$\overline{CK_oUDSS}$	0	1,1.98,4.12	SHANSEP

θ = varve inclination.

Table 3.6

SUMMARY OF THE INDEX PROPERTIES OF VARVED CLAY SAMPLES FROM AMHERST, EAST WINDSOR, NORTHAMPTON, AND HACKENSACK VALLEY

Typical Varved Thickness 1/2 to 1 inch.

	W_L	W_P	PI
Bulk	53 ± 4	26 ± 4	27 ± 8
Clay*	70 ± 5	35 ± 5	35 ± 10
Silt*	40 ± 4	26 ± 4	14 ± 8

CR = 0.2 to 0.3

*Average of available data where the strength tests were made.

Table 3.7

COMPARISON OF THE UNDRAINED SHEAR STRENGTHS FROM THE $\overline{CK_oUE}$ TESTS FROM DIFFERENT TESTING PROCEDURES

(a) Stregnth Data from New Testing Procedure:[3]

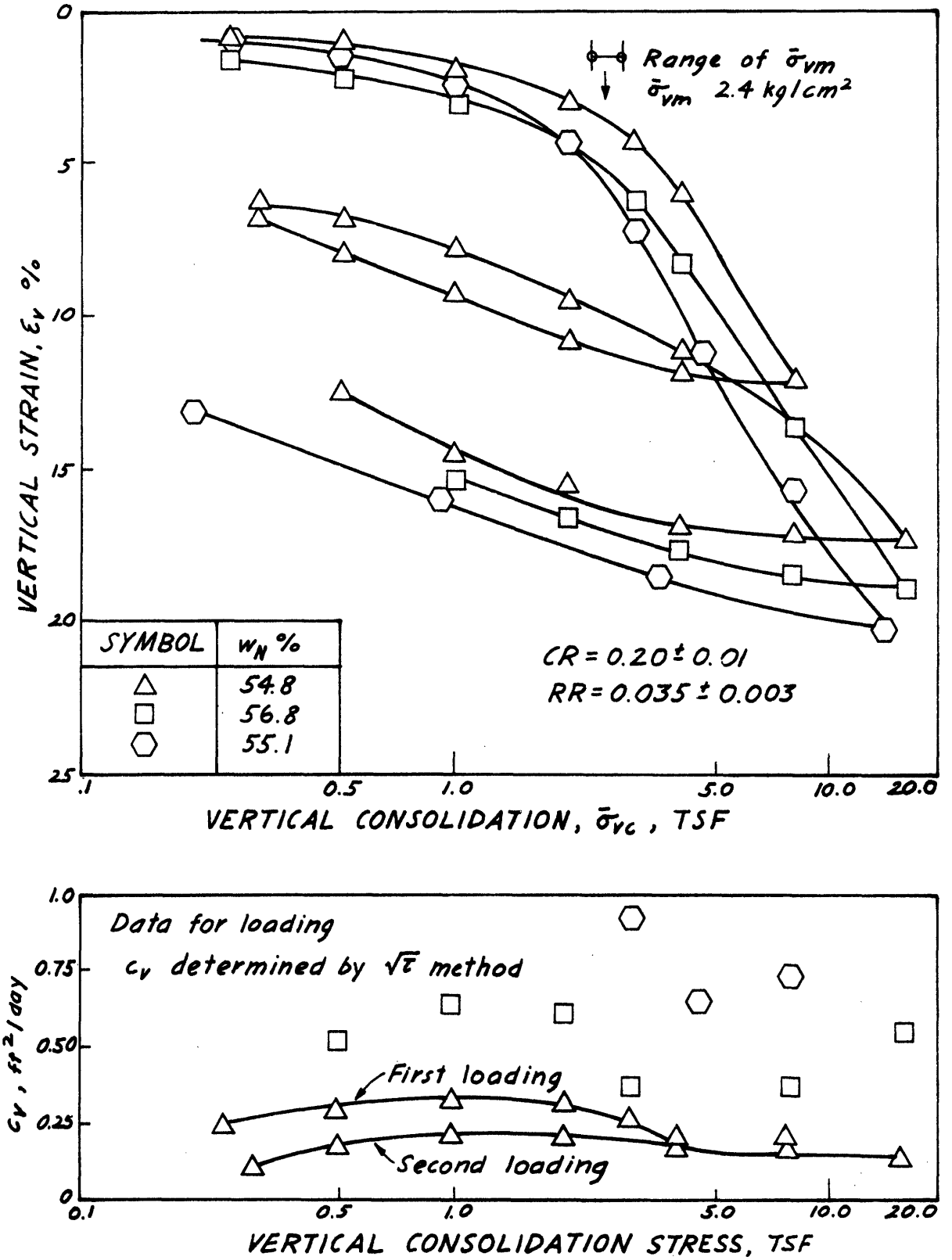
Soil Location	$\frac{\bar{\sigma}_{vm}^{(L)}}{\bar{\sigma}_{vm}}$	OCR	$q_f/\bar{\sigma}_{vc}$
Amherst	1.71	4	0.63
East Windsor	1.50	4	0.65

(b) Strength Data from Previous and New Testing Procedure[3] at OCR = 1.0

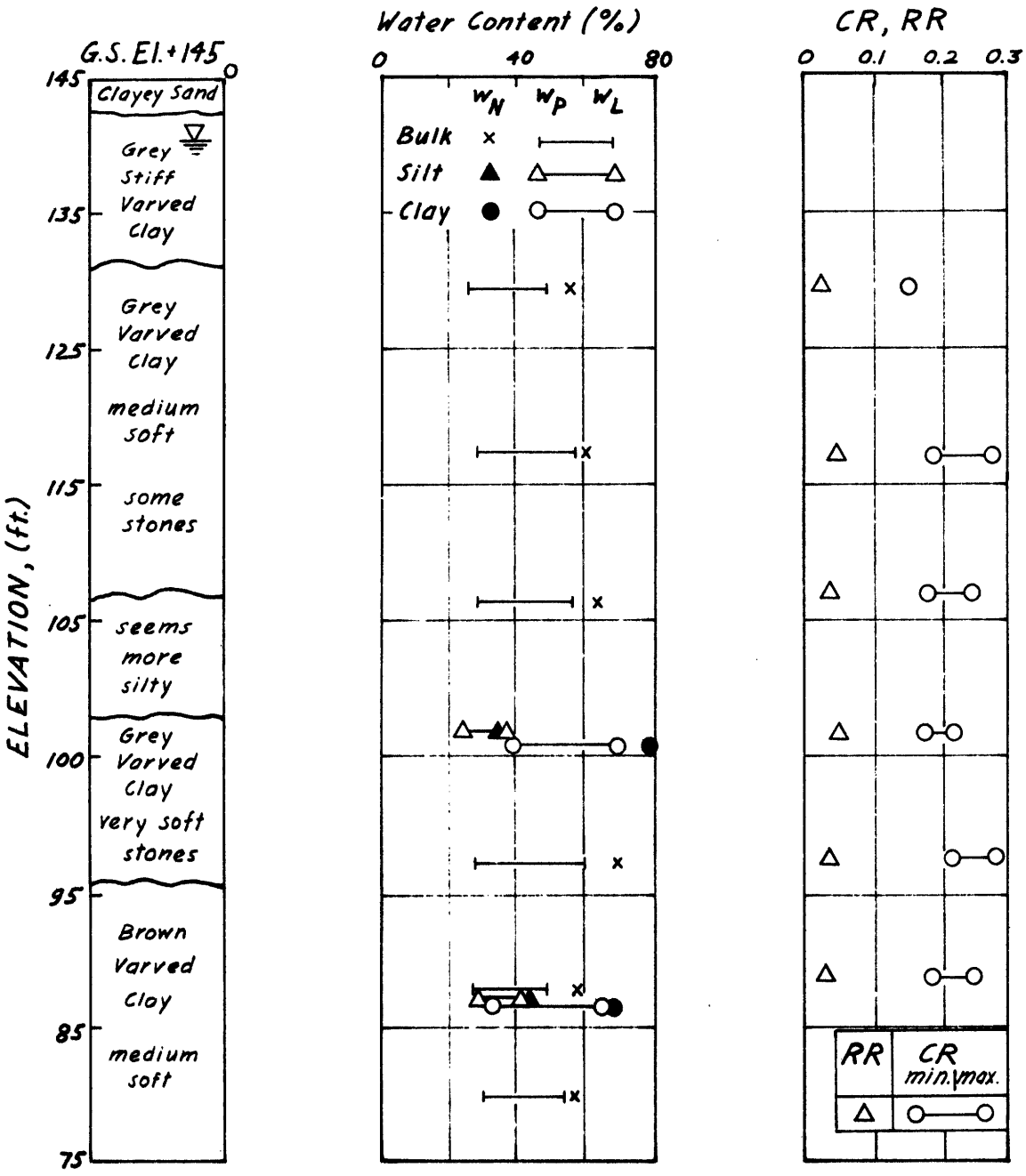
Soil Location	$\frac{\bar{\sigma}_{vm}^{(L)}}{\bar{\sigma}_{vm}}$	Testing Procedure	$q_f/\bar{\sigma}_{vc}$	Remark
Amherst	1.76	Previous	0.16[1]	From Ladd (1975)
Northampton	1.1 $\pm$ 0.10	Previous	0.15 $\pm$ 0.01[1]	Ladd (1975)
East Windsor	1.50	Now	0.21[2]	2 Tests

Note: (1) Failure at the top of the sample by necking.
(2) Failure at the middle of the sample by necking across the varve.
(3) Described in Section 3.5.2.

Table 3.8



RESULTS OF CONSOLIDATION TESTS
ON EAST WINDSOR VARVED CLAY FIGURE 3.1



SOIL PROFILE, ATTERBERG LIMITS, RECOMPRESSION RATIO, AND VIRGIN COMPRESSION RATIO OF AMHERST VARVED CLAY

FIGURE 3.2

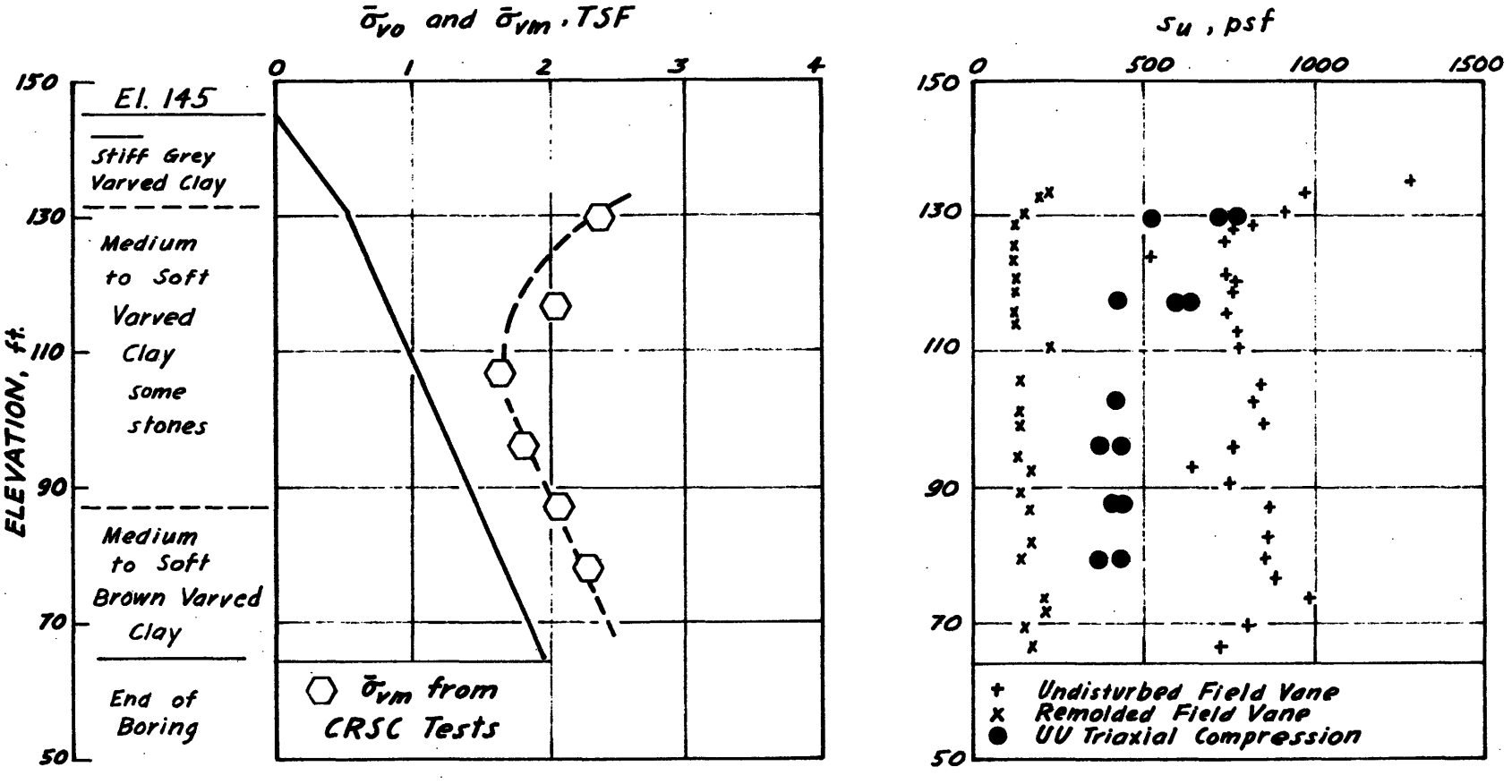
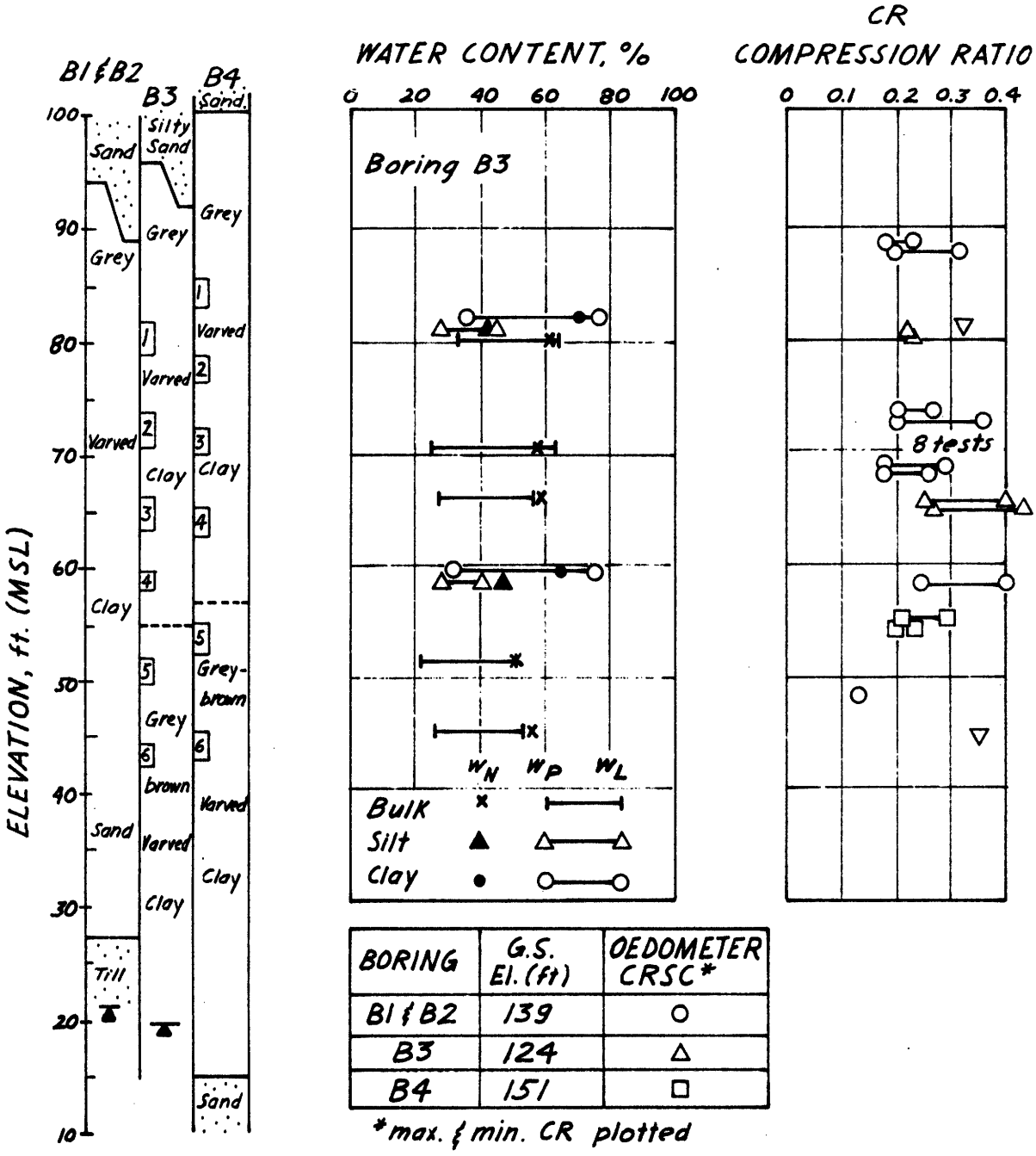


FIGURE 3.3

SOIL PROFILE, STRESS HISTORY, AND UNDRAINED STRENGTH DATA AT AMHERST, MA.

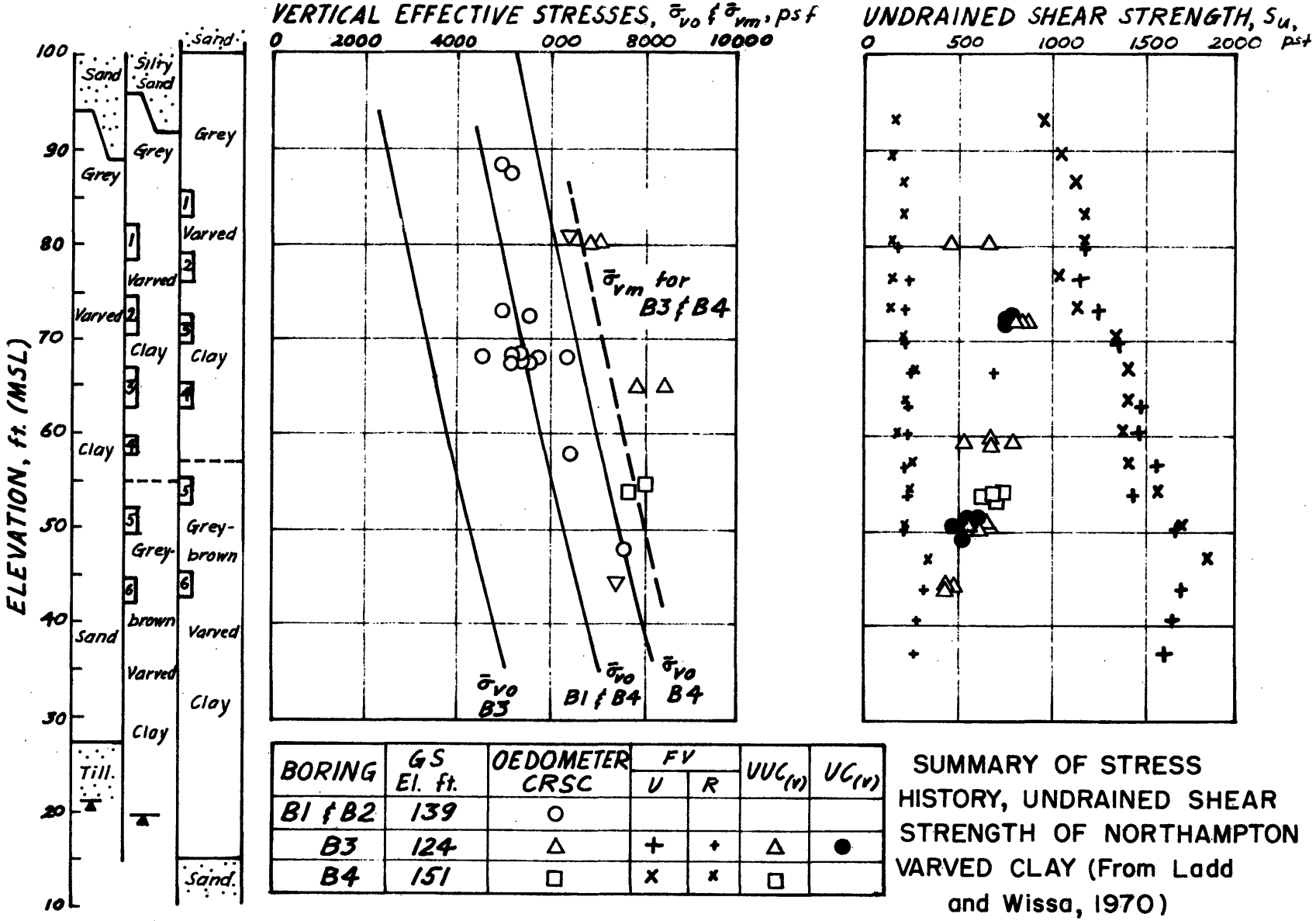


BORING	G.S. El. (ft)	OEDOMETER CRSC*
B1 & B2	139	○
B3	124	△
B4	151	□

*max. & min. CR plotted

SUMMARY OF ATTERBERG LIMITS AND COMPRESSIBILITY OF NORTHAMPTON VARVED CLAY (From Ladd & Wissa, 1970)

FIGURE 3.4



BORING	GS El. ft.	OEDOMETER CRSC	FV U	FV R	$UUC_{(v)}$	$UC_{(v)}$
B1 & B2	139	○				
B3	124	△	+	+	△	●
B4	151	□	×	×	□	

SUMMARY OF STRESS HISTORY, UNDRAINED SHEAR STRENGTH OF NORTHAMPTON VARVED CLAY (From Ladd and Wissa, 1970)

FIGURE 3.5

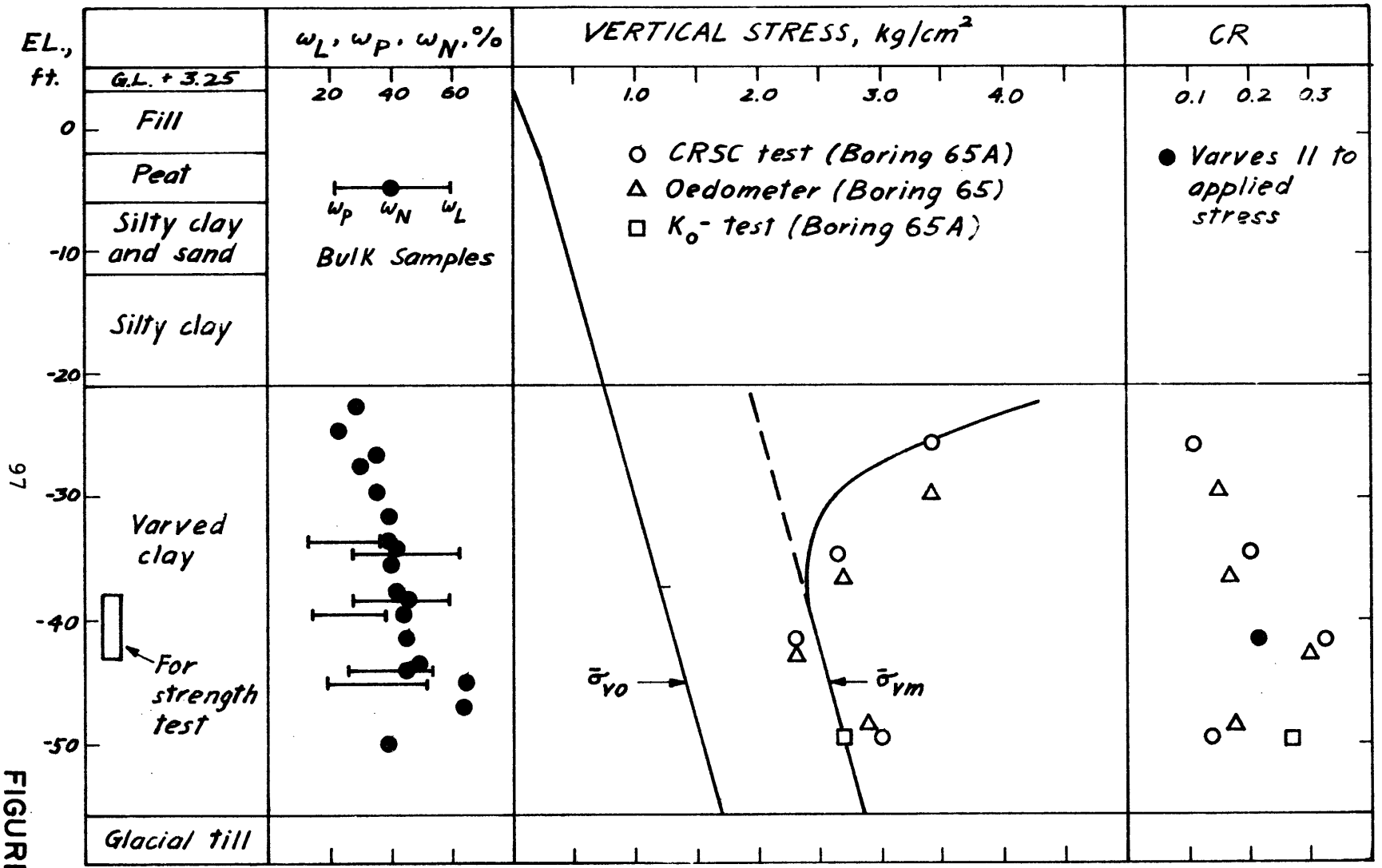
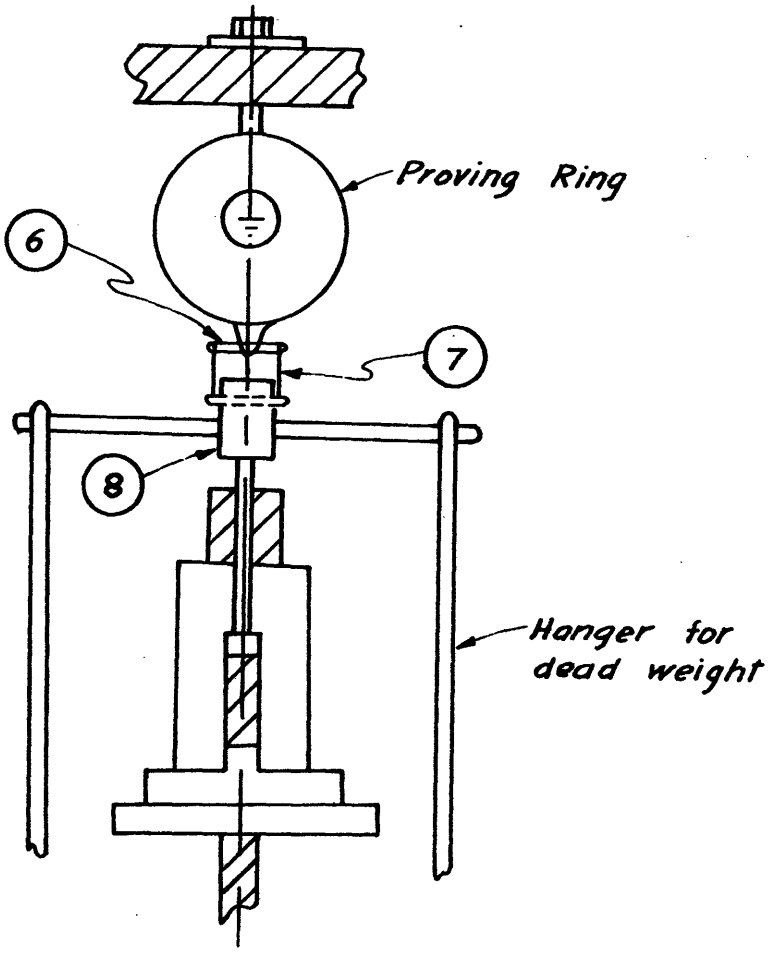
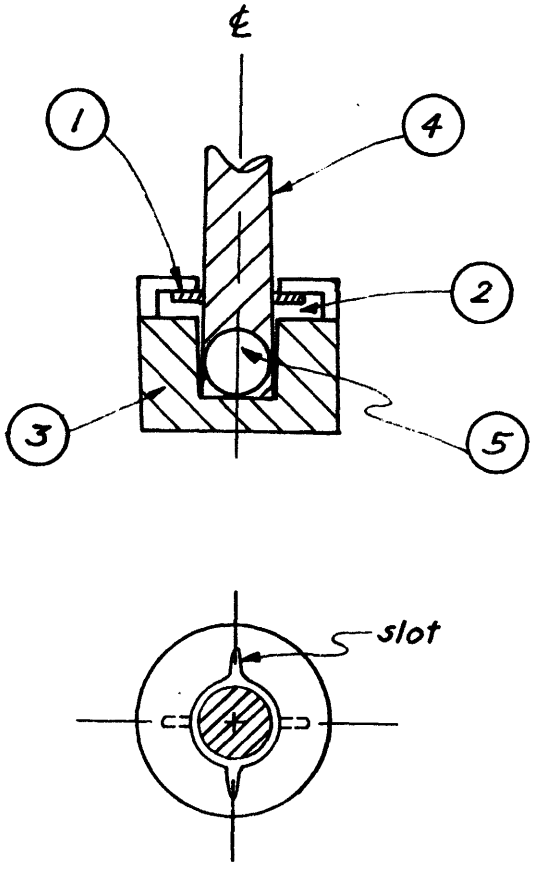


FIGURE 3.6

SUMMARY OF CONSOLIDATION DATA AT BORING 65 AND 65A, SECAUCUS, NEW JERSEY
(From Saxena, et al., 1974)

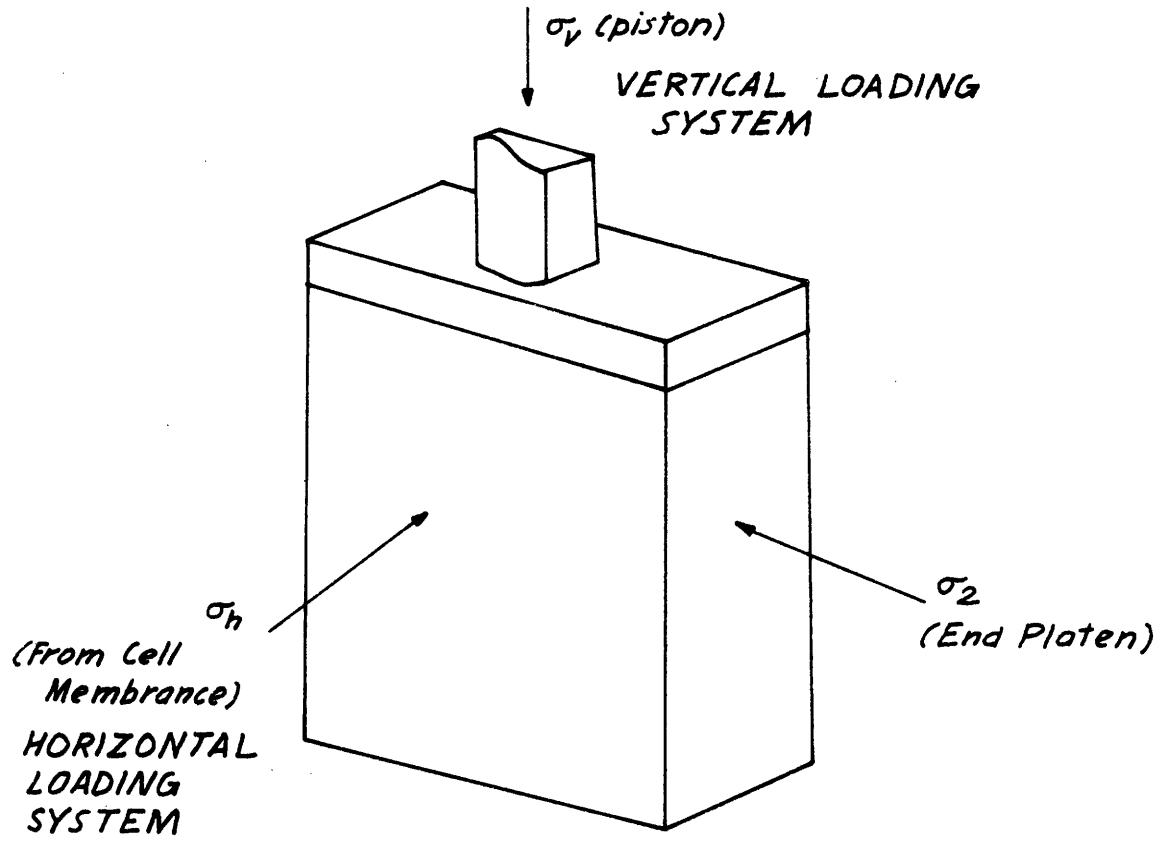


a) GENERAL SET UP



b) DETAIL OF TOP CAP

FIGURE 3.7 SCHEMATIC DIAGRAM OF ESSENTIAL ELEMENTS IN TRIAXIAL EXTENSION TEST



DIAGRAMATIC DRAWING OF PLANE STRAIN SAMPLE AND LOADING SYSTEMS

FIGURE 3.8

4. COMPARISON OF NORMALIZED BEHAVIOR OF DIFFERENT VARVED CLAYS

4.1 INTRODUCTION AND BACKGROUND

Normalized soil parameters (NSP) such as $q_f/\bar{\sigma}_{vc}$, A_f, $E_u/\bar{\sigma}_{vc}$ $\bar{\phi}$ and $\bar{c}/\bar{\sigma}_{vm}$, obtained using the SHANSEP approach (Ladd and Foott, 1974) have been used extensively for studying the stress-strain-strength properties of the Connecticut Valley varved clays (Ladd, 1975). The objectives of this chapter are: (1) to demonstrate experimentally whether or not varved clays, particularly those from Connecticut Valley, exhibit normalized behavior; and (2) to compare the NSP of varved clays from several locations.

The SHANSEP approach uses a maximum laboratory consolidation stress that is at least about 1.5 to 2.0 times the in situ maximum past pressure ($\bar{\sigma}_{vm}$). Overconsolidated samples are obtained by unloading from this maximum stress. Thus, in general, the SHANSEP samples which have the same laboratory OCR can have different magnitudes of the laboratory maximum past pressure [$\bar{\sigma}_{vm}(L)$], and thus, the maximum past pressure ratio, (MPPR), $\frac{\bar{\sigma}_{vm}(L)}{\bar{\sigma}_{vm}}$. At a given laboratory OCR, a difference in the MPPR of SHANSEP samples can result from differences in the magnitude of $\bar{\sigma}_{vm}(L)$ for samples with the same $\bar{\sigma}_{vm}$ and/or from differences of $\bar{\sigma}_{vm}$ of the samples used for testing.

If a soil exhibits normalized behavior, then at a given laboratory OCR and for the same stress system (both during consolidation and shear), the MPPR will not have a significant effect on NSP. Thus, plots of NSP versus OCR obtained for a given stress system should be independent of $\frac{\bar{\sigma}_{vm}(L)}{\bar{\sigma}_{vm}}$. Since the soil structure (Lambe, 1958) prior to shear can change with the magnitude of the laboratory applied consolidation stress, SHANSEP samples at a given OCR, but having different MPPR, may have different soil structure . For a soil that exhibits near-perfect normalized behavior, changes in soil structure with MPPR are minimal, and NSP from SHANSEP samples which are consolidated and sheared with a stress system similar to that of the in situ soil, should represent the in situ soil parameters.

The experiment program shown in Table 4.1 is divided into two parts according to the objectives of the investigation , as follows:

(1) Does varved clay from a given location (Amherst, Northampton, East Windsor and Hackensack Valley) exhibit normalized behavior? For this investigation, $\overline{CK_oUC}$ and $\overline{CK_oUDSS}$ tests were performed on NC and OC vertical samples at various MPPR (see upper part of Table 4.1) for vertical and inclined loading respectively. It should be noted, however, that the range in MPPR is relatively small.

(2) Comparison of SHANSEP NSP from soils at different locations -- This testing program, shown in the bottom part of Table 4.1, included several types of tests ranging from $\overline{CIUC}$

tests on NC samples with different inclinations to $\overline{CK_oU}$DSS tests on both NC and OC samples, obtained from two to four locations. Ideally, comparison of NSP from SHANSEP samples at different locations should be made with samples which have the same values of $\bar{\sigma}_{vm}$, $\bar{\sigma}_{vm}(L)$ and $\bar{\sigma}_{vc}$. However, it was not always possible to have such conditions, since some of the tests were performed prior to start of this thesis. Therefore, the comparison is made with samples having approximately the same $\frac{\bar{\sigma}_{vm}(L)}{\bar{\sigma}_{vm}}$ ratio, as shown in the lower part of Table 4.1. The normalized soil parameters (NSP) studied are $q_f/\bar{\sigma}_{vc}$, A_f, ε_f or γ_f, $E_u/\bar{\sigma}_{vc}$ (or $G/\bar{\sigma}_{vc}$), $\bar{\phi}$ and $\bar{c}/\bar{\sigma}_{vm}$. For the sake of completeness, NSP data from the Connecticut Valley varved clays are compared to those from New Liskeard and Matagami in Canada.

4.2 EFFECT OF SAMPLE STRESS HISTORY (MPPR)

Test data from $\overline{CK_oU}$C and $\overline{CK_oU}$DSS tests used to investigate the effect of sample stress history are summarized in Tables 4.2 through 4.4. Figures 4.1 and 4.2 show normalized stress-strain curves from $\overline{CK_oU}$C tests on OC (OCR=4) Amherst varved clay and from $\overline{CK_oU}$DSS tests on NC Northampton varved clay at various MPPR respectively. Test data in Tables 4.1 through 4.4 are samples which have a relatively small range in $\bar{\sigma}_{vm}$ ($\bar{\sigma}_{vm}$ ranges from 2.0 to 4.0 kg/cm^2) and MPPR ($\frac{\bar{\sigma}_{vm}(L)}{\bar{\sigma}_{vm}}$) ranges from 1.01 to 2.35). The following observations are made with regard to

the effect of MPPR on NSP ($q_f/\bar{\sigma}_{vc}$, A_f, $E_u/\bar{\sigma}_{vc}$, $\bar{\phi}$ and $\bar{c}/\bar{\sigma}_{vm}$) from four locations.

(1) The MPPR has an insignificant effect on the normalized undrained shear parameters ($q_f/\bar{\sigma}_{vc}$, A_f, and NC $\bar{\phi}$ (assuming $\bar{c}$ = 0)) from $\overline{CK_oU}C$ tests on NC samples from Amherst, Northampton, and East Windsor. This is also true with respect to $q_f/\bar{\sigma}_{vc}$ and A_f of OC samples from Amherst.

(2) The MPPR has an insignificant effect on $\frac{\tau_{h_{max}}}{\bar{\sigma}_{vc}}$ and $(\frac{\bar{\sigma}_v}{\bar{\sigma}_{vc}})$ at $\tau_{h_{max}}$ from $\overline{CK_oU}$DSS tests on NC samples from Amherst, Northampton, and Hackensack Valley. OC samples from Northampton also show the same trends.

(3) In general, $E_u/\bar{\sigma}_{vc}$ at $q_f/\Delta q_f$ = 1/2 from $\overline{CK_oU}C$ tests is practically unaffected by MPPR, considering the accuracy with which soil modulus can be measured. The variation of $E_u/\bar{\sigma}_{vc}$ is also less than the variation of $G/\bar{\sigma}_{vc}$ from DSS tests.

(4) Based on limited data from the same soil and from MPPR range for OC samples (OCR=4), the MPPR has a significant effect on $3G/\bar{\sigma}_{vc}$ at 50% and 20% stress levels from $\overline{CK_oU}$DSS tests (Fig. 4.3).

(5) For NC sample, Fig. 4.3 also shows the following:

a. Based on Amherst data (two tests), the increase in MPPR decreases $G/\bar{\sigma}_{vc}$ at 50% and 20% stress levels, while Northampton data (3 tests) show no consistent trend. But, collective data from East Windsor, Hackensack Valley, and Northampton varved

clay suggest a decrease in $G/\bar{\sigma}_{vc}$ with increasing MPPR:

b. The fact that the NC Amherst sample has a lower $\bar{\sigma}_{vm}$ and a higher $G/\bar{\sigma}_{vc}$ than at other sites with the same MPPR suggests that the magnitude of in situ $\bar{\sigma}_{vm}$ per se affects shear modulus (i.e., $G/\bar{\sigma}_{vm}$ decreases with increasing $\bar{\sigma}_{vm}$).

(6) Though limited data are available the results from Amherst and Hackensack Valley varved clay suggest that the MPPR has little effect on γ_f, though data from Northampton indicate an increase in γ_f with increasing MPPR.

Based on results from NC and OC (limited data) samples which have a small MPPR range and for two loading directions (vertical and inclined loading), the important findings are:

1. Varved clays from Amherst, Northampton, East Windsor and Hackensack Valley exhibit normalized behavior with respect to the undrained shear parameters $S_u^*/\bar{\sigma}_{vc}$, A_f, $\bar{\phi}$ (data only available for TC), and $\frac{\bar{\sigma}_v}{\bar{\sigma}_{vc}}$ from DSS tests;
2. The normalized undrained modulus for vertical loading is little affected by MPPR; but
3. The normalized undrained shear modulus of NC sample as measured from DSS tests most likely decreases with increasing values of both the in

*$S_u = q_f$ for TC and $S_u = \tau_{h_{max}}$ for DSS.

situ $\bar{\sigma}_{vm}$ and MPPR.

Based on the above, for any study of the effect of soil location on $G/\bar{\sigma}_{vc}$, the samples should have the same values of both $\frac{\bar{\sigma}_{vm}(L)}{\bar{\sigma}_{vm}}$ and $\bar{\sigma}_{vm}$. However, this requirement can not always be fulfilled since some of the sites have quite different values of in situ maximum past pressure, $\bar{\sigma}_{vm}$.

4.3 EFFECT OF SOIL LOCATION

The results presented in Section 4.3 are from samples at four locations which have approximately the same MPPR, and $\bar{\sigma}_{vm}$. $\overline{CIUC}$ tests on samples with different inclinations, and $\overline{CIU}E$, $\overline{CK_oU}C$, $\overline{CK_oU}DSS$ and $\overline{CK_oU}E$ tests on vertical samples are used for this study.

4.3.1 Results from $\overline{CIU}$ Tests

A series of $\overline{CIUC}$ tests on samples with different varve inclinations was performed on NC Amherst and East Windsor samples. In addition, results from one vertical NC sample from Hackensack Valley are available. These samples had approximately the same magnitude of $\bar{\sigma}_{vm}$, ranging from 2.1 to 2.6 kg/cm^2. The MPPR ranged from 1.5 to 1.9 (Table 4.5). Thus, comparison of the normalized undrained modulus data for all directions of loading can be made.

The results from the $\overline{CIUC}$ tests are shown in Table 4.5 and Figs. 4.4 to 4.7. These data show that the soil location:

1. Does not have a significant effect on $q/\bar{\sigma}_{vc}$ either at the maximum shear stress or at the

maximum obliquity condition obtained from θ=0, 45 and 90° tests.

2. Shows no apparent effect on A_f and ε_f (both at the q_f and $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ conditions) for all three loading directions, though the Hackensack Valley $\overline{CIUC}$ (θ=0) test indicates a lower ε_f at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$. [Such a difference may result from difficulty in picking the strain at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ since the test had an almost horizontal $(\bar{\sigma}_v/\bar{\sigma}_h)$ versus ε curve (see Fig. 4.5)]. The test on East Windsor was stopped at ε=13.5% before reaching $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ conditions.
3. Has no effect on $\bar{\phi}$ at q_f (same as $\bar{\phi}$ at $(\bar{\sigma}_1/\bar{\sigma}_3)$) from $\overline{CIUC}$ (θ=90) tests, though $\bar{\phi}$ at q_f from $\overline{CIUC}$ θ = 45 and θ = 0 tests are affected. A significant effect of soil location on $\bar{\phi}$ at q_f is only observed from $\overline{CIUC}$ (θ = 0) tests. The reason is not clear.
4. Shows no practical effect on normalized stress-strain curves from tests with θ = 0, 45 and 90° (Figs. 4.4 to 4.7). Variations in $E_u/\bar{\sigma}_{vc}$ at ε =0.5% are not considered significant when considering the difficulty in measuring E_u (Table 4.5).

It should be noted that the variation in q_f with loading direction from the $\overline{CIUC}$ tests is much smaller than that from UUC tests (see Chapter 2). The explanation of such a difference will be presented in Chapter 6. It was also observed

that the mode of failure varied with sample inclination. The vertical samples failed by bulging. The inclined samples had failure planes within clay layers and the horizontal samples had failure planes across the varves.

Two $\overline{CIUE}$ (θ=0) tests were performed on normally consolidated samples from Amherst and East Windsor. These samples have approximately the same $\bar{\sigma}_{vm}$, $\bar{\sigma}_{vm}(L)$, and thus $\frac{\bar{\sigma}_{vm}(L)}{\bar{\sigma}_{vm}}$ ratio. The normalized stress-strain-obliquity curves from these two tests are shown in Figs. 4.4 and 4.5 to be essentially identical.

In summary, the results from $\overline{CIU}$ tests on normally consolidated samples indicate that the Amherst and East Windsor varved clays have essentially identical undrained stress-strain strength parameters for vertical, inclined and horizontal loading. Section 4.3.2 will provide additional data from $\overline{CK_oU}$ tests which show generally identical undrained stress-strain strength properties from Amherst, East Windsor, Northampton and Hackensack Valley.

4.3.2 Results from $\overline{CK_oU}$ Tests

Test results from $\overline{CK_oU}$ triaxial compression, extension, and direct simple shear tests on NC and some OC vertical samples from the four different locations, are presented in this section.

4.3.2(a) Data from $\overline{CK_oUC}$ Tests

Table 4.6 summarizes the test results from $\overline{CK_oUC}$ tests at the maximum shear stress condition. Data are shown for

four locations at OCR=1 and for two locations with OCR's equal to 2 and 4. These results were obtained from samples which have approximately the same $\frac{\bar{\sigma}_{vm}(L)}{\bar{\sigma}_{vm}}$ ratio so that a comparison of the normalized undrained modulus could be made. A comparison of NSP can not be made at maximum obliquity condition for all tests, because some tests were stopped before reaching this condition.

Test data in Table 4.6 and Figs. 4.8 to 4.14 show that, for both NC and OC samples, the soil location has practically no effect on:

1. q_f, A_f, ε_f and $E_u/\bar{\sigma}_{vc}$ at $\Delta q/\Delta q = 0.5$ (Table 4.6);
2. Normalized stress-strain curves (Figs. 4.8 and 4.11, normalized stress paths (Fig. 4.9 for NC samples), the failure envelope at q_f for normally consolidated samples (Fig. 4.10), and the normalized effective stress envelope (NESE), a plot of $q_f/\bar{\sigma}_{vm}$ versus $\bar{p}_f/\bar{\sigma}_{vm}$, in Fig. 4.13; and
3. Unique plots of A_f, ε_f, and $q_f/\bar{\sigma}_{vc}$ versus OCR (Figs. 4.12 and 4.14).

The test data in Table 4.6 also show larger scatter in A_f, ε_f, $\bar{\phi}$ and $E_u/\bar{\sigma}_{vc}$ than in $q_f/\bar{\sigma}_{vc}$.

4.3.2(b) Data from $\overline{CK_oU}$DSS Tests

Detailed NC and OC (OCR=4) DSS test data having about the same MPPR are compared to show the effect of soil location. Test results from samples having other OCR values are also used for comparison of $\tau_{h_{max}}/\bar{\sigma}_{vc}$ versus OCR and

$\tau_{h_{max}}/\bar{\sigma}_{vm}$ versus $\bar{\sigma}_v/\bar{\sigma}_{vm}$.

Table 4.7 presents a summary of results from $\overline{CK_oU}$DSS tests on NC and OC (OCR=4) samples which have about the same MPPR. The comparison is made at the maximum horizontal shear stress ($\tau_{h_{max}}$) condition. The soil parameters are $\tau_h/\bar{\sigma}_{vc}$, γ_f, $\bar{\sigma}_v/\bar{\sigma}_{vc}$. The results in Table 4.7 and Fig. 4.3 and Figs. 4.14 to 4.16 shows:

1. For both NC and OC samples, the soil location has practically no effect on: (a) $\tau_{h_{max}}/\bar{\sigma}_{vc}$ and $\bar{\sigma}_v/\bar{\sigma}_{vc}$ (Table 4.7); (b) the "unique" plot of $\tau_{h_{max}}/\bar{\sigma}_{vc}$ versus OCR (Fig. 4.14); (c) the normalized-stress strain curves ($\tau_h/\bar{\sigma}_{vc}$ versus γ) (Fig. 4.15); and (d) the NESE, a plot of $\tau_{h_{max}}/\bar{\sigma}_{vm}$ versus $\bar{\sigma}_v/\bar{\sigma}_{vm}$ (Fig. 4.16);
2. For both NC and OC samples, $G/\bar{\sigma}_{vc}$ at 50% stress level from $\overline{CK_oU}$DSS tests are mostly affected by the MPPR rather than the soil location (Fig. 4.3), though the data from Amherst show some deviation (probably because of a significant difference in the magnitudes of $\bar{\sigma}_{vm}$),

4.3.2(c) Data from $\overline{CK_oU}$E Tests

$\overline{CK_oU}$E test data are only available for Amherst and East Windsor. From the results shown in Figs. 4.13, 4.14, and 4.18, it is seen that the location had little effect on: (1) plots of $q_f/\bar{\sigma}_{vc}$, A_f, and ε_f versus OCR; and (2) the NESE at q_f. The difference in the normalized stress-strain curves

in Fig. 4.17 might be the result of differences in the values of $\bar{\sigma}_{vm}$ and MPPR rather than in soil location. The higher modulus of the Amherst sample compared to that of the East Windsor sample is due to the lower $\bar{\sigma}_{vm}$ at Amherst (same effect as observed with $G/\bar{\sigma}_{vc}$ data from DSS tests), though the MPPR is higher. Based on NC $\overline{CIUE}$ test data (Figs. 4.4 and 4.5) from Amherst and East Windsor samples which had smaller differences in MPPR compared to the $\overline{CK_oUE}$ tests, one would expect the same normalized stress-strain curves for these two clays if they had similar values of $\bar{\sigma}_{vm}$.

The normalized effective stress envelope (NESE) at q_f (Fig. 4.13) for vertical and horizontal loading are different. For horizontal loading, it is interesting to note that the results from NC $\overline{CIUE}$ and $\overline{CK_oUE}$ test samples and OC $\overline{CK_oUE}$ test samples plot on the same NESE. Further discussion of this point will be presented in Chapter 5.

In summary, $\overline{CK_oUC}$, $\overline{CK_oUDSS}$, and $\overline{CK_oUE}$ test data show that the samples from Amherst, Northampton, East Windsor, and Hackensack Valley (limited data) have very similar NSP ($q_f/\bar{\sigma}_{vc}$, A_f, ε_f, $E_u/\bar{\sigma}_{vc}$, $\bar{\phi}$ and $\bar{c}/\bar{\sigma}_{vc}$). The differences in $G/\bar{\sigma}_{vc}$ values from DSS tests and in $E_u/\bar{\sigma}_{vc}$ from $\overline{CK_oUE}$ tests are thought to be due to the effect of variation in stress history rather than the soil location per se.

4.4 DISCUSSION

The experimental evidence in Section 4.2 demonstrated for the range of MPPR under study that the varved clays from Amherst, East Windsor, Northampton, and Hackensack Valley exhibit normalized behavior with respect to undrained shear parameters ($q_f/\bar{\sigma}_{vc}$, A_f, $\bar{\phi}$ and $\bar{c}/\bar{\sigma}_{vm}$), and fairly consistent undrained modulus data for vertical loading, i.e., from $\overline{CK_oU}C$ tests. However, shear modulus data from DSS tests show a trend of decreasing $3G/\bar{\sigma}_{vc}$ with increasing MPPR and/or in situ $\bar{\sigma}_{vm}$.

Section 4.3 presented experimental evidence which indicated that: the varved clays from Northampton, Amherst, and East Windsor, in the Connecticut Valley, have the same NSP. Furthermore, limited data from the Hackensack Valley varved clay suggest that this deposit has essentially the same normalized properties as those obtained from the Connecticut Valley.

The above results suggest that varved clays which have similar bulk index properties and composition, though not necessarily having the same varve thickness, should have the same NSP. Thus, if the nature of the varves in other deposits varies due to:

1. Large differences in relative positions of "silt" and "clay" layers;
2. Large differences in Atterberg limits of the silt and/or clay layers;
3. Variation in mineral composition of varves, especially the presence of cementing agents

such as carbonates;

then, one might expect their NSP to differ significantly from those of the Connecticut Valley varved clay.

An example of the possible effect of the difference in the nature of varves on NSP is provided by data from the Matagami varved clay located in Canada near James Bay (NGI, 1972). The two layers of this varve deposit are composed of a dark grey layer of soft fissured clay and a light gray layer of soft silty clay, generally with the thicknesses of 3/8 - 1/2 in. and less than 1/5 in. respectively. Compositional analyses show the presence of large amounts of carbonates in both layers. Thus, this soil is most likely naturally cemented. The soil profile shows that the thickness of the silt layer increases with depth. Table 4.8 shows the bulk Atterberg limits and undrained shear strength data from samples at two depths. These $S_u/\bar{\sigma}_{vo}$ values are higher than those for the Connecticut Valley (compared S_u at the same OCR [see Fig. 4.14]). The increase is thought be be largely due to the difference in the nature of the varves, though the method of consolidation (SHANSEP method for Connecticut Valley varved clay versus RECOMPRESSION ($\bar{\sigma}_{vc} = \bar{\sigma}_{vo}$) method for Matagami varved clay) was also different (see Chapter 6 for general effect). Based on FV data (Fig. 4.19) where the method of consolidation is not a factor, $S_u(FV)/\bar{\sigma}_{vo}$ for the Matagami varved clay is also significantly higher than that from the Connecticut Valley. As a result, the difference in laboratory strengths are probably due to inherent differences

in the nature of the varves, and especially the carbonate cementation.

Basic differences in the nature of the varves are not always shown by variation in the bulk Atterberg limits. The best illustration of this fact can be seen from a comparison of the NSP of the varved clays from the Connecticut Valley and New Liskeard in Canada. A description of the general soil properties and undrained shear strength (S_u) data of the latter soil are presented in Lacasse and Ladd (1973). Briefly, the varved clay from New Liskeard has about 1/2 in. thick clay layers and 3/8 in. thick silt layers. The natural water content of the silt portion varies between 24 and 30%, and varies between 60 and 80% for the clay portion. On Casagrande's plasticity chart, the CH "clay" layers plot above the A-line with $W_L = 70 \pm 10\%$ and $PI = 47 \pm 13\%$, and the CL and ML "silt" layers plot above and on the A-line with $W_L = 30 \pm 5\%$ and $PI = 10 \pm 6\%$. Thus, the silt and clay portions of New Liskeard varved clay have similar Atterberg limits to those from the Connecticut Valley (presented in Table 3.7), though composition analyses show large amounts of carbonates in both layers of New Liskeard varved clay (Quigley and Ogunbadejo, 1972). Furthermore, comparison of the bulk Atterberg limits of varved clays from Connecticut Valley and New Liskeard indicates that the former is only slightly more plastic. Figure 4.19 shows plots of $S_u/\bar{\sigma}_{vo}$ and $S_u/\bar{\sigma}_{vc}$ versus OCR from field vane and the $\overline{CK_oU}$DSS tests (RE-COMPRESSION consolidation for New Liskeard varved clay versus

SHANSEP method of consolidation for Connecticut Valley varved clay). The significantly higher strengths for the New Liskeard varved clay compared to those for the Connecticut Valley are probably the result of the presence of natural cementing agents, e.g., calcite and carbonate. (Both the Connecticut Valley and New Liskeard varved clays have mainly illite and chlorite clay minerals.)

These differences in the mineralogical composition, especially the presence of cementating agents, seem to be a very important factor affecting the NSP of varved clays. The plots of $S_u(FV)$ and $S_u(DSS)/\bar{\sigma}_{vo}$ in Fig. 4.19 for Matagami and New Liskeard, both having the same cementing agent (carbonates), are essentially identical, despite having very different bulk Atterberg limits.

Large variations in the relative portions of silt and clay layer may have a small influence on FV strengths. Limited data from the Lake Flushing varved clay in NY (Parsons, 1976) surprisingly suggest this. Despite a much smaller percentage of clay layers in the Lake Flushing varved clay compared with those from the Connecticut Valley and Hackensack Valley, $S_u(FV)/\bar{\sigma}_{vc}$ at an average OCR of four for the Lake Flushing varved clay is roughly equal to that for the Connecticut Valley (0.6 for Lake Flushing versus 0.74 for Connecticut Valley). The clay layers which have essentially the same Atterberg limits as those from Connecticut Valley represent about 10% of total thickness compared to about 50% for the other two deposits.

Important conclusions obtained from a comparison of the NSP of several varved clays are summarized, as follows: Varved clays which have the same bulk Atterberg limits, but coming from different geological deposits, do not always have the same NSP. Thus, bulk index properties do not necessarily indicate singificant differences in soil composition, especially the presence of cementing agents. Variations in the mineral composition of varves seems to be the most important factor that affects NSP. Varve thickness and differences in the relative portions of "silt" and "clay" layer seem to have less influence at least on FV strengths.

It should be clearly pointed out that the soil samples which came from the Connecticut Valley have the same bulk index properties and basic soil composition, and therefore should have the same NSP.

4.5 SUMMARY AND CONCLUSIONS

Experimental evidence from this chapter has illustrated that, besides having the same index properties (see Chapter 3), varved clays from Connecticut Valley and Hackensack Valley have the same normalized behavior with respect to undrained shear strength parameters ($q_f/\bar{\sigma}_{vc}$, A_f, $\bar{\phi}$ and $\bar{c}/\bar{\sigma}_{vm}$). This is true for all directions of loading: vertical, inclined, and horizontal. Though the data show some differences in the normalized undrained shear modulus from the $\overline{CK_oU}$DSS tests, such differences

are due to the changes in the MPPR ($\frac{\bar{\sigma}_{vm}(L)}{\bar{\sigma}_{vm}}$) and/or the in situ $\bar{\sigma}_{vm}$ rather than the soil location per se.

Analysis of the normalized undrained modulus of a given varved clay shows that for samples which have the same OCR, the sample's stress history (i.e., $\frac{\bar{\sigma}_{vm}(L)}{\bar{\sigma}_{vm}}$ ratio) and magnitude of $\bar{\sigma}_{vm}$ had:

1. Little effect on $E_u/\bar{\sigma}_{vc}$ and ε_f obtained from the $\overline{CK_oU}C$ test; and
2. Some effect on $G/\bar{\sigma}_{vc}$ obtained from the $\overline{CK_oU}$DSS tests, i.e., decreasing modulus with increasing MPPR and/or $\bar{\sigma}_{vm}$.

For the Connecticut Valley varved clay, the effect of sample stress history on the normalized undrained modulus and failure strains is expected to be more than the soil location. In general, the increase in the $\frac{\bar{\sigma}_{vm}(L)}{\bar{\sigma}_{vm}}$ ratio will result in a decrease in the normalized undrained shear modulus and an increase in γ_f at $\tau_{h_{max}}$ for SHANSEP samples which have the same $\bar{\sigma}_{vm}$ and OCR. However, based on data for a small MPPR range, as used in the study, the MPPR ratio has practically no effect on $q_f/\bar{\sigma}_{vc}$, A_f, $\bar{\phi}$ and $\bar{c}/\bar{\sigma}_{vm}$ (i.e., these clays exhibit the normalized behavior with respect to these parameters). Consequently, these normalized parameters from Amherst, Northampton, and East Windsor samples can be used to study the effect of stress system presented in Chapter 5. The suitability of using the SHANSEP method of consolidation for measuring the in situ normalized soil parameter is studied in Chapter 6.

The importance of soil composition was also illustrated. Variation in the mineral composition of varves, especially the presence of cementing agents, seems to be very important, this leading to increased strength compared to uncemented varved clay. Also, bulk index properties do not necessarily indicate changes in soil composition and thus soil properties.

EXPERIMENTAL PROGRAM FOR INVESTIGATING THE EFFECTS OF SOIL SAMPLE'S STRESS HISTORY AND SOIL LOCATION ON NSP

Topic Under Investigation	Type of Test	Soil Location	θ°	OCR	Values of $\frac{\bar{\sigma}_{vm}(L)}{\bar{\sigma}_{vm}}$
Stress History of Soil Sample	$\overline{CK_oUC}$	A		1	1.71, 1.81
				4	1.90, 2.25
		E		1	1.12, 1.25, 1.50
		N		1	1.07, 1.33
	$\overline{CK_oUDSS}$	A	0	1	1.76, 2.35
		N		1	1.01, 1.46, 1.52
				2	1.25, 1.47
		H		1	1.20, 1.60
Soil Location: A = Amherst E = East Windsor H = Hackensack Valley N = Northampton	$\overline{CIUC}$	A	0,45,90	1	1.90
		E	0,45,90	1	1.67
	$\overline{CIUE}$	A	0	1	1.90
		E	0	1	1.67
	$\overline{CK_oUC}$	A	0	1	1.71
		E			1.50
		H			1.40
		N			1.33
		A	0	2	1.71
		E			1.50
		A	0	4	1.90
		E			1.67
	$\overline{CK_oUE}$	E	0	1	1.50
		A		2	1.71
		A	0	4	2.20
		E			1.67
	$\overline{CK_oUDSS}$	A	0	1	1.76
		N			1.52
		E			1.67
		H			1.60
	$\overline{CK_oUDSS}$	E	0	4	1.67
		H			1.60

Table 4.1

EFFECT OF SAMPLE STRESS HISTORY ON THE NSP OF AMHERST VARVED CLAY

Test No.	Soil Location	Type of Test	W_N %	W_L (1) W_P %	In Situ $\bar{\sigma}_{vo}$ σ_{vm}	Lab. $\bar{\sigma}_{vc}$ $\bar{\sigma}_{vm}$(L)	MPPR	OCR	$\frac{q_f}{\bar{\sigma}_{vc}}$	A_f	$\bar{\phi}°$	$\frac{\bar{c}}{\bar{\sigma}_{vm}}$ (2)	ε_f %	$E_u/\bar{\sigma}_{vc}$ at $\frac{\Delta q}{\Delta q_f}=1/2$
AKC-1	Amherst	$\overline{CK_oUC}$	--	--	0.80 1.6	2.99 2.99	1.87	1	0.25	1.32	21	--	1.00	150
AKC-4a			62.8	56.0 28.6	1.5 2.1	3.6 3.6	1.71		0.25	1.27	20.5	--	0.65	189
AKC-3			66.0	56.0 28.6	0.8 1.6	0.9 3.6	2.25	4	0.65	0.33	14	0.088	8.37	196
AKC-4b			62.8	56.0 28.6	1.5 2.1	1.0 4.0	1.90		0.67	0.29	14	0.088	8.06	218

Sample No.	Soil Location	Type of Test	W_N %	W_L W_P %	In Situ $\bar{\sigma}_{vo}$ σ_{vm}	Lab. $\bar{\sigma}_{vc}$ $\bar{\sigma}_{vm}$(L)	MPPR	OCR	$\frac{\tau h_{max}}{\bar{\sigma}_{vc}}$	$\frac{\bar{\sigma}_v}{\bar{\sigma}_{vc}}$	ε_f %	$3G/\bar{\sigma}_{vc}$ at $\Delta\tau/\Delta\tau_{max}=1/2$
ADS-1	Amherst	$\overline{CK_oUDSS}$	61.9	--	1.10 1.70	4.0 4.0	2.35	1	0.17	0.56	8.5	102
ADS-2			60.0	--	1.10 1.70	3.0 3.0	1.76		0.16	0.52	9.0	144

Notes: (1) Bulk material.
(2) From Figure 4.13.

Table 4.2

EFFECT OF STRESS HISTORY ON THE NSP OF NORTHAMPTON VARVED CLAY

Stress in kg/cm^2

Test No.	Soil Location	Type of Test	W_N %	W_L / W_P %	In Situ $\bar{\sigma}_{vo}$ / $\bar{\sigma}_{vm}$	Lab. $\bar{\sigma}_{vc}$ / $\bar{\sigma}_{vm}$ (L)	$\bar{\sigma}_{vm}(L)$ / $\bar{\sigma}_{vm}$	OCR	q_f / $\bar{\sigma}_{vc}$	A_f	$\bar{\phi}°$ ($\bar{c}$=0)	ε_f %	$E_u/\bar{\sigma}_{vc}$ at $\Delta q/\Delta q_f = 1/2$
NKC-1	N	$\overline{CK_oUC}$	--	3.44 3.44	3.02 3.22	3.44 3.44	1.07	1	0.25	1.40	20.1	1.25	188
NKC-2			--	4.0 4.0	-- 3.3	4.0 4.0	1.21		0.24	1.68	20.6	1.07	120 (2)

Test No.	Soil Location	Type of Test	W_N %	W_L / W_P %	In Situ $\bar{\sigma}_{vo}$ / $\bar{\sigma}_{vm}$	Lab. $\bar{\sigma}_{vc}$ / $\bar{\sigma}_{vm}$	$\bar{\sigma}_{vm}(L)$ / $\bar{\sigma}_{vm}$	OCR	$\tau_{h_{max}}$ / $\bar{\sigma}_{vc}$	$\bar{\sigma}_v$ / $\bar{\sigma}_{vc}$	γ_f %	$3G/\bar{\sigma}_{vc}$ at $\Delta\tau/\Delta\tau_{max}=1/2$
NDS-1	N	$\overline{CK_oUDSS}$	61.4	53.7 27.4	2.20 4.00	6.0 6.0	1.52	1	0.15	0.61	8.0	60
NDS-2			57.1	64.3 32.5	1.36 3.20	5.0 5.0	1.46		0.17	0.61	5.9	88
NDS-3			56.3	53.7 27.4	2.24 4.00	4.0 4.0	1.00		0.16	0.66	5.1	75
NDS-4	N	$\overline{CK_oUDSS}$	62.5	63.9 25.6	1.55 3.4	2.50 5.00	1.47	2	0.29	0.94	7.9	92
NDS-5			59.4	--	3.68 4.00	2.50 5.00	1.25		0.28	0.88	7.4	145

Notes: (1) Bulk material.
(2) Shear at $\dot{\varepsilon}$ = 0.05%/hr compared to ε = 0.5% of another sample.

Table 4.3

EFFECT OF SAMPLE STRESS HISTORY ON THE NSP OF EAST WINDSOR AND HACKENSACK VALLEY VARVED CLAY

Stress in kg/cm^2

Test No.	Soil Location	Type of Test	W_N %	(1) W_L W_P %	In Situ $\frac{\bar{\sigma}_{vo}}{\bar{\sigma}_{vm}}$	Lab. $\bar{\sigma}_{vc}$ $\bar{\sigma}_{vm}(L)$	$\frac{\bar{\sigma}_{vm}(L)}{\bar{\sigma}_{vm}}$	OCR	$\frac{q_f}{\bar{\sigma}_{vc}}$	A_f	$\bar{\phi}°$ ($\bar{c}$=0)	ε_f %	$E_u/\bar{\sigma}_{vc}$ at $\frac{\Delta q}{\Delta q_f} = 1/2$
EKC - 3	East Windsor	$\overline{CK_oUC}$	--	50.0	0.6 2.4	2.69 2.69	1.12	2	0.24	1.40	21.9	1.06	110
EKC - 2			--	28.9	0.6 2.4	3.0 3.0	1.25		0.24	1.68	21.1	1.09	140
EKC - 1			56.3		0.6 2.4	3.6 3.6	1.50		0.25	1.30	21.0	0.91	150

Test No.	Soil Location	Type of Test	W_N %	W_L W_P %	In Situ $\frac{\bar{\sigma}_{vo}}{\bar{\sigma}_{vm}}$	Lab. $\bar{\sigma}_{vc}$ $\bar{\sigma}_{vm}(L)$	$\frac{\bar{\sigma}_{vm}(L)}{\bar{\sigma}_{vm}}$	OCR	$\frac{\tau_{h\,max}}{\bar{\sigma}_{vc}}$	$\frac{\bar{\sigma}_v}{\bar{\sigma}_{vc}}$	γ_f %	$3G/\bar{\sigma}_{vc}$ at $\Delta\tau/\Delta\tau_{max} = 1/2$
HDS - 1	Hackensack Valley	$\overline{CK_oUDSS}$	47.0	54.0 22.0	1.3 2.5	4.0 4.0	1.60	1	0.16	0.61	11.7	31 (2)
HDS - 2			46.9	54.0 22.0	1.3 2.5	3.0 2.0	1.20		0.16	0.57	10.7	93

Notes: (1) Bulk material.
(2) May be due to poor testing.

Table 4.4

EFFECT OF SOIL LOCATION ON NSP FROM $\overline{CIUC}$ TESTS ON NORMALLY CONSOLIDATED SAMPLES

Test No.	(1) Soil Location	In Situ $\bar{\sigma}_{vo}$ / $\bar{\sigma}_{vm}$	Lab. $\bar{\sigma}_{vc}$ / $\bar{\sigma}_{vm}$	$\bar{\sigma}_{vm}(L)$ / $\bar{\sigma}_{vm}$	$\theta°$	At $(\sigma_1-\sigma_3)_{max}$ $q/\bar{\sigma}_{vc}$	A_f	ε_f %	$\bar{\phi}°$	At $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ $q/\bar{\sigma}_{vc}$	A_f	ε_f %	$\bar{\phi}°$	$E_u/\bar{\sigma}_{vc}$ at ε = 1/2%
AIC-1-1	A	1.5 / 2.1	4.0 / 4.0	1.90	0	0.25	1.43	5.0	28.6	0.25	1.51	16.8	33.0	78
EIC-1-1	E	0.6 / 2.4	4.0 / 4.0	1.50	0	0.24	1.39	4.45	25.0	--	--	--	--	70
HIC-1-1	H	1.5 / 2.1	4.0 / 4.0	1.54	0	0.25	1.27	5.3	23.6	0.24	1.44	9.5	25.3	70
AIC-2-1	A	1.5 / 2.1	4.0 / 4.0	1.90	45	0.21	1.63	5.9	24.5	0.21	1.78	9.9	27.0	70
EIC-2-1	E	0.6 / 2.4	4.0 / 4.0	1.50	45	0.21	1.60	5.2	22.0	0.19	2.09	10.3	25.5	50
AIC-3-1	A	1.5 / 2.1	4.0 / 4.0	1.90	90	0.28	1.19	9.7	28.0	0.28	1.19	9.7	28.0	70
EIC-3-1	E	0.6 / 2.4	4.0 / 4.0	1.50	90	0.28	1.18	9.3	28.5	0.28	1.18	9.3	28.5	64

Note: (1) A = Amherst; H = Hackensack Valley; E = East Windsor

Table 4.5

EFFECT OF SOIL LOCATION ON NSP FROM $\overline{CK_oUC}$ TESTS

Test No.	Soil Location	Type of Test	W_N %	(1) W_L W_P %	In Situ $\bar{\sigma}_{vo}$ $\bar{\sigma}_{vm}$	Lab. $\bar{\sigma}_{vc}$ $\bar{\sigma}_{vm}$	$\frac{\bar{\sigma}_{vm}(L)}{\bar{\sigma}_{vm}}$	OCR	$\frac{q_f}{\bar{\sigma}_{vc}}$	A_f	ε_f %	$\bar{\phi}°$	(2) $\bar{c}/\bar{\sigma}_{vm}$	$E_u/\bar{\sigma}_{vc}$ at $\frac{\Delta q}{\Delta q_f} = 1/2$
ACK-4a	A	$\overline{CK_oUC}$	--	53.0 28.6	1.0 2.1	3.6 3.6	1.71	1	0.25	1.27	0.65	20.5	0	189
EKC-1	E		56.3	50.0 28.9	0.6 2.4	3.6 3.6	1.50	1	0.25	1.30	0.91	21.0	0	150
HKC-1	H		--	54.0 22.0	1.28 2.50	3.0 3.0	1.40	1	0.23	1.36	0.59	18.6	0	187
NKC-2	N		--	--	-- 3.0	4.0 4.0	1.33	1	0.24	1.68	1.1	20.6	0	108 (3)
AKC-2	A		67.9	53.0 28.6	1.0 2.1	1.8 3.6	1.71	2	0.40	0.54	2.8	14	0.088	260
EKC-4	E		58.5	50.0 28.9	0.6 2.4	1.8 3.6	1.50	2	0.41	0.45	2.8	14	0.088	213
AKC-4b	A		62.8	53.0 28.6	1.5 2.1	1.0 4.0	1.90	4	0.67	0.29	8.1	14	0.088	218
EKC-5	E		56.3	50.0 28.9	0.6 2.4	1.0 4.0	1.67	4	0.68	0.21	6.2	14	0.088	150

Notes: (1) Bulk material.
(2) Obtained from Figure 4.16.
(3) Sheared at very low strain rate $\dot{\varepsilon}$ = 0.05%/hr.

Table 4.6

EFFECT OF SOIL LOCATIONS ON THE NSP FROM $\overline{CK_oUDSS}$ TESTS

Test No.	Soil Location (1)	Type of Test	W_N (2) %	W_L (2) / W_P %	In Situ $\bar{\sigma}_{vo}$ / $\bar{\sigma}_{vm}$	Lab. $\bar{\sigma}_{vc}$ / $\bar{\sigma}_{vm}$ (L)	$\frac{\bar{\sigma}_{vm}(L)}{\bar{\sigma}_{vm}}$	OCR	$\frac{\tau_{h_{max}}}{\bar{\sigma}_{vc}}$	$\frac{\bar{\sigma}_v}{\bar{\sigma}_{vc}}$ (4)	γ_f %	$3G/\bar{\sigma}_{vc}$ at $\tau_h/\tau_{h_{max}} = 1/2$
ADS-2	A	$\overline{CK_oUDSS}$	48.4	56.0 / 28.3	1.10 / 1.70	3.0 / 3.0	1.76	1	0.16	0.58	9.0	144 (3)
NDS-1	N		61.4	53.7 / 27.4	2.20 / 4.00	6.00 / 6.00	1.52		0.15	0.61	8.0	60 (3)
EDS-1	E		54.8	50.0 / 28.9	0.60 / 2.40	4.01 / 4.01	1.67		0.16	0.59	7.4	50
HDS-1	H		47.0	54.0 / 22.0	1.30 / 2.50	4.00 / 4.00	1.60		0.16	0.61	11.7	27
EDS-2	E	$\overline{CK_oUDSS}$	54.1	50.0 / 28.9	0.60 / 2.40	1.03 / 4.02	1.68	4	0.46	1.21	8.8	92
HDS-4	H		46.0	54.0 / 22.0	1.30 / 2.50	0.97 / 4.00	1.60		0.50	1.36	14.8	50

Notes:
(1) A = Amherst; N = Northampton; E = East Windsor; H = Hackensack Valley
(2) Bulk Material.
(3) Samples did not have the same $\bar{\sigma}_{vm}$.
(4) At $\tau_{h_{max}}$.

Table 4.7

UNDRAINED STRENGTH DATA ON SAMPLES FROM TWO DEPTHS OF MATAGAMI VARVED CLAY

Data from Matagami Varved Clay from NGI Report No. 71307-3

$W_L = 94.1 \pm 3.0\%$; $W_p = 40.9 \pm 3.2\%$; PI = $53.2 \pm 6.2\%$

Depth = 23 ft; $W_N = 98.7 \pm 3.5\%$; $\bar{\sigma}_{vo} = 550$ lb/ft^2; $\bar{\sigma}_{vm}$ [3] = 1000 lb/ft^2

Type of Test	K_c	OCR	$S_u/\bar{\sigma}_{vo}$	Remarks
$\overline{CK_oUC}$ [1]	0.6	1.8	0.72 ± 0.01	$S_u = q_f$ (avg. of 2 tests)
$\overline{CK_oU}$DSS	0.6	1.8	0.39 ± 0.01	$S_u = \tau_{hmax}$ (avg. of 4 tests)
$\overline{CK_oUE}$ [2]	0.6	1.8	0.48 ± 0.05	$S_u = q_f$ (avg. of 3 tests)

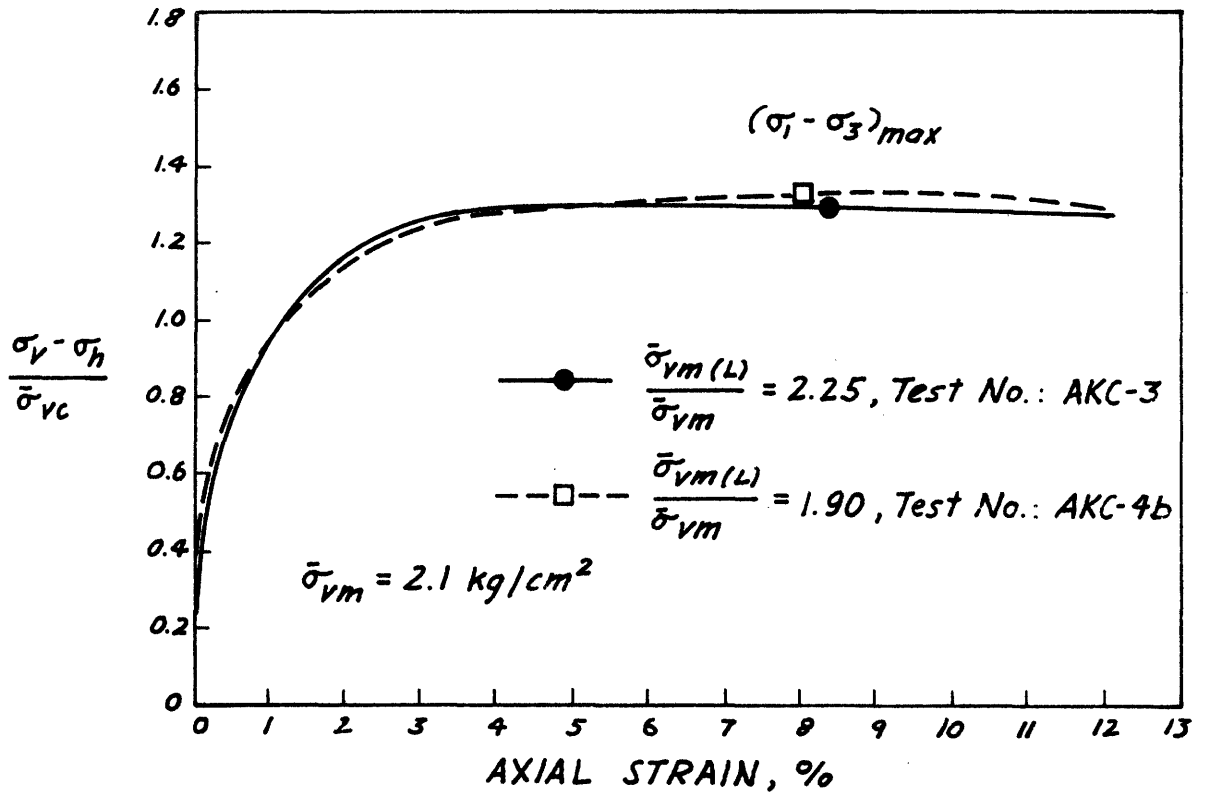
$W_L = 76.1 \pm 0.7\%$; $W_p = 36.9 \pm 0.9\%$; PI = $39.2 \pm 1.6\%$

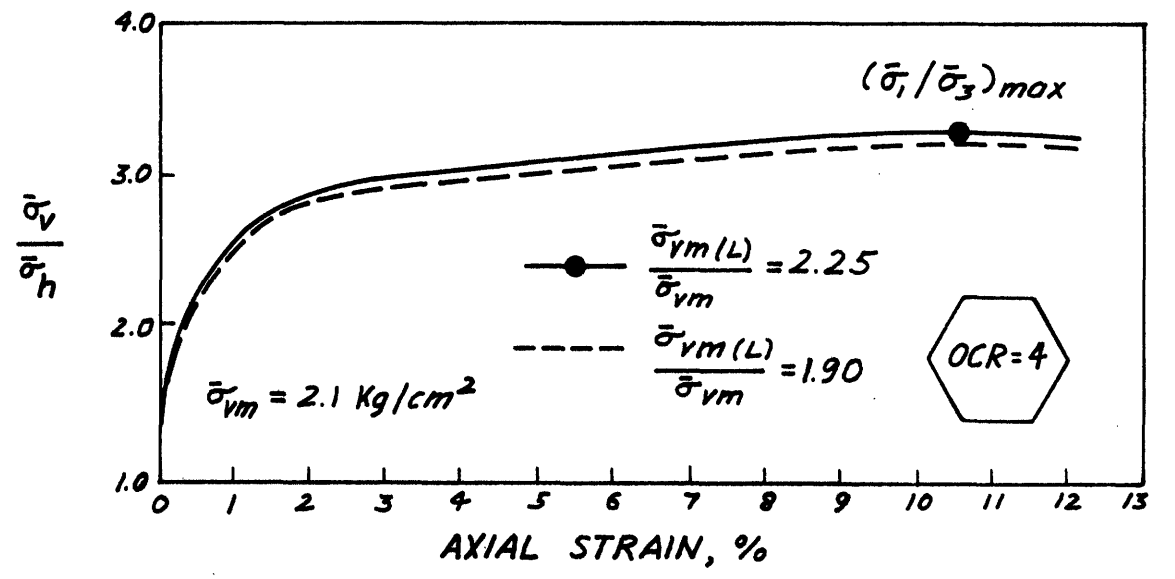
Depth = 28 ft; $W_N = 87.8 \pm 0.8\%$; $\bar{\sigma}_{vo} = 760$ lb/ft^2; $\bar{\sigma}_{vm}$ [3] = 1300 lb/ft^2

Type of Test	K_c	OCR	$S_u/\bar{\sigma}_{vo}$	Remarks
$\overline{CK_oUC}$ [1]	0.60	1.7	0.57	$S_u = q_f$
$\overline{CK_oU}$DSS	0.60	1.7	$0.34 \pm .05$	$S_u = \tau_{hmax}$ (avg. of 3 tests)
$\overline{CK_oUE}$ [2]	0.60	1.7	0.43	$S_u = q_f$

Notes:
(1) Failure across the varves was observed in $\overline{CK_oUC}$ tests.
(2) Failure by bulging of silt was observed in $\overline{CK_oUE}$ tests.
(3) Interpolated to these particular depths.

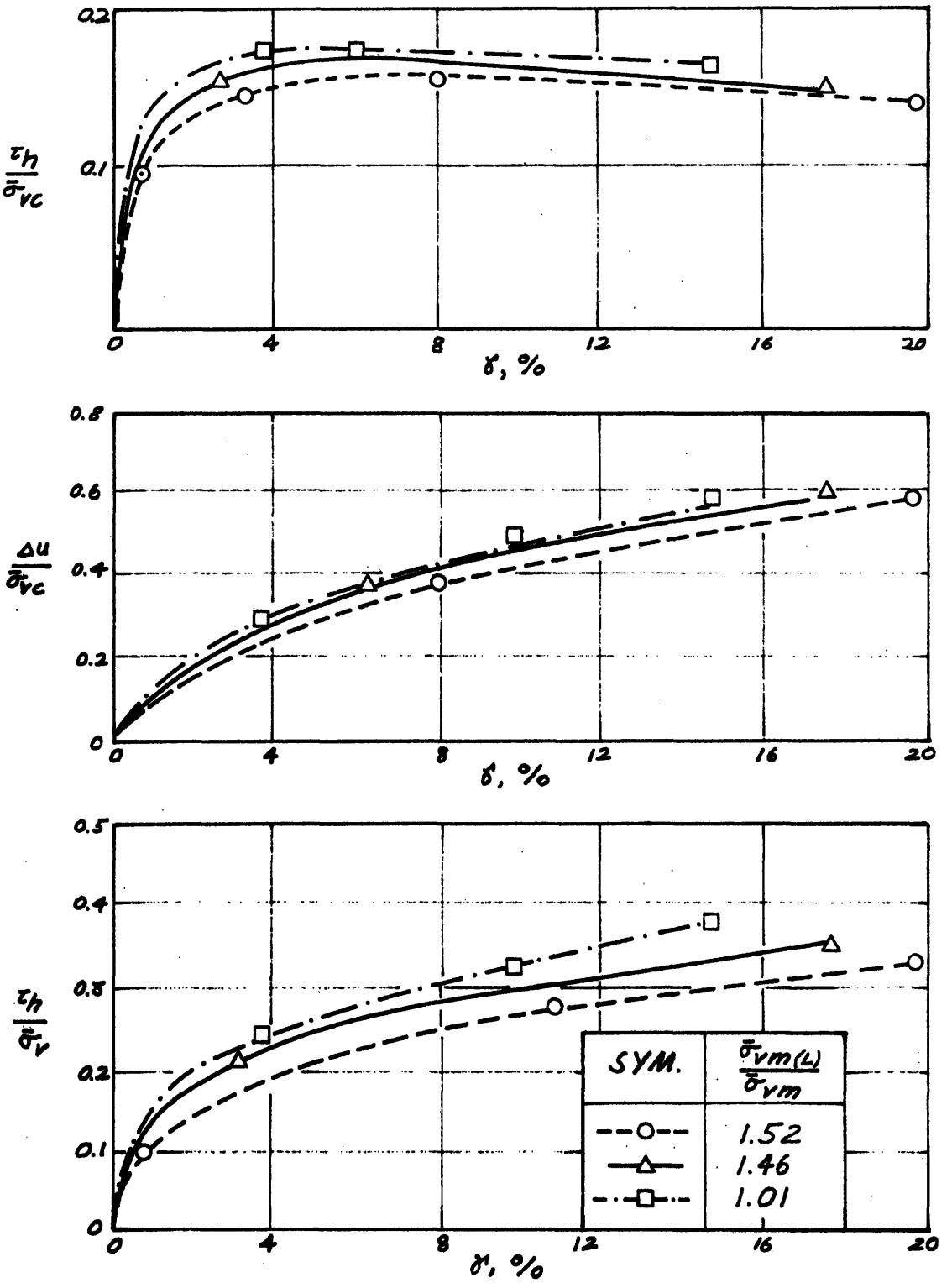
Table 4.8





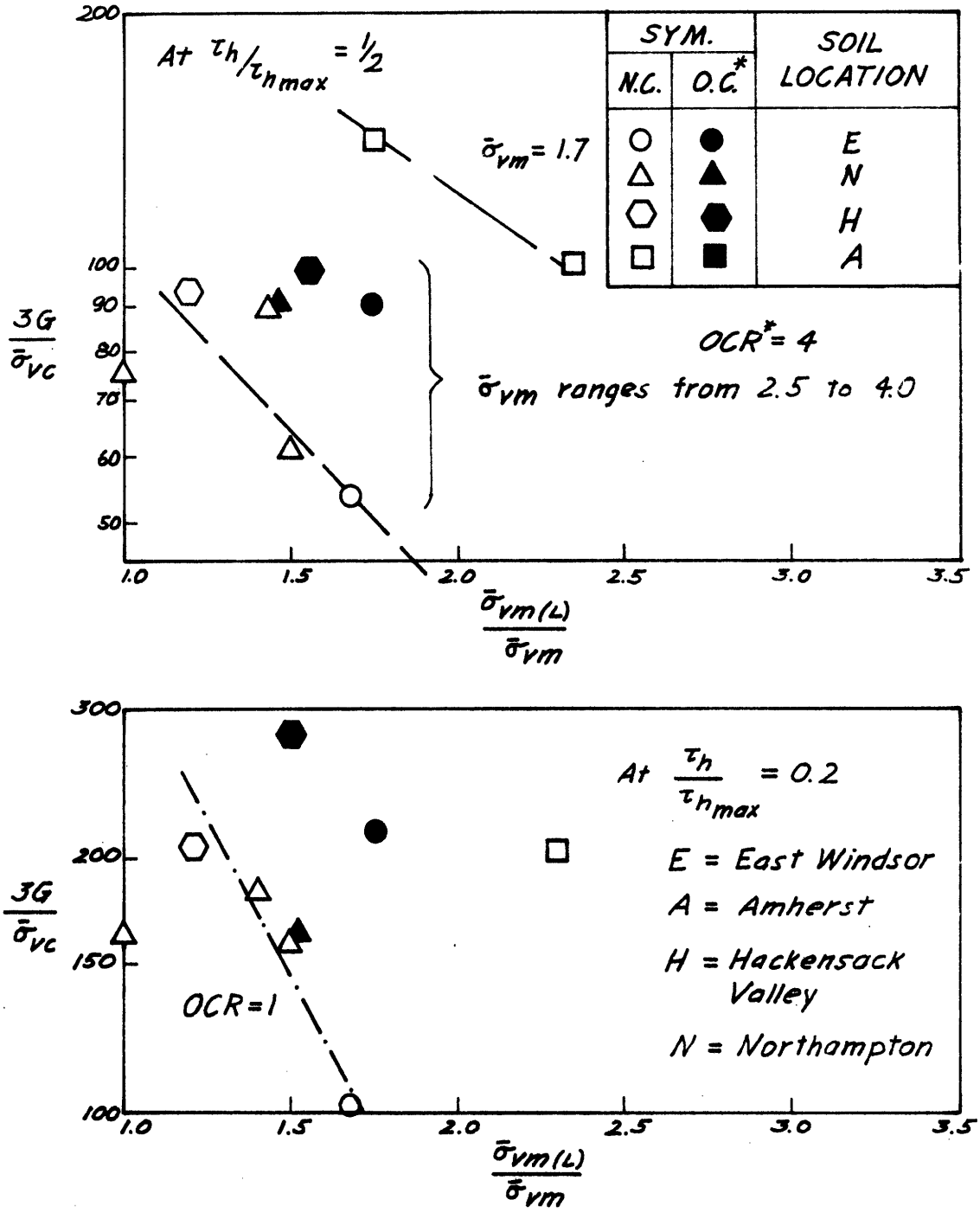
NORMALIZED STRESS-STRAIN CURVES FROM $\overline{CK_oUC}$ TESTS ON OVERCONSOLIDATED AMHERST VARVED CLAY AT TWO $\frac{\bar{\sigma}_{vm(L)}}{\bar{\sigma}_{vm}}$ RATIOS

FIGURE 4.1



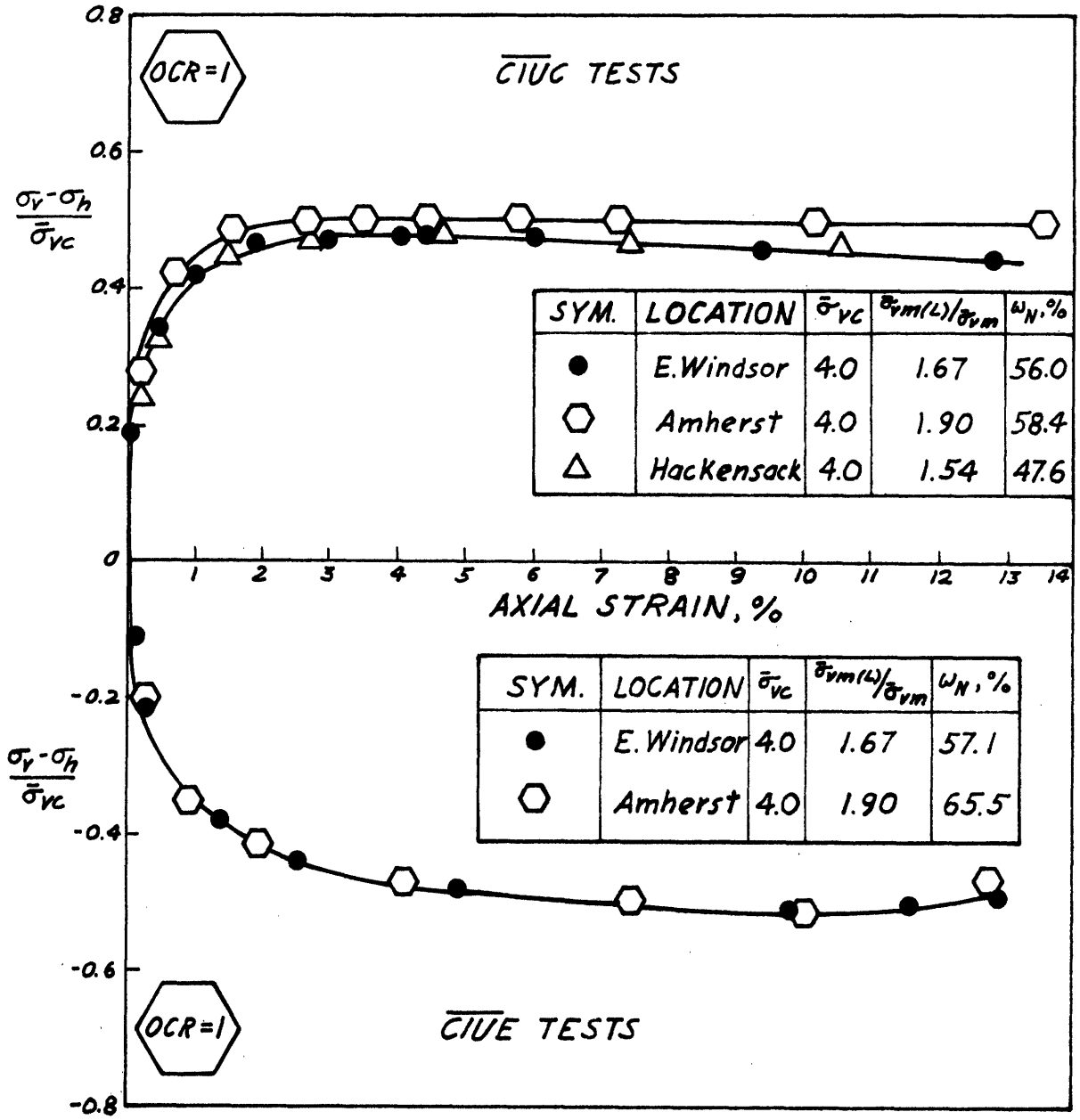
NORMALIZED STRESS-STRAIN CURVES FROM $\overline{CK_oUDSS}$ TESTS ON NORMALLY CONSOLIDATED NORTHAMPTON VARVED CLAY

FIGURE 4.2



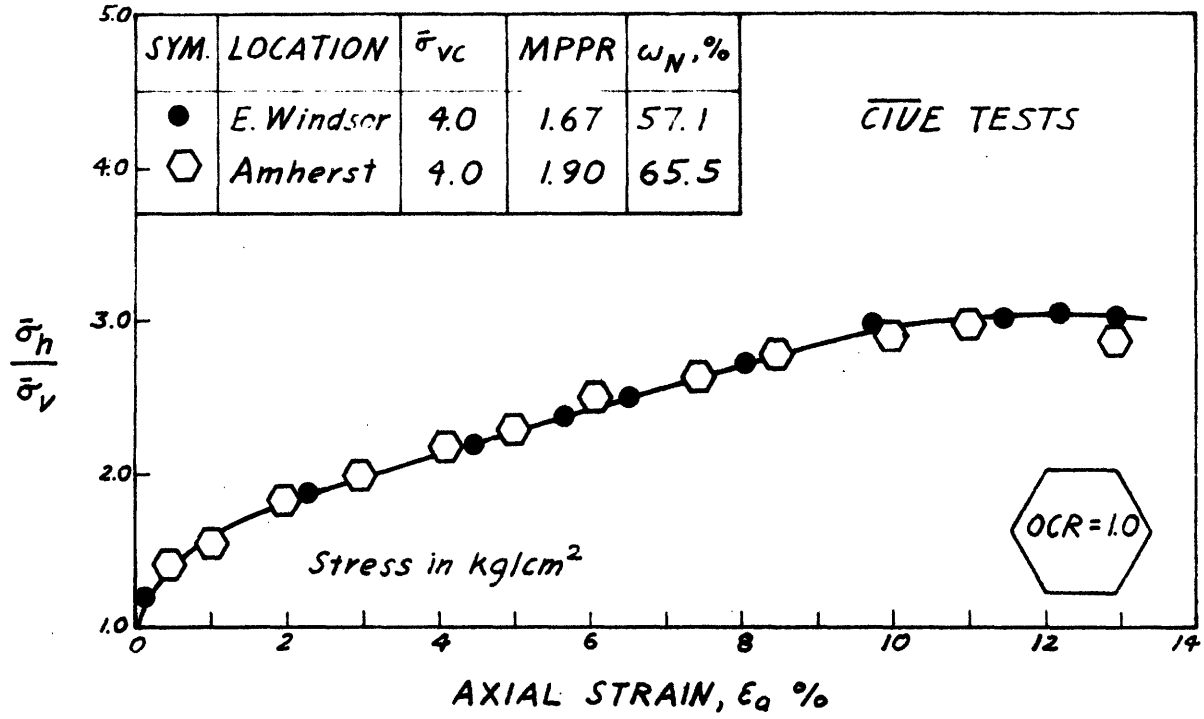
EFFECT OF MPPR ON VALUES OF $\frac{3G}{\bar{\sigma}_{vc}}$ FROM $\overline{CK_oUDSS}$ TESTS ON NORTHAMPTON, AMHERST, EAST WINDSOR AND HACKENSACK VALLEY VARVED CLAYS

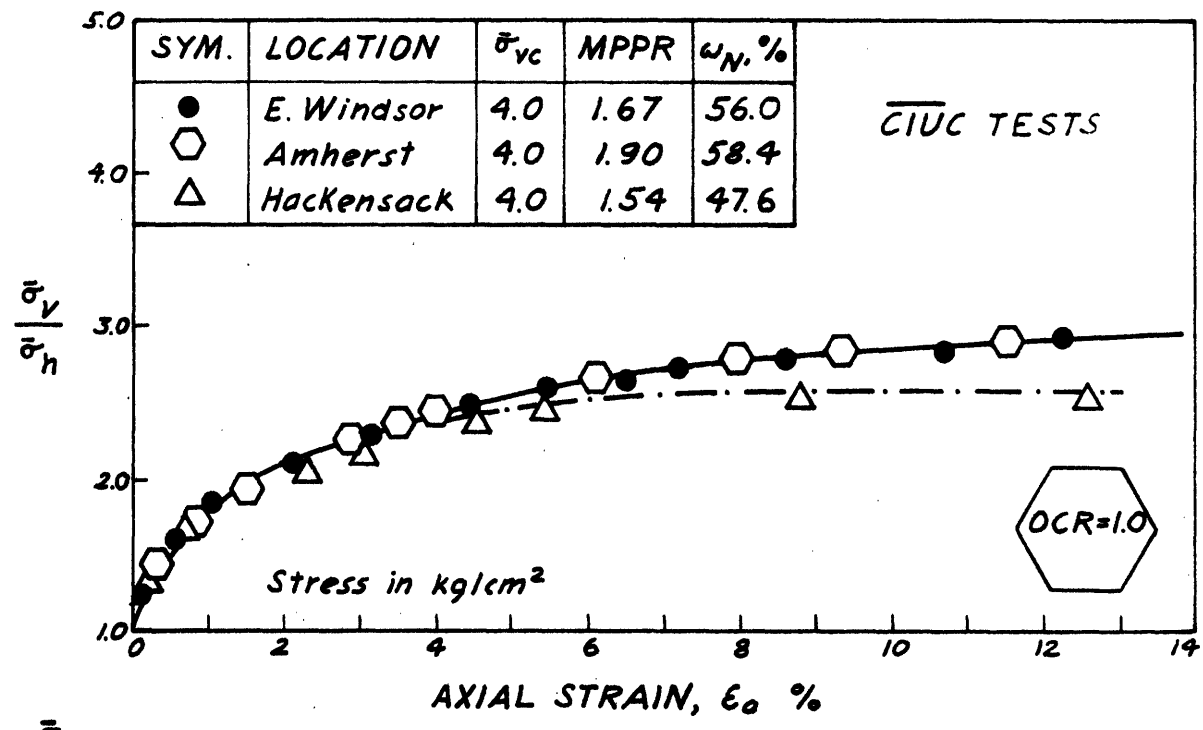
FIGURE 4.3



COMPARISON OF STRESS-STRAIN CURVES FROM $\overline{CIUC}$ AND $\overline{CIUE}$ TESTS ON NORMALLY CONSOLIDATED VERTICAL SAMPLES FROM AMHERST, EAST WINDSOR, AND HACKENSACK VALLEY

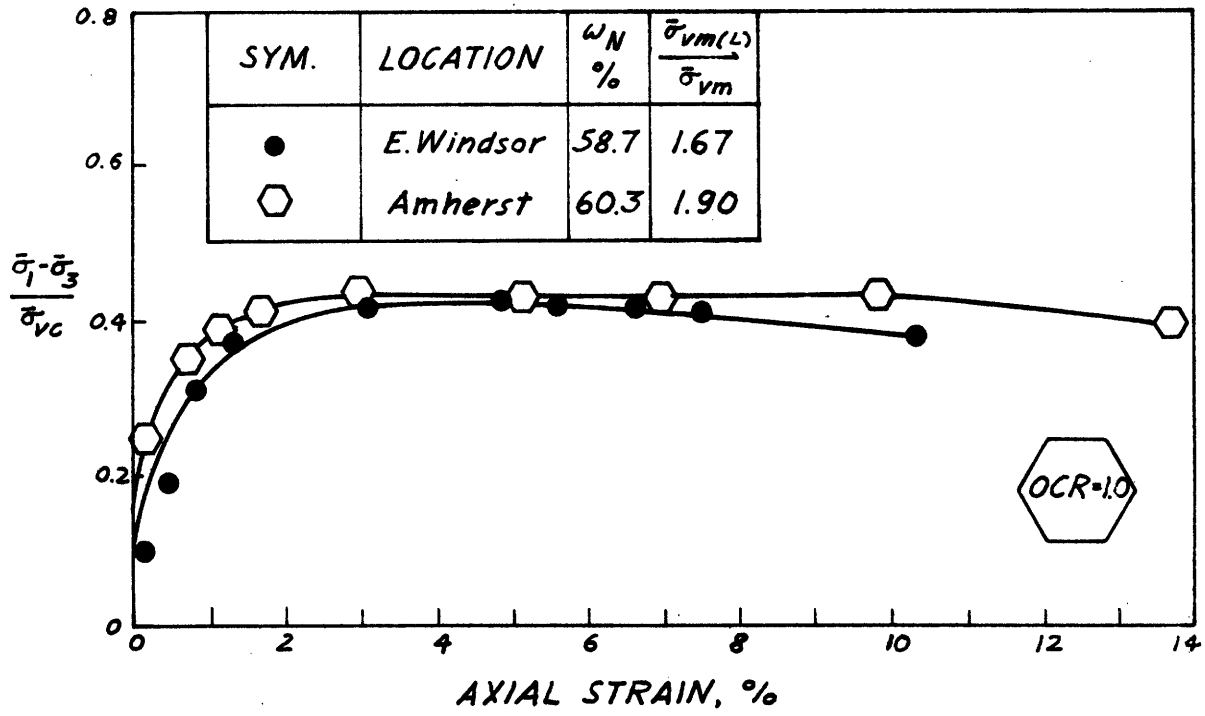
FIGURE 4.4

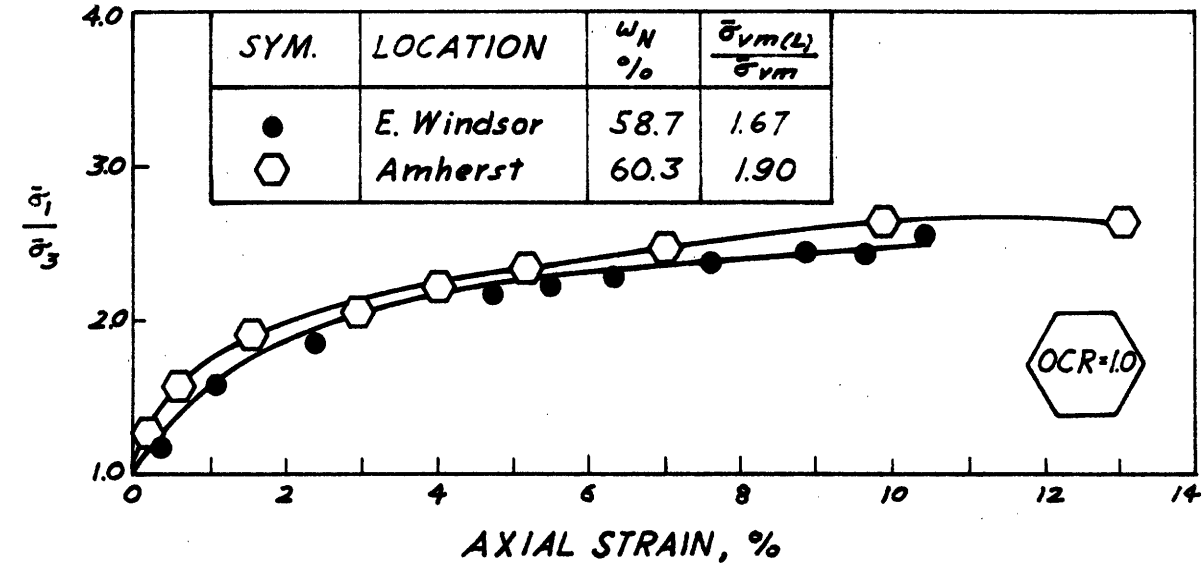




$\frac{\bar{\sigma}_1}{\bar{\sigma}_3} - \varepsilon$ CURVES FROM $\overline{CIUC}$ AND $\overline{CIUE}$ TESTS ON NORMALLY CONSOLIDATED VERTICAL SAMPLES FROM AMHERST, EAST WINDSOR, AND HACKENSACK VALLEY

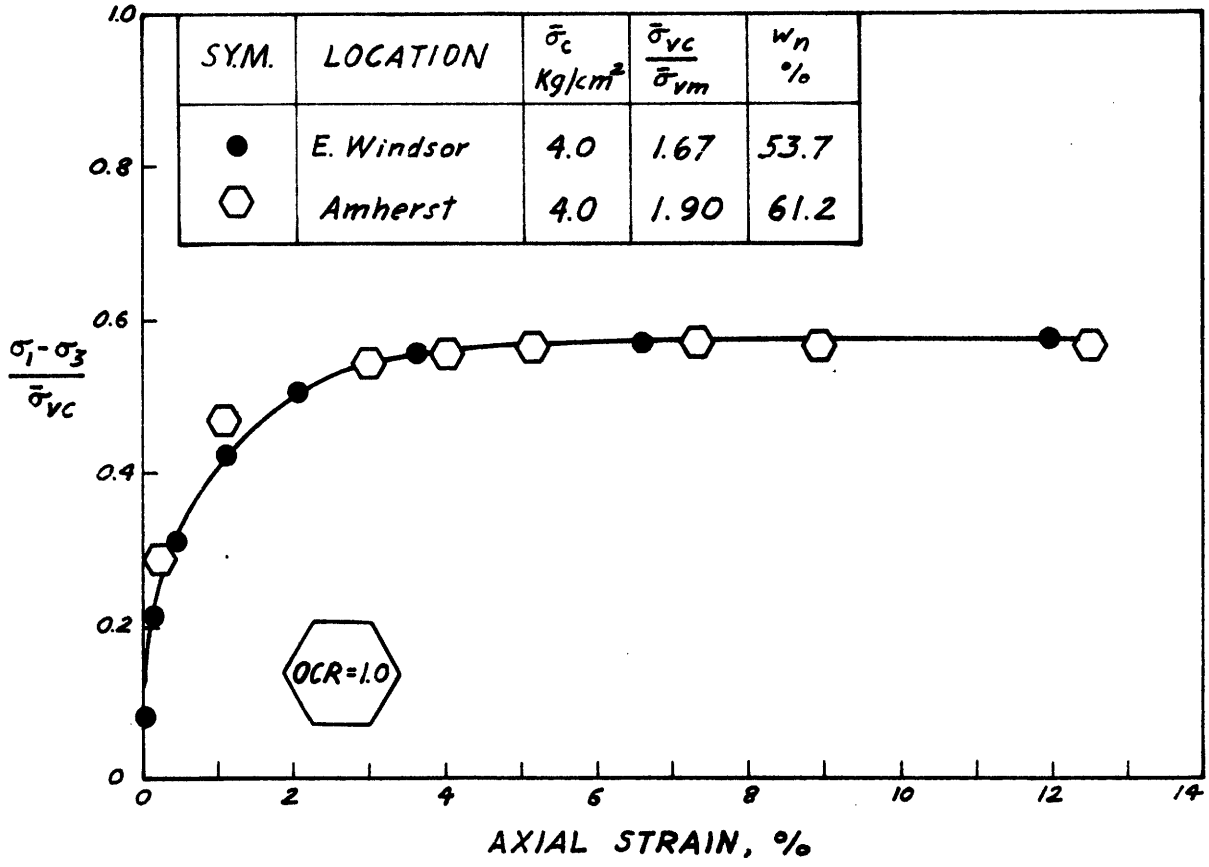
FIGURE 4.5

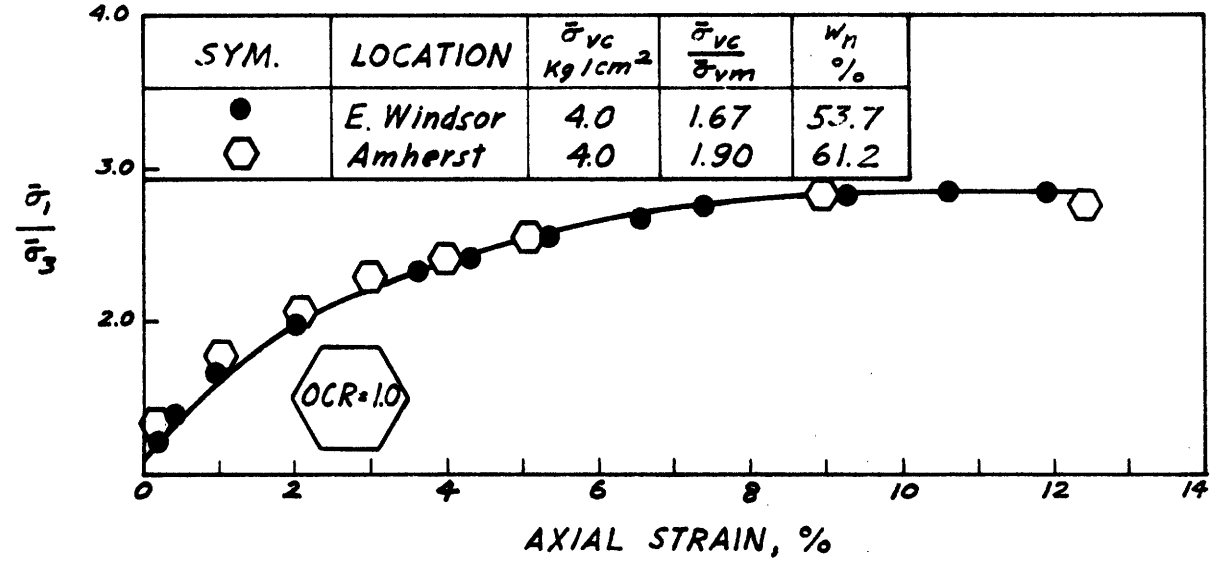




COMPARISON OF STRESS-STRAIN CURVES FROM $\overline{CIUC}$ TESTS ON INCLINED SAMPLES (45°) OF NORMALLY CONSOLIDATED AMHERST AND E. WINDSOR VARVED CLAYS

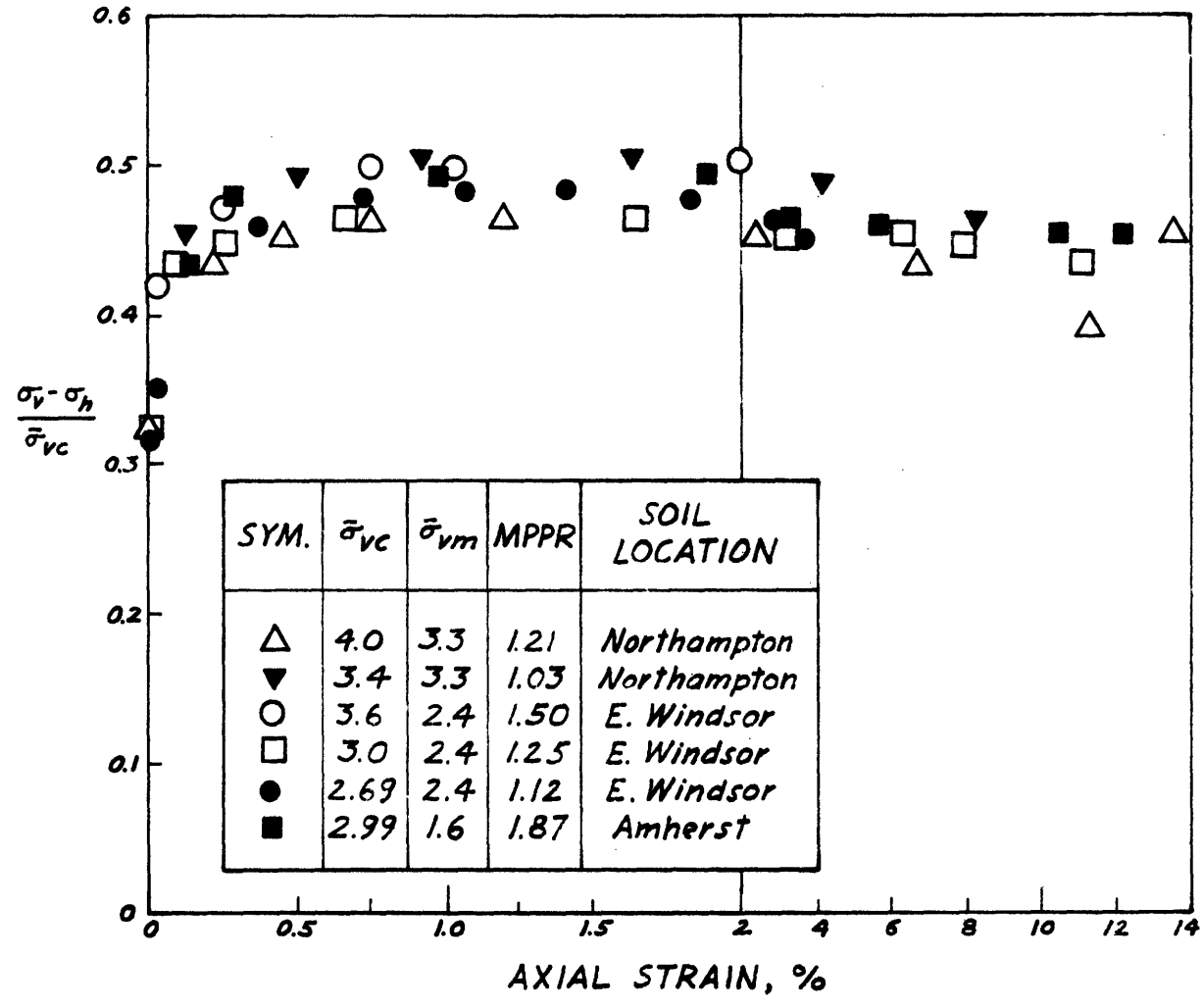
FIGURE 4.6





COMPARISON OF STRESS-STRAIN CURVES FROM $\overline{CIUC}$ TESTS ON HORIZONTAL SAMPLES OF NORMALLY CONSOLIDATED AMHERST AND E. WINDSOR VARVED CLAYS

FIGURE 4.7



NORMALIZED STRESS-STRAIN CURVES FROM $\overline{CK_oUC}$ TESTS ON NORMALLY CONSOLIDATED VERTICAL SAMPLES FROM CONNECTICUT VALLEY

FIGURE 4.8

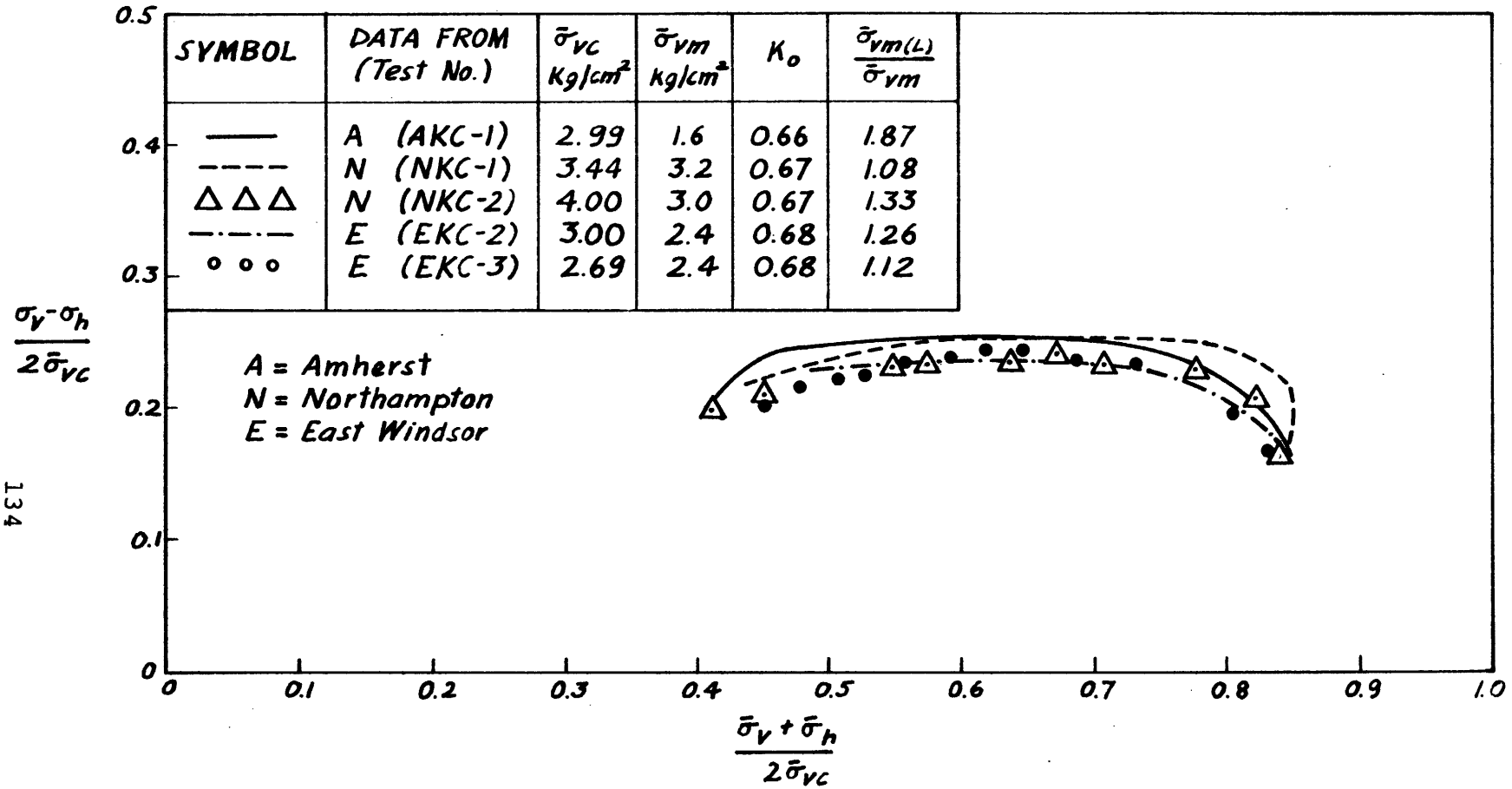
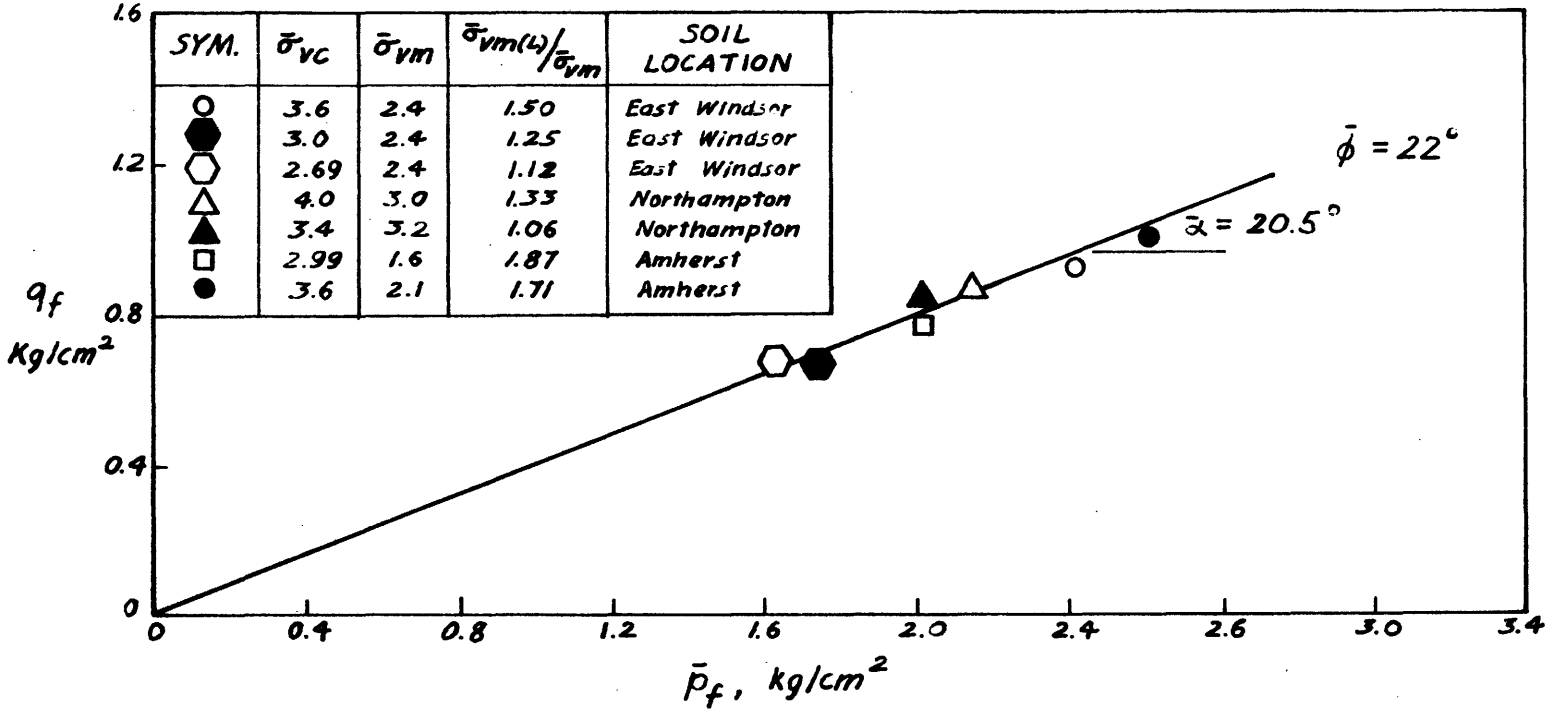


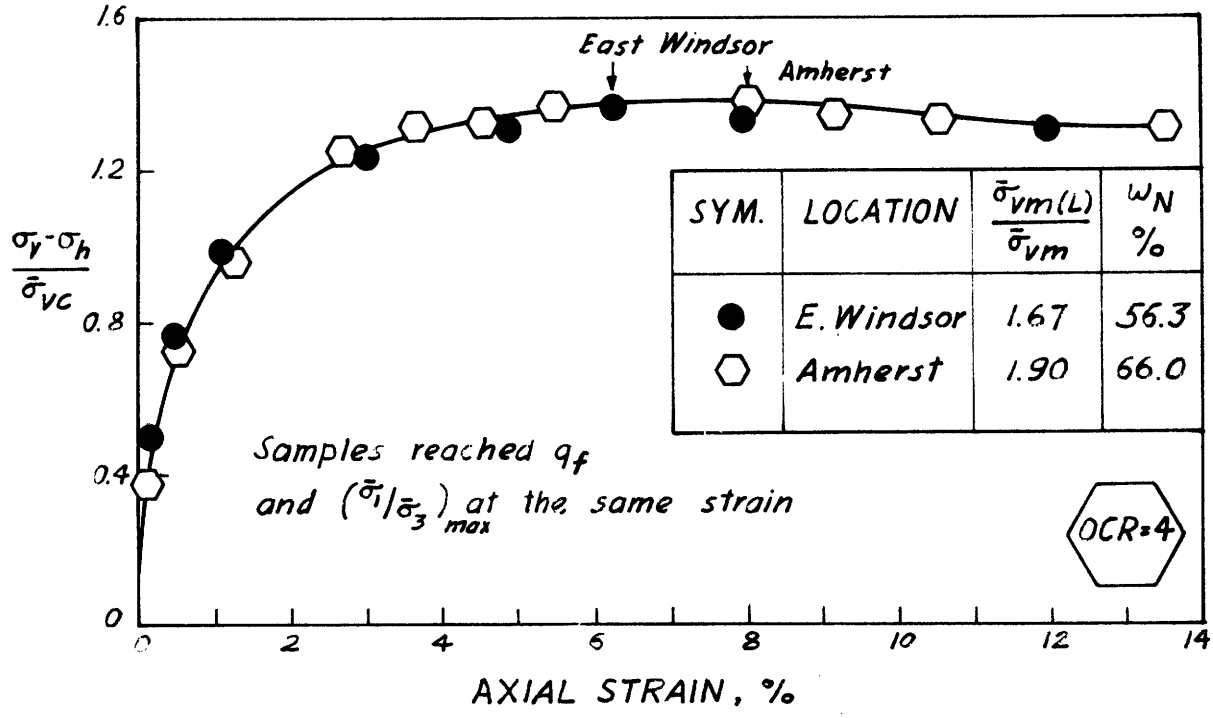
FIGURE 4.9

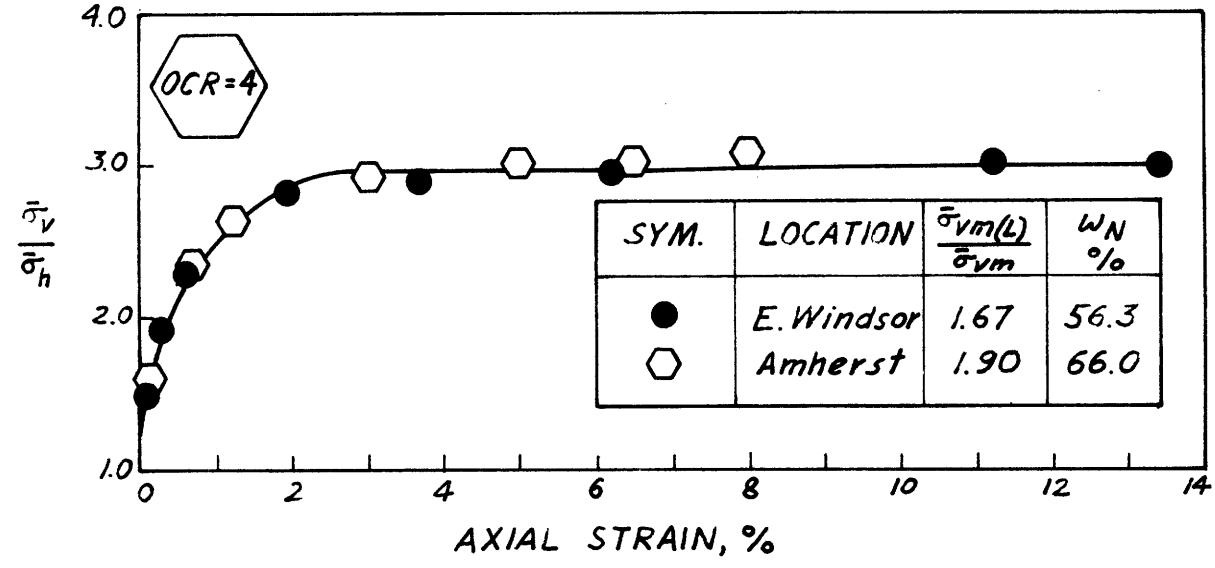
NORMALIZED EFFECTIVE STRESS PATHS FROM $\overline{CK_oUC}$ TESTS ON NORMALLY CONSOLIDATED VERTICAL SAMPLES FROM CONNECTICUT VALLEY



$\bar{p}_f$ vs q_f AT MAXIMUM SHEAR STRESS FROM $\overline{CK_oUC}$ TESTS ON NORMALLY CONSOLIDATED VERTICAL SAMPLES FROM CONNECTICUT VALLEY

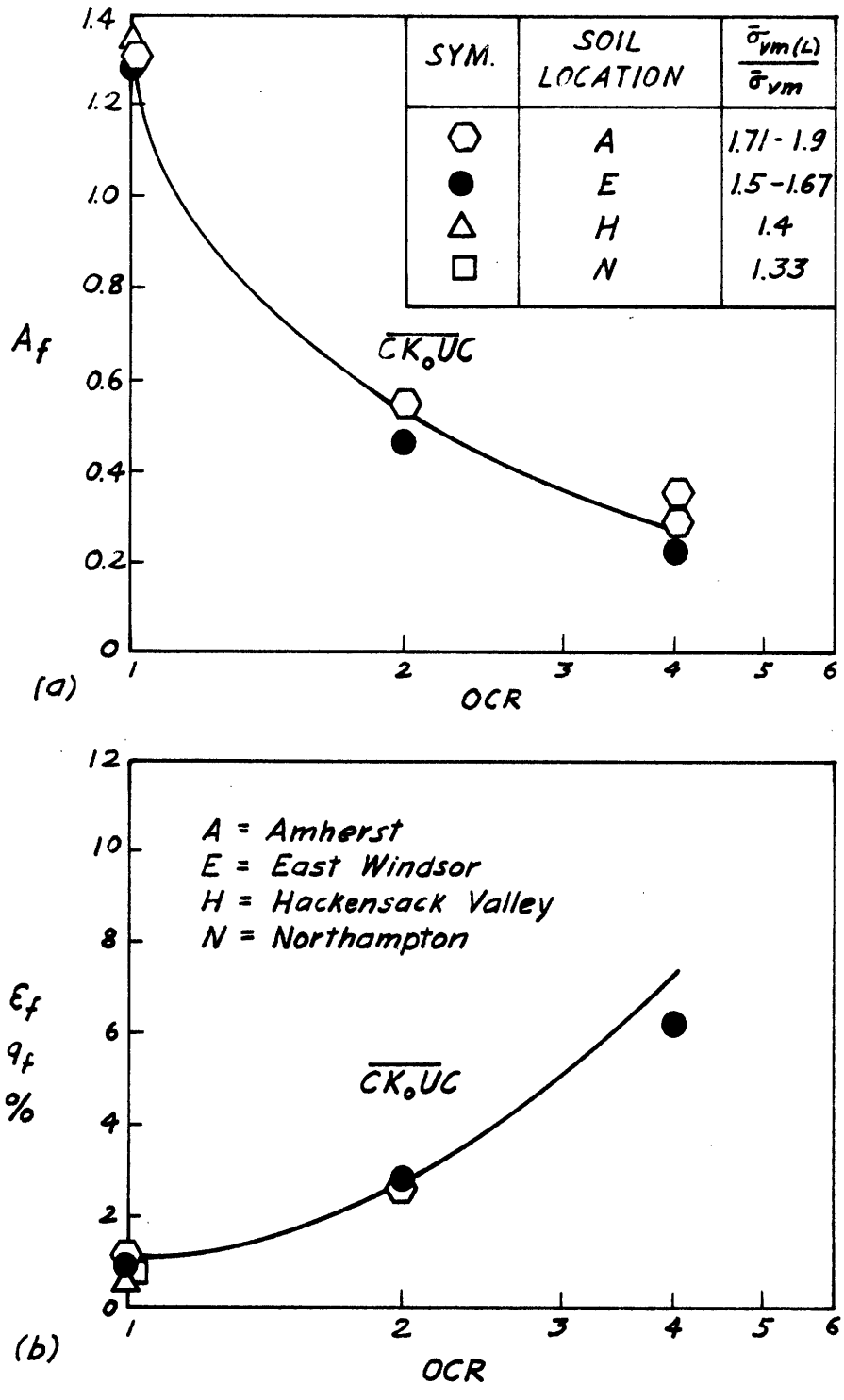
FIGURE 4.10





COMPARISON OF NORMALIZED STRESS-STRAIN CURVES FROM $\overline{CK_oUC}$ TESTS ON OVERCONSOLIDATED AMHERST AND EAST WINDSOR VARVED CLAYS

FIGURE 4.11



A_f AND ε_f VS OCR FROM $\overline{CK_oUC}$ TESTS ON AMHERST AND EAST WINDSOR VARVED CLAYS

FIGURE 4.12

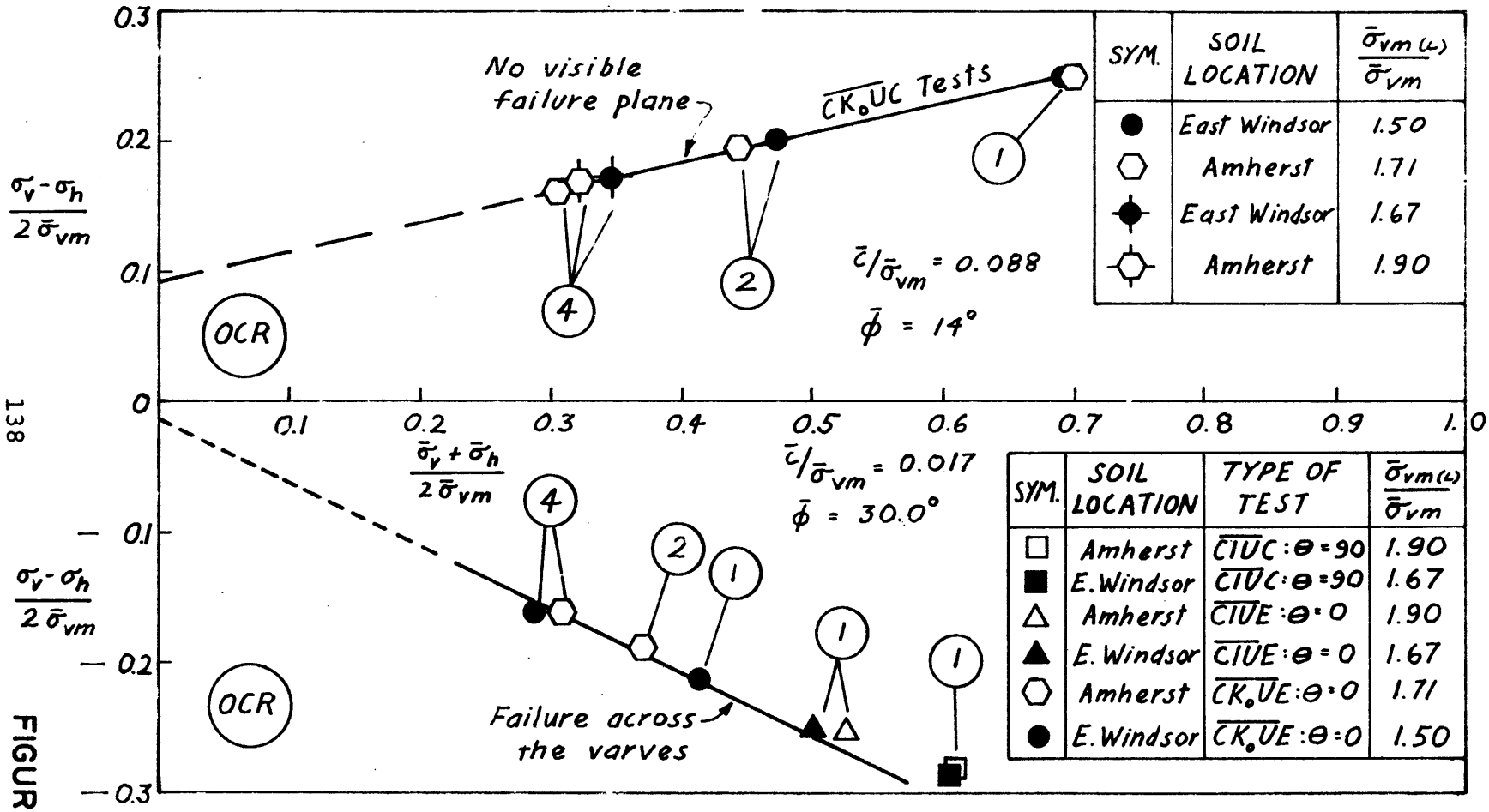
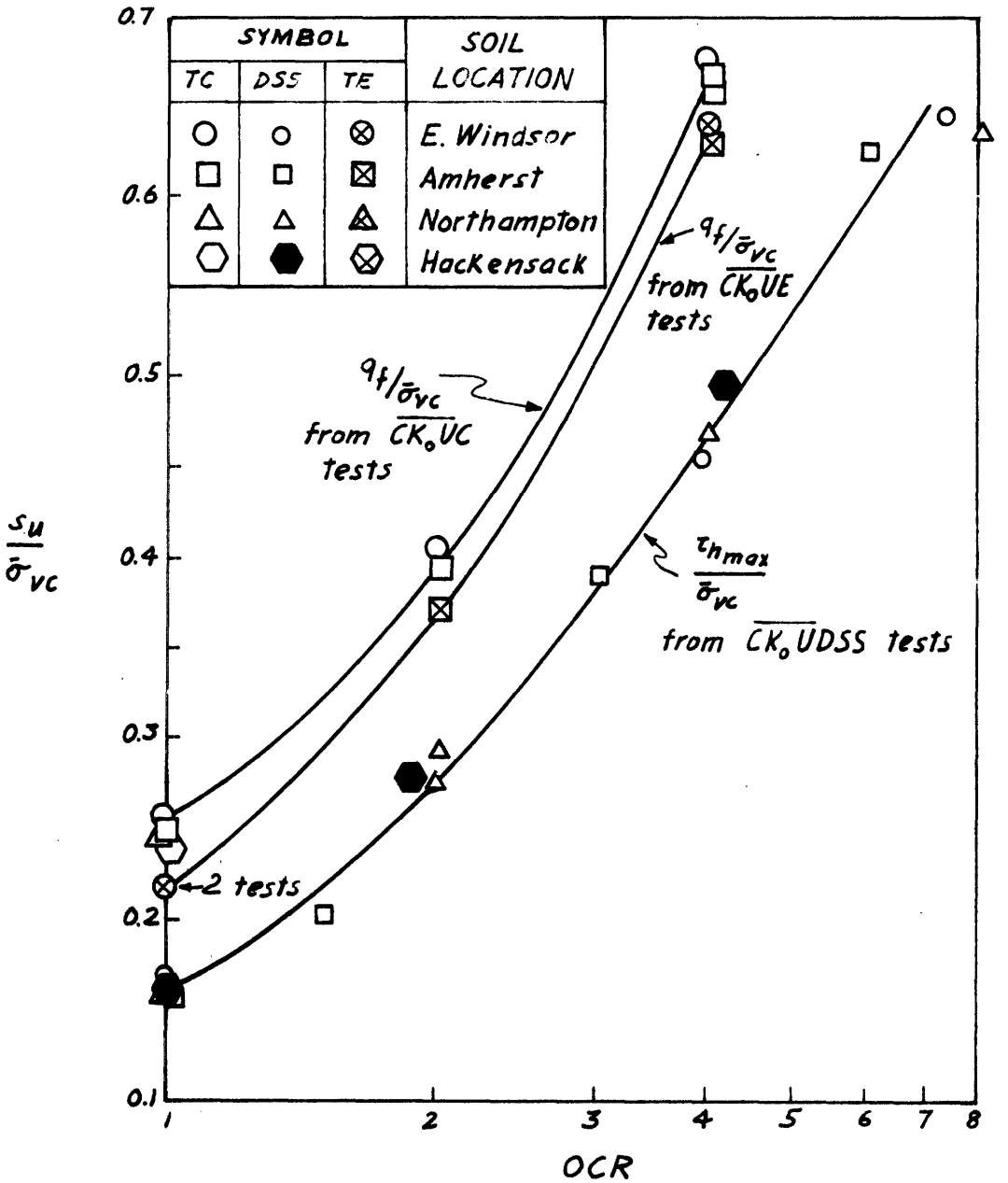


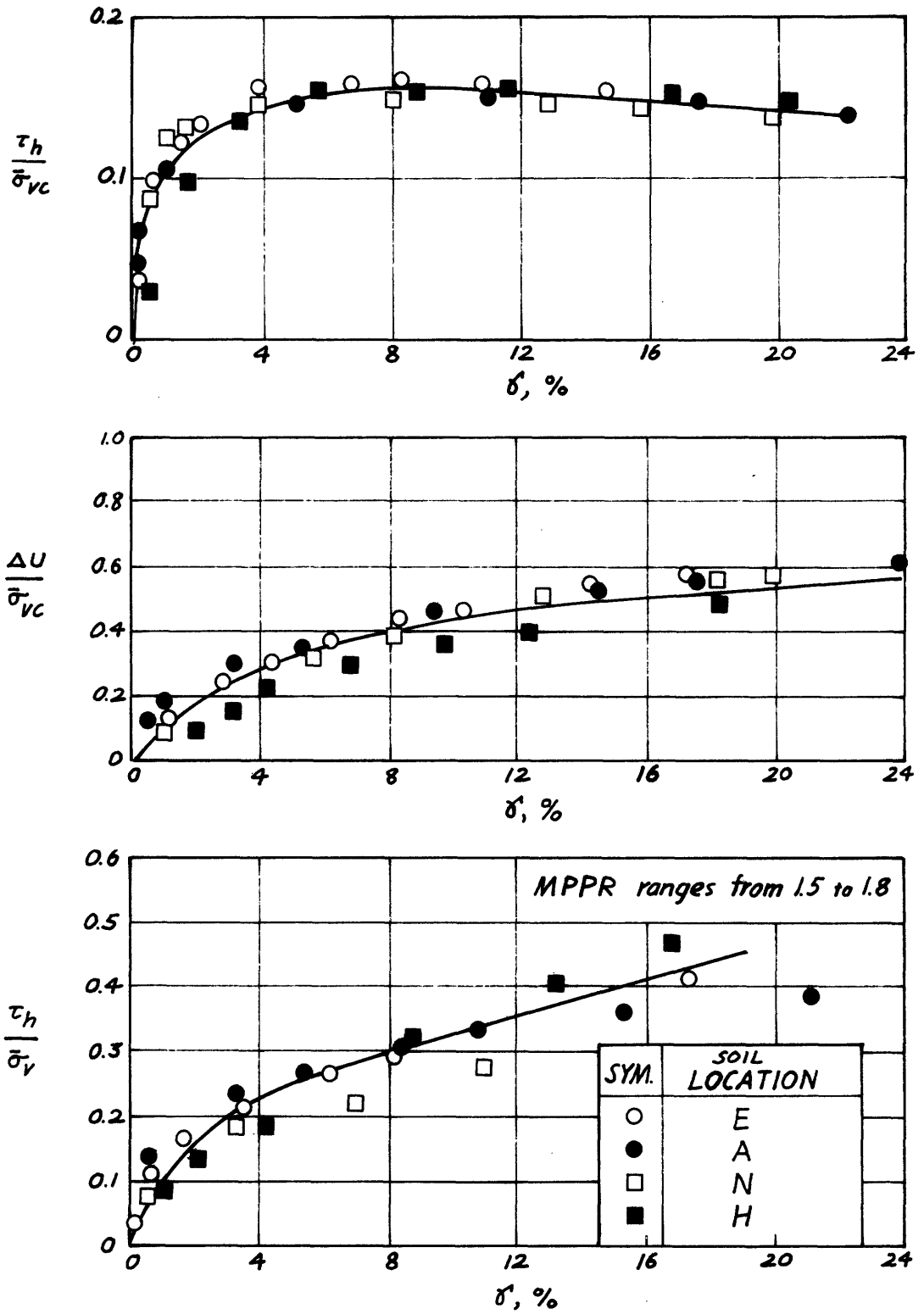
FIGURE 4.13

EFFECTS OF SOIL LOCATION ON
NORMALIZED EFFECTIVE STRESS ENVELOPE AT MAXIMUM SHEAR STRESS

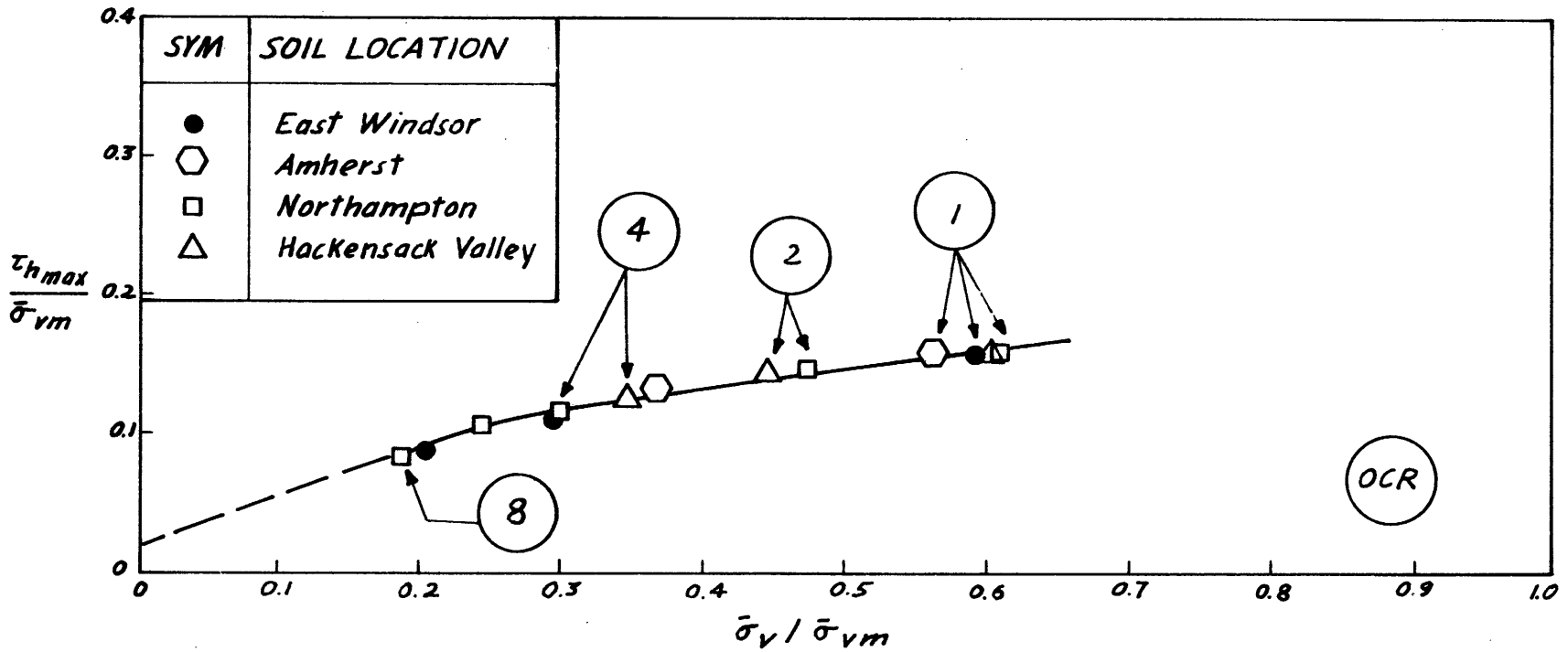


NORMALIZED UNDRAINED SHEAR STRENGTH OF CONNECTICUT VALLEY AND HACKENSACK VALLEY VARVED CLAYS

FIGURE 4.14

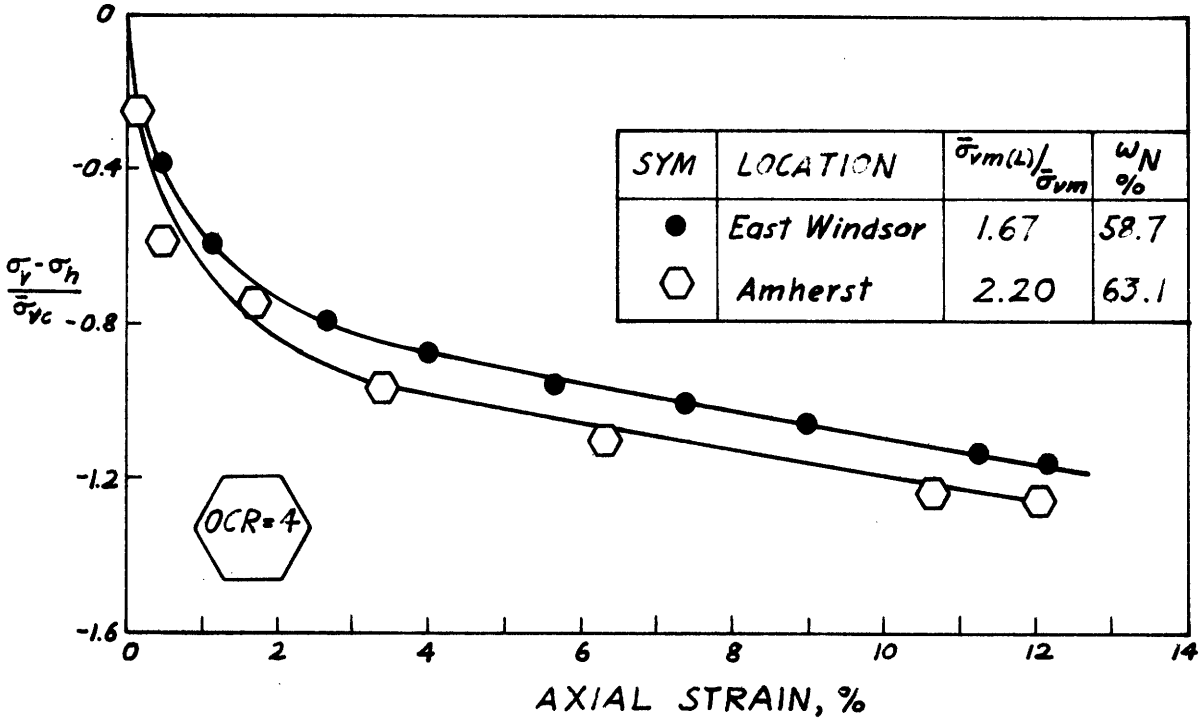


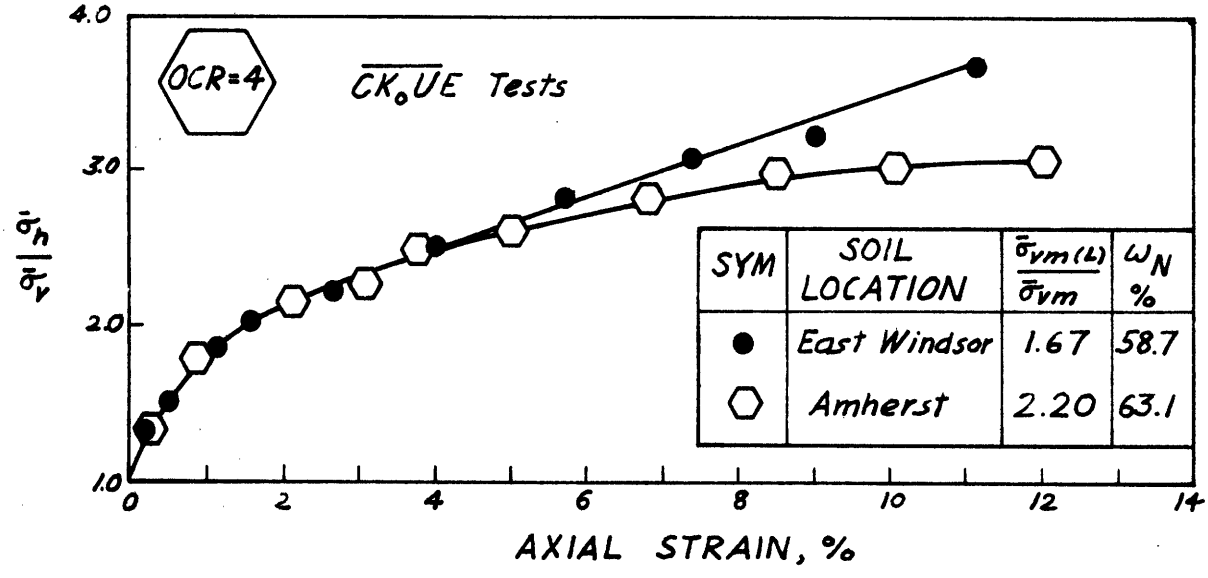
NORMALIZED STRESS-STRAIN CURVES FROM $\overline{CK_oUDSS}$ TESTS ON NORMALLY CONSOLIDATED NORTHAMPTON, AMHERST, EAST WINDSOR, AND HACKENSACK VALLEY VARVED CLAYS FIGURE 4.15



NORMALIZED $\frac{\tau_{h\,max}}{\bar{\sigma}_{vm}}$ VS $\frac{\bar{\sigma}_v}{\bar{\sigma}_{vm}}$ OF CONNECTICUT VALLEY AND HACKENSACK VALLEY VARVED CLAYS FROM $\overline{CK_oU}$DSS TESTS

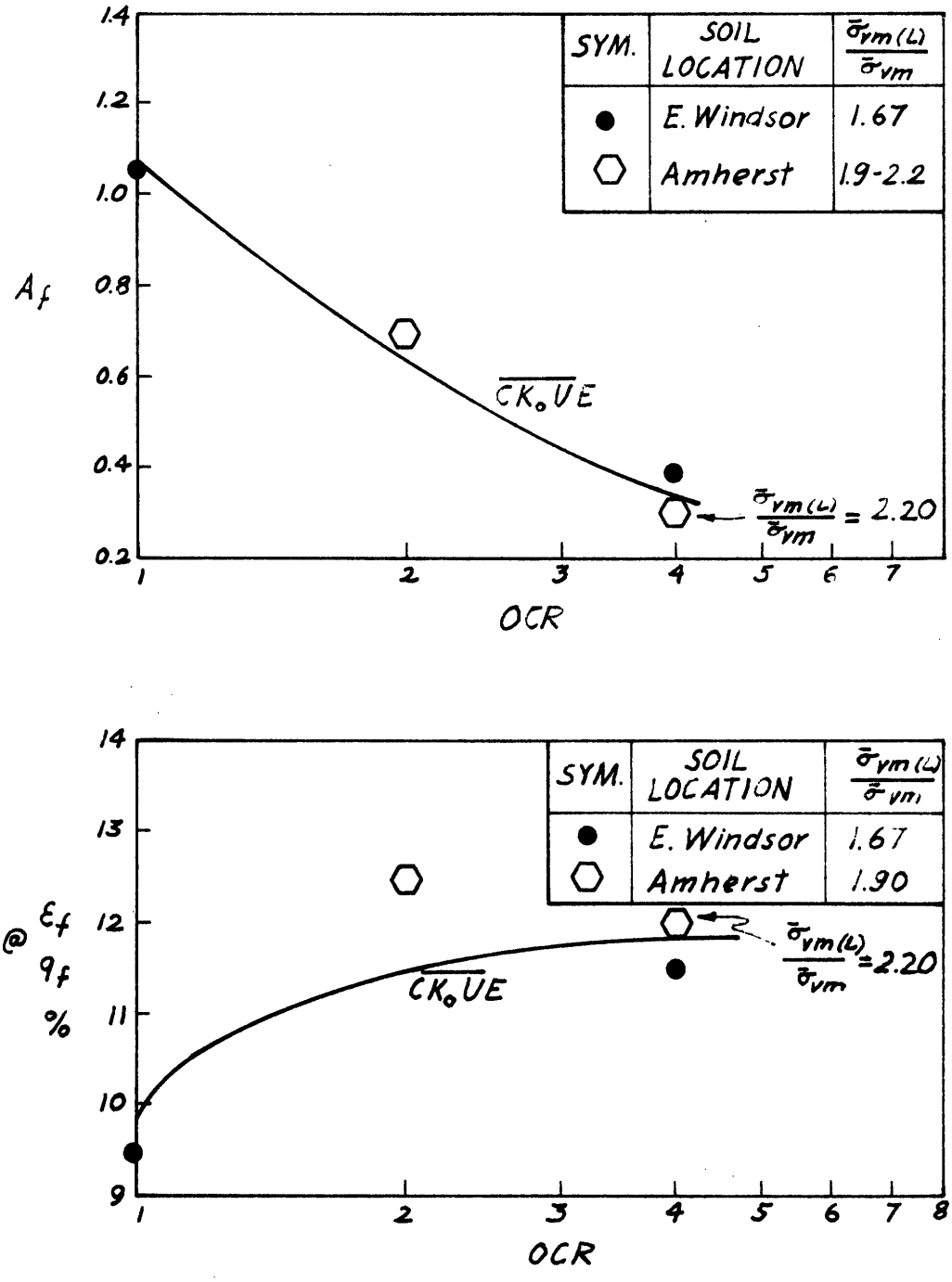
FIGURE 4.16





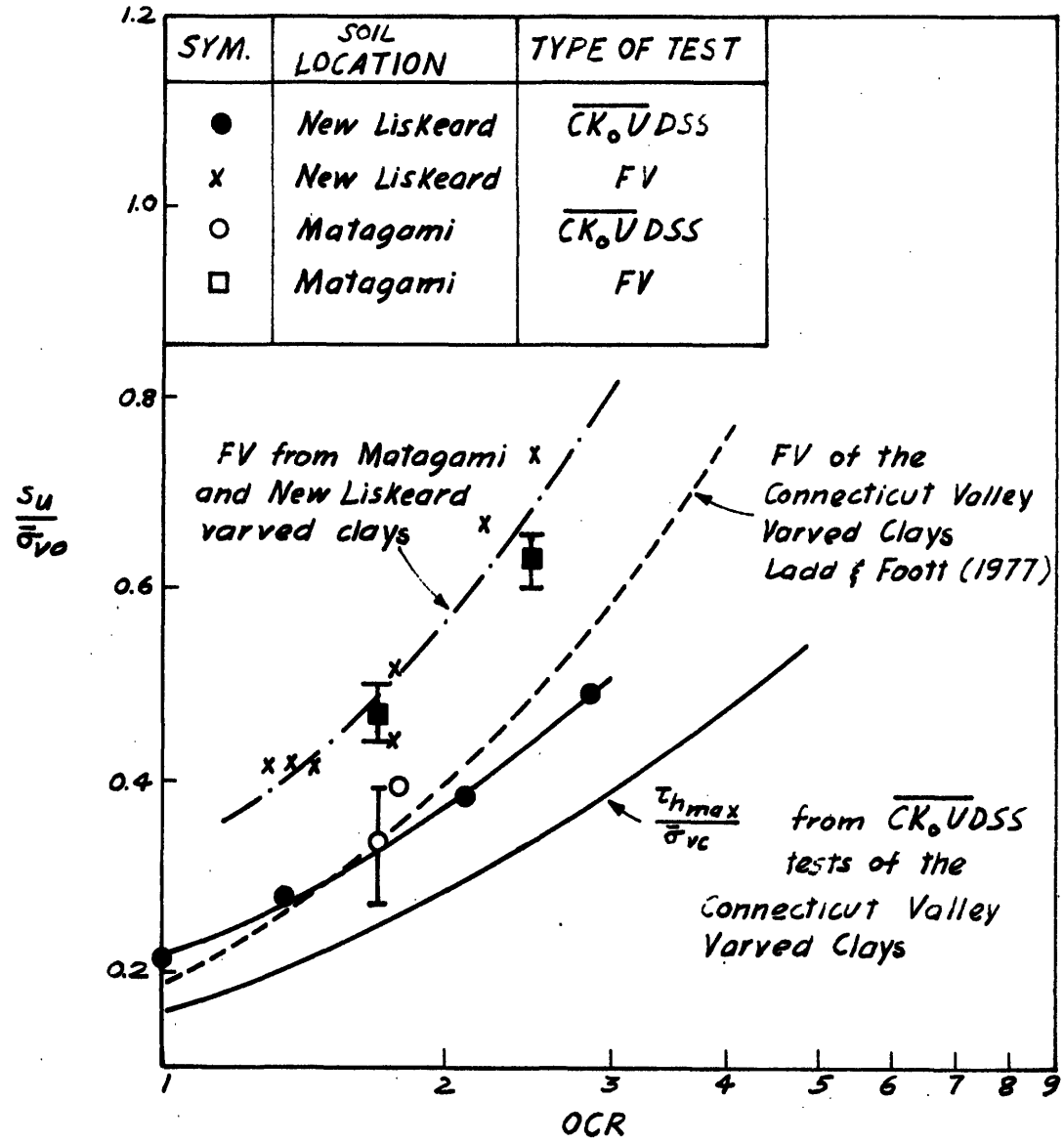
NORMALIZED STRESS-OBLIQUITY-STRAIN FROM $\overline{CK_oUE}$ TESTS ON OVERCONSOLIDATED AMHERST AND EAST WINDSOR VARVED CLAYS

FIGURE 4.17



A_f AND ε_f VS OCR FROM $\overline{CK_oUE}$
TESTS OF EAST WINDSOR AND AMHERST VARVED CLAYS

FIGURE 4.18



COMPARISON OF THE NORMALIZED UNDRAINED SHEAR STRENGTHS OF CONNECTICUT VALLEY, MATAGAMI, AND NEW LISKEARD VARVED CLAYS

FIGURE 4.19

5. EFFECT OF STRESS SYSTEM ON UNDRAINED BEHAVIOR FROM SHANSEP SAMPLES

5.1 INTRODUCTION

The general requirements for accurately measuring the undrained stress-strain-strength behavior of varved clay and its anisotropy are:

1. Performing tests on samples having the same soil structure and physical conditions as the in situ varved clay;
2. Performing tests on samples in a manner to insure that the stress system, time, and environment (temperature, pore fluid characteristics, etc.) are the same as those which will be imposed in the field.

From these requirements, the major factors that could affect the measured behavior are; (1) sample disturbance; (2) the stress system before and during shear; (3) time prior to and during shear; and (4) the environment.

The effects of stress system on undrained behavior are studied in this chapter. All data are obtained by the SHANSEP method of consolidation and time and environment are treated as constants. The effects of sample disturbance and alternate methods of consolidation will be covered in Chapter 6.

The stress system, both prior to shear and that applied during shear means the direction and relative magnitudes of the

three principal stresses. The stress system prior to shear is therefore that resulting from the relative magnitude and the direction of $\bar{\sigma}_{1c}$, $\bar{\sigma}_{2c}$, and $\bar{\sigma}_{3c}$. Laboratory samples are frequently isotropically consolidated whereas the in situ stress condition is generally anisotropic, with a K_o stress state often being of greatest interest. The applied stress system during shear is that resulting from the applied total stress path, which in turn specifies the relative magnitude and direction of $\Delta\sigma_1$, $\Delta\sigma_2$ and $\Delta\sigma_3$ during shear. Thus, the direction of $\bar{\sigma}_{1f}$ versus $\bar{\sigma}_{1c}$ (i.e., rotation of principal stresses) and the magnitude of the intermediate principal stress are the variables for the applied stress system during shear.

The study in this chapter concentrates on three major topics, as follows:

1. Effect of K_o versus isotropic consolidation, i.e., the study of the effect of stress system prior to shear;
2. Effect of the intermediate principal stress (σ_2);
3. Effect of different stress system used for causing shear parallel to the varves.

The section presents data from tests having different directions of $\bar{\sigma}_{1f}$ relative to the orientation of the varves and from tests with rotation of the principal stresses. Such effects are investigated by comparing the resultant NSP ($q_f/\bar{\sigma}_{vc}$, A_f, $E_u/\bar{\sigma}_{vc}$, $\bar{\phi}$ and $\bar{c}/\bar{\sigma}_{vm}$).

Table 5.1 presents a summary of the experimental program. It consisted of:

1. $\overline{CIU}$ and $\overline{CK_oU}$ tests on vertical and horizontal samples of OCR's of 1, 2, and 4 for investigating the effect of K_o versus isotropic consolidation;
2. $\overline{CIUC}$ and $\overline{CIUE}$ tests on vertical and horizontal samples at OCR's of 1, 2, and 4 for investigating the effect of σ_2 in $\overline{CIU}$ tests;
3. $\overline{CK_oUC}$, $\overline{CK_oUE}$, $\overline{CK_oUPSC}$, $\overline{CK_oUPSE}$, $\overline{CK_oUDSS(C)}$, and $\overline{CK_oUDSS(E)}$ tests on normally consolidated samples for investigating the effect of σ_2 in $\overline{CK_oU}$ tests;
4. $\overline{CIUC}$ tests on inclined samples at OCR's of 1, 4, and 20, $\overline{CK_oUDSS}$ and CDDS tests on vertical samples with various OCR's for investigating the effect of testing method for shearing parallel to the varves.

As noted previously, the SHANSEP method of consolidation was used in the study. Experimental results from K_o consolidated SHANSEP samples in Chapter 4 indicated that Connecticut Valley varved clays exhibit normalized behavior over the stress range of interest (except perhaps for modulus) and samples from different locations also had the same NSP. Therefore, the NSP concept and test results from three varved clays (Amherst, Northampton, and East Windsor) are used for this study.

Appendix C presents details regarding the test results and testing conditions. The general testing procedures for triaxial, direct simple shear, and plane strain tests were presented in Section 3.5. The special method of consolidation used for

$\overline{CK_oU}$DSS(C) and $\overline{CK_oU}$DSS(E) tests is discussed in Section 5.3.4.

It is emphasized that, as a result of using the SHANSEP method of consolidation, these samples do not necessarily have the same soil structure as the in situ soil, as will be discussed in Chapter 6. In addition, the structure of isotropically consolidated laboratory specimens will obviously be different from that of the in situ soil.

Because the soil structure of SHANSEP samples is different from that of the in situ soil, the effect of stress systems, as shown from the studies presented in this chapter, may not truely reflect the in situ soil behavior. However, the SHANSEP method of consolidation was chosen because disturbance due to separations of the varve , especially with horizontal and inclined samples, could seriously affect the results, if the RECOMPRESSION method of consolidation had been used (see Chapter 6). In addition, the effect of material anisotropy component (defined in Chapter 2) could be mostly separated from that due to the structural and induced anisotropy (see Chapter 7), therefore helping to provide a basic understanding of anisotropic behavior of the varved clay. Finally, the results are applicable for a given soil structure even if this material does not necessarily have the same NSP as the in situ soil.

5.2 EFFECT OF K_o VERSUS ISOTROPIC CONSOLIDATION

5.2.1 Method of Investigation

Table 5.1 lists the experimental program that is used

for this study. For vertical loading, the effect of K_o consolidation is shown by comparing NSP from $\overline{CK_oUC}$ and $\overline{CIUC}$ tests, while $\overline{CK_oUE}$ versus $\overline{CIUE}$ tests show the effect for horizontal loading. The laboratory maximum past pressure $\bar{\sigma}_{vm}(L)$ was either 3.6 or 4 kg/cm^2, this ranging from 1.5 to 1.7 times the in situ $\bar{\sigma}_{vm}$.

5.2.2 Volume Change Data from $\overline{CIU}$ and $\overline{CK_oU}$ Tests

Figure 5.1 shows plots of volumetric strain, $\frac{\Delta V}{V_o}$, versus the vertical consolidation stress, $\bar{\sigma}_{vc}$, for samples from East Windsor. Data from $\overline{CIUC}$ and $\overline{CIUE}$ tests are compared with those from $\overline{CK_oUC}$ and $\overline{CK_oUE}$ tests. Figure 5.2 presents K_o versus OCR data from a K_o cell and Ladd's (1975) recommended relationship. Measured K_o values from $\overline{CK_oUC}$ and E tests agreed quite well with the recommended relationship. For simplicity, since K_o is nearly equal to unity at an OCR of 4, $\overline{CK_oU}$ tests with an OCR of 4 started shear with $K_c = 1.0$

It is seen in Fig. 5.1 that isotropically consolidated East Windsor samples compress only slightly more than K_o consolidated samples. Based upon these rather limited data, one can conclude that the stress system during consolidation (i.e., K_o versus isotropical consolidation) has little effect on the volume change during consolidation, at least in NC range. Since there was little variation in the natural moisture content (W_N) of the East Windsor samples, both $K_c = 1$ and K_o consolidated specimens have approximately the same water content prior to shear. Thus, water content per se is not a factor in comparing $\overline{CIU}$ versus

$\overline{CK_oU}$ test data.

However, the same final water content does not necessarily mean that $\overline{CIU}$ and $\overline{CK_oU}$ test samples have the same soil structure. One would expect K_o consolidation to retain and/or produce a more parallel orientation of particles perpendicular to the vertical direction. Hence, it is likely that the soil structure of these samples will be different. Moreover, this difference should still exist for overconsolidated samples of an OCR of 4 even though K_c is equal to 1.0 (Martin and Ladd, 1975).

5.2.3 Results from $\overline{CIUC}$ and $\overline{CK_oUC}$ Tests

Test results from $\overline{CK_oUC}$ and $\overline{CIUC}$ tests are summarized in Table 5.2 and Figs. 5.3 through 5.10. Before discussing the results, some discussion of "normalized effective stress envelope (NESE)," a plot in terms of $\bar{p}_f/\bar{\sigma}_{vm}$ versus $q_f/\bar{\sigma}_{vm}$ is in order.

1. In order to compare NC* and OC samples and to account for variation in $\bar{\sigma}_{vm}$, all results are divided by $\bar{\sigma}_{vm}$.
2. Results of triaxial tests are plotted for convenience in terms of q and $\bar{p}$ (or $q/\bar{\sigma}_{vm}$ vs. $\bar{p}/\bar{\sigma}_{vm}$) which yield an intercept $\bar{a}$ (or $\bar{a}/\bar{\sigma}_{vm}$) and a slope $\bar{\alpha}$.
3. In order to convert the above parameters to normalized Mohr-Coulomb (M-C) strength parameters, i.e., to plots of $\tau_{ff}/\bar{\sigma}_{vm}$ versus $\bar{\sigma}_{ff}/\bar{\sigma}_{vm}$ to give $\bar{c}/\bar{\sigma}_{vm}$ and $\bar{\phi}$, the following relationships were used.

*Note that $\bar{p}$ and q divided by $\bar{\sigma}_{vc}$ of all NC samples will end up at the same point in the plot of NESE if the soil exhibits normalized behavior.

$$\sin \phi = \tan \bar{\alpha} \qquad \tau_{ff}/\bar{\sigma}_{vm} = q_f/\bar{\sigma}_{vm} \cos \bar{\phi}$$

$$\bar{c}/\bar{\sigma}_{vm} = \bar{a}/\cos \phi \, \bar{\sigma}_{vm} \qquad \bar{\sigma}_{ff}/\bar{\sigma}_{vm} = \bar{p}_f/\bar{\sigma}_{vm} - q_f/\bar{\sigma}_{vm} \sin \bar{\phi}$$

These assume that the failure plane is given by the point of tangency between the normalized (with respect to $\bar{\sigma}_{vm}$) Mohr circle at failure and the normalized M-C envelope, whereas, in fact, the inclination of the actual failure plnae is not known.

4. Reference to Fig. 5.8 shows that there are several normalized envelopes when NC and OC samplesa are considered collectively. Note, however, that Line A represents the NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ for all samples and also at q_f when $\bar{p}_f/\bar{\sigma}_{vm}$ is less than about 0.3. These envelopes were used to obtain the corresponding values of $\bar{\phi}$ and $\bar{c}/\bar{\sigma}_{vm}$ except in the case of NC samples when $\bar{\phi}$ was computed from $\bar{\alpha}$ assuming $\bar{a}=\bar{c}=0$.
5. To avoid confusion, a NESE with a bar on top (i.e., $\overline{\text{NESE}}$) refers to Mohr-Coulomb normalized effective stress envelope (i.e., a plot of $\bar{\sigma}_{ff}/\bar{\sigma}_{vm}$ versus $\tau_{ff}/\bar{\sigma}_{vm}$), whereas the term NESE designates an envelope in terms of $\bar{p}_f/\bar{\sigma}_{vm}$ versus $q_f/\bar{\sigma}_{vm}$.

Data in Figs. 5.3 through 5.10 and the results in Table 5.2 show that K_o consolidation (relative to isotropic consolidation):

1. Did not significantly change $q_f/\bar{\sigma}_{vc}$ at OCR's of 1,2, and 4.

2. Did not significantly change $\tau_{ff}/\bar{\sigma}_{vc}$ at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ at an OCR of four, but decreased it at OCR's of 1 and 2 (Table 5.2).
3. Had little effect on A_f at q_f, but increased A at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ at low OCR (Table 5.2).
4. Significantly decreased ε_f at q_f (Fig. 5.6(a)).
5. Did not change ε_f at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ of samples at OCR's of one and two, but decreased ε_f of the sample with an OCR of four.
6. At low OCR, caused a significant lowering of the NESE at q_f (Fig. 5.8). At an OCR of one, $\bar{\phi}$ ($\bar{c}$=0) of NC sample decreases from 27° $\pm$ 1.5° to only 21° $\pm$ 0.7°. This reduction in NESE becomes less pronounced as the OCR approaches four.
7. Did not change the NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ except perhaps only slightly changing $\bar{\phi}$ ($\bar{c}$=0) for NC samples because of the difference in $\bar{p}_f$.
8. Increased the secant E_u/q_f at the same applied shear stress level ($\Delta q/\Delta q_f$) for OC samples (OCR=2 and 4), based on limited data, whereas K_o consolidated apparently did not significantly affect E_u/q_f for NC samples (Fig. 5.9).
9. Increased $E_u/\bar{\sigma}_{vc}$ at ε_a = 0.2% for OC samples and decreased $E_u/\bar{\sigma}_{vc}$ at the same strain with NC samples (Fig. 5.10).

The compensating effects of the stress system during

consolidation on the NESE at q_f and the pore pressure response caused $q_f(V)$ to be nearly identical. $\overline{CK_oUC}$ samples have a lower NESE at q_f (Fig. 5.8), but higher values of $\bar{p}_f$ than $\overline{CIUC}$ samples (Figs. 5.7 and 5.8). The decrease in NESE at q_f with K_o consolidation is consistent with data on other clays (Ladd, 1965).

The lowering of the NESE at q_f from $\overline{CK_oUC}$ tests might be explained by a difference in the degree of mobilization of friction and cohesion in the clay and silt portions. According to the concepts of Schmertmann and Osterberg (1960), the mobilization of cohesion and friction and a potential failure plane are strain dependent. The friction component only becomes fully mobilized at large strain, whereas the cohesion reaches its peak at low strains. Since $\overline{CIUC}$ test samples have a higher ε_f at q_f than $\overline{CK_oUC}$ test samples, the amount of friction mobilized at q_f is more in isotropically consolidated samples than in K_o consolidated samples at low OCR and hence K_o values.

At $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$, the NESE is hypothesized to represent the fully mobilized frictional strength of both clay and silt layers. The fact that, despite having different failure mechanisms ($\overline{CIUC}$ test samples failed by bulging while $\overline{CK_oUC}$ test samples failed by developing distinct failure planes across the varves at the end of the test), NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ of $\overline{CIUC}$ and $\overline{CK_oUC}$ test samples are identical (Fig. 5.8) suggests that in the undrained tests the frictional strengths of both clay and silt layers in the $\overline{CIUC}$ and $\overline{CK_oUC}$ samples are fully mobilized. It should be noted in Fig. 5.8 that the same NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$

also holds for both NC and OC samples from $\overline{CIUC}$ and $\overline{CK_oUC}$ tests. Consequently, NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ for vertical loading is independent of the stress history and stress system during consolidation.

At low OCR, the decrease in $\tau_{ff}/\bar{\sigma}_{vc}$ at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ with $\overline{CK_oUC}$ tests is due to the fact that $\overline{CK_oUC}$ samples exhibit more strain softening. This can probably be explained by the fact that distinct failure planes formed in the K_o consolidated samples

The "soil structure" effect on NSP might be inferred by comparing NSP of samples from $\overline{CIUC}$ and $\overline{CK_oUC}$ tests at an OCR of four, since in both tests shear started at the same stresses. Table 5.2 shows that $q_f(V)$ and A_f values of these samples are practically identical. These results, therefore, indicate little effect of soil structure on $q_f(V)$ and A_f. However, soil structure apparently does have an effect on the undrained modulus, as indicated in Fig. 5.9.

In conclusion, the stress system during consolidation does not significantly affect $q_f(V)$ and NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ for vertical loading, but it has an effect on the envelope at q_f and the stress strain and pore pressure characteristics.

5.2.4 Results from $\overline{CIUE}$ and $\overline{CK_oUE}$ Tests

Test results from the $\overline{CK_oUE}$ and $\overline{CIUE}$ tests are summarized in Table 5.3 and Figs. 5.5, 5.6(b), 5.7(b) and Figs. 5.10 to 5.13. These data show that K_o consolidation relative to isotropic consolidation:

1. Decreased $q_f/\bar{\sigma}_{vc}$ by about 15% practically independent of OCR (Table 5.3).
2. Decreased A_f at q_f and $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ of normally consolidated samples, but did not significantly change A_f of overconsolidated samples (OCR=2 and 4), shown in Fig. 5.5.b. Unlike the samples from triaxial compression tests, samples from $\overline{CIUE}$ and $\overline{CK_oUE}$ tests reached q_f and $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ condition at the same strain. Thus, for $\overline{CIUE}$ and $\overline{CK_oUE}$ test samples, A_f and NESE at q_f are identical to those at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$. However, $\bar{\phi}$ ($\bar{c}$=0) of NC sample is increased due to lower $\bar{p}_f/\bar{\sigma}_{vc}$ (Table 5.2).
3. Did not change ε_f at q_f or $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$. The ε_f is also practically independent of OCR (Fig. 5.6(b)),
4. Decreased $E_u/\bar{\sigma}_{vc}$ at the same strain of both NC and OC samples (Fig. 5.10).
5. Had little effect on E_u/q_f at OCR of one, but OC $\overline{CK_oUE}$ samples have lower E_u/q_f values than $\overline{CIUE}$ samples. This reduction increases with increasing OCR (Fig. 5.13).

The fact that samples from both $\overline{CIUE}$ and $\overline{CK_oUE}$ tests had distinct failure planes across the varves (though less visible in NC $\overline{CK_oUE}$ test sample) probably partly explains why the NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ or at q_f from $\overline{CIUE}$ and $\overline{CK_oUE}$ tests are identical. The lower $\bar{p}_f$ in the NC $\overline{CK_oUE}$ test sample relative to that in

the $\overline{CIUE}$ test sample explains why $\bar{\phi}$ ($\bar{c}$=0) of the NC $\overline{CK_oUE}$ test sample is larger, since NESE are identical.

Since NESE is not affected by the stress system during consolidation, the decrease in $q_f(H)/\bar{\sigma}_{vc}$ and $E_u/\bar{\sigma}_{vc}$ of $\overline{CK_oUE}$ test samples relative to those from $\overline{CIUE}$ test samples is caused by the development of larger excess pore pressures in $\overline{CK_oUE}$ test samples. These are two possible reasons to explain this different excess pore pressure at failure (Δu_f).

1. The difference in the magnitudes of Δq_f due to the rotation of principal planes: The principal planes are rotated by 90 degrees in $\overline{CK_oUE}$ test samples which have K_o less than one. In these samples, shearing first causes q to decrease to zero and then to increase in the horizontal direction. At OCR=1.0, when q = 0, the K_o consolidated sample has a much lower $\bar{p}$ than the isotropically consolidated sample (Fig. 5.7). Thus, for this NC $\overline{CK_oUE}$ sample, in spite of having a lower A_f, $\bar{p}_f$ is smaller.
2. A difference in soil structure: The $\overline{CK_oUE}$ test samples may have a more preferred particle orientation compared to that of the $\overline{CIUE}$ test samples, in spite of insignificant differences in the final water contents.

It is difficult to assess the magnitude of the effect of soil structure on the difference in Δq_f. However, the fact that the K_o consolidated sample developed larger pore pressure during

shear at OCR = 4 (K_o=1), suggests that there is a soil structure effect. That is, a more preferred particle orientation leads to an increase in the pore pressure developed in $\overline{CK_oU}E$ samples, i.e., when $\bar{\sigma}_{1f}$ acts in the horizontal direction. This hypothesis is supported by the results of Duncan and Seed (1966(a)) on Kaolin.

In summary, the decrease in $q_f/\bar{\sigma}_{vc}$ and $E_u/\bar{\sigma}_{vc}$ of NC $\overline{CK_oU}E$ samples are thought to be due to both the effects of the difference in Δq_f and in the soil structure, while the decrease in $q_f/\bar{\sigma}_{vc}$ and $E_u/\bar{\sigma}_{vc}$ of OC samples are mainly due to the differences in the soil structure.

Test data in this section, therefore, show that the stress system during consolidation has a considerable effect on $q_f/\bar{\sigma}_{vc}$, A_f, and $E_u/\bar{\sigma}_{vc}$ from horizontal loading. However, it has no effect on NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)$ (or at q_f) nor perhaps ε_f.

5.2.5 Summary and Discussion

A. Summary of Principal Findings from the Study of K_o Versus Isotropic Consolidation

(a) Behavior from Vertical Loading (Triaxial Compression)

Test data from $\overline{CIUC}$ and $\overline{CK_oUC}$ tests show that:

1. The stress system during consolidation has an insignificant effect on q_f at all OCR values studied, due to the compensating effect of K_o on developed pore pressure and NESE at q_f. The $\overline{CK_oUC}$ test samples ($1 \leq OCR < 4$) have a lower NESE at q_f, but have larger $\bar{p}_f$ values than the $\overline{CIUC}$ test samples.

At an OCR of four, $\overline{CK_oUC}$ and $\overline{CIUC}$ test samples have essentially the same q_f and A_f (also K_o=1).

2. The $\overline{CK_oUC}$ test samples, especially those at OCR of one and two, exhibit considerable more strain softening behavior than $\overline{CIUC}$ test samples. That is probably due to the formation of failure planes during shear in the $\overline{CK_oUC}$ test samples.
3. Because ε_f at q_f of $\overline{CK_oUC}$ test samples, especially that of the NC sample, is smaller than ε_f of $\overline{CIUC}$ test samples, NESE at q_f from $\overline{CK_oUC}$ tests is lower.
4. For both $\overline{CIUC}$ and $\overline{CK_oUC}$ test samples, the difference between NESE at q_f and $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ decreases with increasing OCR, as the difference in axial strains at q_f and $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ of both tests also decreases with increasing OCR.
5. Despite having different failure mechanisms ($\overline{CIUC}$ test samples having failed by bulging while ($\overline{CK_oUC}$ test samples developed distinct failure planes across the varves), NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ from $\overline{CK_oUC}$ and $\overline{CIUC}$ tests are identical.
6. E_u/q_f from OC $\overline{CIUC}$ test samples are lower than those from OC $\overline{CK_oUC}$ test samples, though E_u/q_f of NC $\overline{CIUC}$ and $\overline{CK_oUC}$ test samples are practically identical.

(b) Behavior from Horizontal Loading (Triaxial Extension)

Test data from $\overline{CK_oUE}$ and $\overline{CIUE}$ tests lead to the following findings:

1. $\overline{CK_oUE}$ test samples have about 15% lower q_f than

$\overline{CIUE}$ test samples at OCR = 1 to 4.

2. The decrease in q_f is due to the lower $\bar{p}_f$ (increased pore pressure), while the NESE is unchanged. For normally consolidated $\overline{CK_oU}$ test samples, the increase in Δu_f is thought to be due to both the increase in Δq_f and to differences in the soil structure. For overconsolidated samples, such a decrease is mainly due to the difference in the soil structure.
3. For both $\overline{CIUE}$ and $\overline{CK_oUE}$ tests, the q_f and $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ conditions occur at the same strain and they have the same failure mechanism. (Both tests develop a failure plane across the varves.) Thus, their NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ which are also the same as those at q_f are identical.

B. Uniqueness of NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$

It is seen in Figs. 5.7 and 5.8 that the same NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ holds for both NC and OC samples from $\overline{CIUC}$, $\overline{CK_oUC}$, $\overline{CIUE}$ and $\overline{CK_oUE}$ tests. Furthermore, upon comparing the stress systems of $\overline{CIUC}$, $\overline{CK_oUC}$, $\overline{CIUE}$ and $\overline{CK_oUE}$ tests, one can conclude that NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ for failure across the varves (since both TC and TE test samples have potential failure planes across the varves) is independent of:

1. The stress system during consolidation;
2. The magnitude of σ_2 and the rotation of principal planes (hence, the stress system during shear);
3. The magnitude of $\bar{\sigma}_{vm}$, ($\bar{\sigma}_{vm}$ of $\overline{CIU}$ tests is 4.0 kg/cm^2

while $\bar{\sigma}_{vm}$ is 3.6 kg/cm^2 for some of the $\overline{CK_oU}$ tests; additional data from RECOMPRESSION samples (presented in Chapter 6) which have $\bar{\sigma}_{vm}$ ranging from 1.6 to 2.4 also indicate this independency).

The fact that despite bulging type of failure observed at the end of the test, the NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ from $\overline{CIUC}$ tests are identical to that from $\overline{CK_oUC}$, $\overline{CIUE}$ and $\overline{CK_oUE}$ tests, in which their samples developed distinct failure plane across the varves at the end of the test, suggests that $\overline{CIUC}$ test samples also failed across the varves.

Additional data confirming the independency of the NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ from the magnitude of σ_2 from $\overline{CIUC}$ (θ=90) and $\overline{CIUE}$ (θ=90) tests on NC and OC samples and from a $\overline{CK_oUPSC}$ test and from two $\overline{CK_oUPSE}$ tests on NC soil will be presented in Section 5.3. The uniqueness of NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ suggests that it can be considered as a material property.

On the other hand, NESE at q_f, when it is not equal to that at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ (e.g., NESE at q_f for $\overline{CIUC}$ and $\overline{CK_oUC}$ tests when $\bar{p}_f/\bar{\sigma}_{vm} > 0.3$) can be greatly affected by the stress system during consolidation and the direction of loading.

5.3 EFFECT OF INTERMEDIATE PRINCIPAL STRESS

5.3.1 Method of Investigation

The investigation of the effect of σ_2 on NSP was conducted on both K_o and isotropically consolidated SHANSEP samples (Table 5.1).

The effect of σ_2 with $\overline{CIU}$ tests is shown by comparing NSP from $\overline{CIUC}$ and $\overline{CIUE}$ tests. $\overline{CIUC}$ tests on vertical samples (θ=0) and $\overline{CIUE}$ tests on horizontal samples (θ=90) are performed to study the effect for vertical loading. NSP from $\overline{CIUE}$ tests on veritical samples (θ=0) and those from $\overline{CIUC}$ tests on horizontal samples (θ=90) are compared for the case of horizontal loading. Figure 5.14 presents the state of stress during shear of these samples. It is seen in Fig. 5.14 that, when using horizontal triaxial samples, one of the σ_2 directions is perpendicular to the varves instead of being parallel as should occur in situ.

For $\overline{CK_oU}$ tests, the effect of σ_2 is shown by comparing NSP from triaxial and plane strain condition. NSP from a $\overline{CK_oUPSC}$ test on a normally consolidated sample from Northampton (Lacasse et al., 1972) and that from a $\overline{CK_oUC}$ test on a normally consolidated sample from Amherst are compared for the case of vertical loading. For horizontal loading, NSP from two $\overline{CK_oUPSE}$ tests (Lacasse et al., 1972) and those from a $\overline{CK_oUE}$ test are compared, all being normally consolidated. Since the DSS device is thought to have a state of stress during shear closer to that of plane strain (Ladd and Edger, 1972), it was also used for tests on two normally consolidated inclined (θ=45) samples. These $\overline{CK_oUDSS(C)}$ and $\overline{CK_oUDSS(E)}$ tests simulate $\overline{CK_oUPSC}$ and $\overline{CK_oUPSE}$ tests, respectively. Section 5.3.4 will present the testing procedures and results as well as the problems of interpreting test data from these special DSS tests.

5.3.2 Results from $\overline{CIU}$ Tests

Test results from $\overline{CIUC}$ and $\overline{CIUE}$ tests on vertical and hori-

zontal samples are summarized in Tables 5.4 and 5.5 and Figs. 5.15 to 5.19. These test results are from samples which have approximately the same moisture content. Thus, the differences in NSP from $\overline{CIUC}$ and $\overline{CIUE}$ test samples are mostly the result of the difference in the magnitudes of $\bar{\sigma}_2$ and perhaps also in the direction of σ_2 relative to the varve structure. Test data show that for vertical loading, shear in extension is relative to shear in compression:

1. Slightly increased (though considered insignificant for practical purposes) $q_f(V)/\bar{\sigma}_{vc}$ and $\tau_{ff}/\bar{\sigma}_{vc}$ at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ for both NC and OC samples;
2. Caused an increase in pore pressure parameter A for OC samples, but had no effect on A for NC soil (Fig. 5.19);
3. Increased NESE at q_f, though NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ is not affected (Fig. 5.17);
4. Increased $E_u/\bar{\sigma}_{vc}$ at $\varepsilon = 0.5\%$, though the maximum increase was less than 30% (Table 5.4);
5. Increased $\bar{\phi}$ ($\bar{c} = 0$) of NC samples (Table 5.4).

For horizontal loading, shear in extension relative to shear in compression:

1. Decreased $q_f(H)/\bar{\sigma}_{vc}$ and $\tau_{ff}/\bar{\sigma}_{vc}$ at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ by 5 to 15%, this effect also decreasing with increasing OCR (Table 5.5);
2. Increased the pore pressure and hence A (Fig. 5.19);
3. Increased $\bar{\phi}$ (assuming $\bar{c} = 0$) of NC soil (Table 5.5)

because of lower $\bar{p}_f/\bar{\sigma}_{vm}$;

4. Did not change NESE at q_f or $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ (same envelope), Figs. 5.17 and 5.19;
5. Slightly decreased $E_u/\bar{\sigma}_{vc}$ at $\varepsilon = 0.5\%$ but the maximum decrease is only 30% or less (Table 5.5).

An analysis of the effect of σ_2 on $E_u/\bar{\sigma}_{vc}$ can not be made without assuming that varved clay is an isotropic material. The vertical strains were measured in $\theta=0$ samples, whereas the "horizontal" strains (with respect to the in situ orientation) were measured in $\theta=90$ samples. Since the effect of σ_2 on $E_u/\bar{\sigma}_{vc}$ is shown by comparing $E_u/\bar{\sigma}_{vc}$ from $\overline{CIUC}$ ($\theta=0$) and $\overline{CIUE}$ ($\theta=90$) tests for vertical loading, and those from $\overline{CIUC}$ ($\theta=90$) and $\overline{CIUE}$ ($\theta=0$) tests for horizontal loading, it is necessary to assume that the material behaves as if it were isotropic. With this assumption, σ_2 has relatively little effect on $E_u/\bar{\sigma}_{vc}$ for both vertical and horizontal loading (Tables 5.4 and 5.5), the variation being within 30%. Though a difference of 30% is not considered significant (considering the limitation in the method of analysis), it can not be concluded that σ_2 has no effect on $E_u/\bar{\sigma}_{vc}$ because of the unknown direction of the error due to the assumption of isotropy.

The fact that the modulus from $\theta=90$ samples is always larger than for $\theta=0$ samples, independent of the type of test (i.e., when comparing TC ($\theta=90$) versus TE ($\theta=0$) tests or TC ($\theta=0$) versus TE ($\theta=90$) tests) suggests that the use of samples with different inclinations affects the outcome of the results at least as much as the value of the intermediate principal stress.

For an isotropic "ideal" material, interpretation of test data in terms of τ_{oct} and $\bar{\sigma}_{oct}$, rather than $\bar{p}$ and q, and use of pore pressure parameter, a, should account for the changes in the magnitude of σ_2. However, Fig. 5.18 shows that the $\tau_{oct}/(\bar{\sigma}_{oct})_m$ versus $\bar{\sigma}_{oct}/(\bar{\sigma}_{oct})_m$ [$(\bar{\sigma}_{oct})_m = \bar{\sigma}_{vm}$) from TC and TE tests are different. For both vertical and horizontal loading, the pore pressure parameter a from $\overline{CIUC}$ test samples is always greater than that from $\overline{CIUE}$ tests (Tables 5.4 and 5.5). Therefore, the increase in the magnitude of σ_2 decreases the pore pressure parameter a. The effect is more pronounced with vertical loading than with horizontal loading. The decrease in a is also larger in OC samples than in NC samples for both loading directions.

Figure 5.17 shows the NESE at q_f and $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ from the test series. These data and data in Fig. 5.7 show that NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ for failure across the varves is practically independent of: (1) the stress system during consolidation; (2) the intermediate principal stress; (3) rotation of principal planes; and (4) the magnitude of $\bar{\sigma}_{vm}$ (limited data only; more data being presented in Chapter 6).

It is stressed that the effects of the differences in the magnitudes of σ_2 and in the directions of $\Delta\sigma_2$ relative to the varve structure (perpendicular or parallel to the varves) shown from comparing TC and TE test data on vertical and horizontal samples can not be separated. In fact, sample orientation might be as important as the σ_2. Thus, no definite conclusion regarding σ_2 can be drawn from this test series.

5.3.3 Results from K_o Consolidated Plane Strain and Triaxial Tests

Figures 5.20 and 5.21 show normalized stress-strain curves of normally consolidated samples from $\overline{CK_oUC}$ and $\overline{CK_oUPSE}$ tests, and those from $\overline{CK_oUE}$ and $\overline{CK_oUPSE}$ tests respectively. The samples from the $\overline{CK_oUC}$ and $\overline{CK_oUPSC}$ tests reached the q_f condition at a very low strain and the $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ condition at a large strain (Fig. 5.20). The $\overline{CK_oUE}$ and $\overline{CK_oUPSE}$ tests, on the other hand, reached q_f and $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ simultaneously at much larger strains especially the TE test (Fig. 5.21). Normalized effective stress paths for these tests are plotted in Fig. 5.22. These data and the data in Table 5.6 and Fig. 5.23 show that shear in plane strain relative to shear in triaxial tests:

1. Slightly increased $q_f(V)/\bar{\sigma}_{vc}$ by 10%;
2. Increased $q_f(H)/\bar{\sigma}_{vc}$ by 20%;
3. Decreased A_f for both vertical and horizontal loading ;
4. Decreased the shear strain at failure (γ_f) computed from the linear elasticity theory (Table 5.6);
5. Increased $E_u/\bar{\sigma}_{vc}$ (Fig. 5.23);
6. Increased $\bar{\phi}$ (assuming $\bar{c} = 0$) at q_f for vertical loading (i.e., $\bar{\phi}$ PSC > $\bar{\phi}$ TC), but decreased $\bar{\phi}$ $\bar{c} = 0$) at q_f for horizontal loading because of larger $\bar{p}_f/\bar{\sigma}_{vm}$ (i.e., $\bar{\phi}$ TE > $\bar{\phi}$ PSE). For vertical loading, $\bar{\phi}$ at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ from $\overline{CK_oUC}$ and $\overline{CK_oUPSC}$ tests are almost identical.

It should be mentioned that because of the inherent problems with friction forces in the plane strain device, the results

from the plane strain tests may be questionable. Any frictional forces along the vertical boundaries of the rectangular specimen will affect the measured vertical force. Thus, the $q_f/\bar{\sigma}_{vc}$ and $E_u/\bar{\sigma}_{vc}$ measured from the plane strain test may be too high. In plane strain compression (PSC) tests, the errors in measuring $q_f/\bar{\sigma}_{vc}$ and perhaps $E_u/\bar{\sigma}_{vc}$ probably are small because the direction of the major principal stress during consolidation and shear are the same. Because of the difference in loading direction during consolidation and shear, the errors in measuring $q_f/\bar{\sigma}_{vc}$ and $E_u/\bar{\sigma}_{vc}$ from plane strain extension (PSE) tests will be more important.

Despite having friction problems in the apparatus, the increases in q_f from triaxial to plane strain tests for both vertical and horizontal loading agree with the other test data reviewed in Chapter 2. As shown in Table 2.7, data from Boston Blue clay (Ladd and Edger, 1972) show an increase in $q_f/\bar{\sigma}_{vc}$ from TE to PSE of about 20%. Data from Haney clay (Vaid and Campannella, 1974) also show the same increase of about 20%. Data from $\overline{CK_oU}$PSC and $\overline{CK_oU}$C on these two soils also show an increase in q_f from TC to PSC similar to that for the Connecticut Valley varved clay.

Regarding the measurement of undrained modulus, it is difficult, if not impossible, to make any assessment of the effect of friction forces. However, Boston Blue clay data obtained from using the same plane strain equipment show no practical difference in $E_u/\bar{\sigma}_{vc}$ from NC $\overline{CK_oU}$PSE and $\overline{CK_oU}$E tests, despite

having friction problems in the apparatus (Ladd et al., 1971). Since the increases in $E_u/\bar{\sigma}_{vc}$ (Fig. 5.23) from TC to PSC and from TE to PSE are very large, there may be a significant effect of σ_2 on $E_u/\bar{\sigma}_{vc}$, though probably not as large as measured.

The plane strain test sample also failed across the varves (though only the $\overline{CK_oU}$PSC test developed distinct failure planes). Thus, based on the result of Section 5.3.2, the NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ for failure across the varves (shown to be independent of the stress system during consolidation and of the magnitude of σ_2) shown in Fig. 5.17 should be identical to that from the plane strain tests (Fig. 5.22). Figure 5.22 shows the NESE at q_f and $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ conditions from $\overline{CK_oU}$C and $\overline{CK_oU}$E tests together with normalized stress paths from $\overline{CK_oU}$PSC and PSE tests on NC samples. This figure shows that:

1. The magnitude of σ_2 has little effect on NESE at q_f and no effect on NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ for vertical loading.
2. Test results from the $\overline{CK_oU}$PSE tests fall slightly below the NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ from $\overline{CK_oU}$E tests (also see Fig. 5.26) and from $\overline{CIU}$E (θ=0) and $\overline{CIU}$C (θ=90) tests. This might be due to the fact that the envelope at high $\bar{p}_f$ may not be a perfect straight line* and/or due to friction problems in the plane strain device.

*Note that data from $\overline{CIU}$C (θ=90) tests on NC samples also fall slightly below the envelope.

Since q_f values for OC plane strain tests are not available, they were estimated as follows. Figure 5.24 shows a plot of q_f at $\bar{\sigma}_{vc}/q_f$ at $\bar{\sigma}_{vm}$ (equal to SR shown in Fig. 5.24) versus OCR from several types of tests. It is seen that for the same loading direction (i.e., either horizontal or vertical loading), the ratio q_f at $\bar{\sigma}_{vc}/q_f$ at $\bar{\sigma}_{vm}$ is independent of the type of SHANSEP test. Therefore, q_f values for a particular loading direction at a given OCR for a plane strain test can be reasonably estimated by the relation given below.

$$q_f/\bar{\sigma}_{vc} = q_f/\bar{\sigma}_{vc} \text{ of the NC sample} \times SR \times OCR \qquad \text{(Eq. 5.3)}$$

Using values of SR for horizontal and vertical loading, Fig. 5.2.4 presents the calculated plane strain test $q_f/\bar{\sigma}_{vc}$ values from Eq. 5.3 and also the measured data from triaxial tests. Use of triaxial tests to estimate S_u will obviously lead to lower strengths than plane strain tests and thus presumably yield somewhat conservative results for field situations involving plane strain loading conditions.

If one wants to obtain the estimates of plane strain q_f values from triaxial tests, $\overline{CIUC}$ and $\overline{CIUE}$ tests on vertical samples seem to be the appropriate tests to perform. As shown below, the agreement in q_f between $\overline{CK_oUPSE}$ and $\overline{CIUE}$ tests is fairly good, while q_f from $\overline{CIUC}$ tests are about 10% lower than those from $\overline{CK_oUPSC}$ tests.

	$q_f/\bar{\sigma}_{vc}$ from Vertical Loading		$q_f/\bar{\sigma}_{vc}$ from Horizontal Loading	
OCR	$\overline{CK_oUPSC}$	$\overline{CIUC}$	$\overline{CK_oUPSE}$	$\overline{CIUE}$
1	0.28	0.245	.25	.25
2	0.47	0.42	.44	.46
4	0.76	0.68	.72	.75

Note, however. that this approach may not be equally applicable to all varved clay deposits.

Since NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ for failure across the varves is a material property (see Section 5.2.5 and later Section 5.3.4), it is believed that NESE at q_f which is the same as that at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ from $\overline{CK_oUE}$ tests should also hold for OC samples from $\overline{CK_oUPSE}$ tests. Furthermore, based on NC test data, NESE at q_f when $\bar{p}_f/\bar{\sigma}_{vm} > 0.3$ for NC and OC $\overline{CK_oUPSC}$ tests may be slightly above the NESE at q_f from $\overline{CK_oUC}$ tests (though by very little). The estimated values of $\bar{\phi}$ and $\bar{c}/\bar{\sigma}_{vm}$ for NESE at q_f from $\overline{CK_oUPSC}$ tests are 16° and 0.080 respectively (versus $\bar{\phi}$ = 14.5° and $\bar{c}/\bar{\sigma}_{vm}$ = 0.088 from $\overline{CK_oUC}$ tests). These NESE values will be used for the interpretation of anisotropic behavior in plane strain loading presented in Chapter 7.

In conclusion, the comparison of NC NSP from $\overline{CK_oU}$ plane strain and triaxial tests on Connecticut Valley varved clays generally indicates trends similar to those for other homogeneous clays.

5.3.4 Interpretation and Results from Special DSS Tests

Problems with the DSS Test

Since the state of stress of the sample in the DSS test is rather complex and only the normal and shear stresses on a horizontal plane are measured, the problems that must be considered in the analysis of DSS test results are:

1. The complete state of stress during shear is unknown;
2. The state of stress in the sample is not uniform.

A uniform stress distribution is expected in the middle of the samples, but high stress concentrations occur along the edges of the samples (Duncan and Dunlop, 1969; and Lucks et al., 1972).

Based on the above mentioned problems, uncertainty regarding the state of stress at failure is a major problem. Ladd and Edgers (1972) analyzed four possible assumptions which might be used for estimating the state of stress at failure.* These are obtained by assuming:

a. That the applied stress system is one of pur shear, i.e., $\Delta\sigma_1 = -\Delta\sigma_3 = \Delta\tau_h$.
b. That the horizontal plane is a failure surface, i.e., $\tau_h = \tau_{ff}$ and $\bar{\sigma}_{vf} = \bar{\sigma}_{ff}$.
c. That the horizontal shear stress is the maximum shear stress in the sample, i.e., $\tau_h = q_f$ and $\bar{\sigma}_{vf} = \bar{p}_f$.
d. The location of Mohr-Coulomb failure envelope. The smallest Mohr circle representing the state of stress at failure must then pass through τ_h and $\bar{\sigma}_f$ and be

*Arbitrarily defined either at $\tau_{h_{max}}$ or at $(\tau_h/\bar{\sigma}_v)_{max}$.

tangent to the assumed Mohr-Coulomb envelope.

Provided the failure envelope is known, assumption (d), where the location of Mohr-Coulomb failure envelope is used, is most reasonable. Based on the analyses performed on NC, CH and CL clays, using DSS ($\theta=0$) tests by Ladd and Edgers (1972), the pure shear assumption yields q_f values which are significantly larger than those from assumptions (b), (c) and (d). The pure shear assumption is only reasonable when the soil is assumed to behave like an elastic isotropic material (Duncan and Dunlop, 1969; and Lucks et al., 1972). Thus, this assumption is questionable and probably should only be used at low strains where the behavior of a soil may be nearly elastic.

Interpretation of the Special DSS Data

For this portion of the study, the effect of σ_2 on NSP ($q_f/\bar{\sigma}_{vc}$, A_f and $E_u/\bar{\sigma}_{vc}$) are shown by comparing NSP from special DSS, PS, and triaxial tests. NSP from $\overline{CK_oU}$C and $\overline{CK_oU}$PSC tests and from a $\overline{CK_oU}$DSS(C) test on a normally consolidated inclined sample are compared for the study of the effect of intermediate principal on stress-strain-strength behavior in the vertical loading. For horizontal loading, NSP from $\overline{CK_oU}$E, $\overline{CK_oU}$PSE and $\overline{CK_oU}$DSS(E) tests are compared.

The assumptions employed for the interpretation of special DSS test data are as follows:

(1) During consolidation: The normal and shear stresses are applied to an inclined sample so that "K_o consolidation"*

*True K_o consolidation can not be maintained because the stresses on the vertical sides of the inclined specimen can not be controlled.

is maintained during consolidation. Such a condition requires that the complimentary shear and normal stress be developed on the vertical sides of the inclined specimen. As a result, the consolidation shear stress on the inclined (α) plane, $\tau_{\alpha c}$ = $\bar{\sigma}_{vc}(1-K_o)\sin\theta\cos\theta$; and the consolidation normal stress on the inclined (α) plane, $\bar{\sigma}_{\alpha c}$, = $\bar{\sigma}_{vc}(\cos^2\theta+K_o \sin^2\theta)$; note that θ is 45° in this case.

(2) During Shear: Two separate assumptions are used, namely: the pure shear sssumption (i.e., assumption (a)) and that of the location of the failure envelope (assumption (d)). The first assumption was mainly used to illustrate that it yields unreasonable results.

For the pure shear assumption, the states of stress during consolidation and shear of $\overline{CK_oU}$DSS(C) and $\overline{CK_oU}$DSS(E) test samples are shown in Fig. 5.25. As a result of assumption (1), the origin of planes, i.e., the pole, of the Mohr circle representing the state of stress during consolidation is located at the point defined by the stresses $\tau_{\alpha c}$ and $\bar{\sigma}_{\alpha_c}$, since the inclination of the varves is 45°. This point (Fig. 5.25(a)) could also be defined as $\bar{p}_c = \frac{\bar{\sigma}_{vc} + \bar{\sigma}_{hc}}{2}$, and $q_c = \frac{\bar{\sigma}_{vc} - \bar{\sigma}_{hc}}{2}$. For a compression test, simulating a $\overline{CK_oU}$PSC condition, the sample is sheared by increasing τ_α. Figure 5.25(b) shows Mohr circles representing the states of stress prior to shear and those at failure. Upon assuming that the applied shear stress is that of pure shear, the origins of planes for the total stress circle (TSC) and the effective stress circle (ESC) at failure are located at a point defined by $p_f = p_c$ and q_f for the TSC and at a point defined by $\bar{p}_f$ and q_f for the ESC. Thus, $\bar{\sigma}_{1f}$ in the $\overline{CK_oU}$DSS(C) test acts perpendicular to the varves and the principal planes

are not rotated during shear. Therefore, based on these assumptions, a $\overline{CK_oU}$DSS test on an inclined sample should measure $q_f(V)$. In the $\overline{CK_oU}$DSS(E) test (Fig. 5.25(c)), the sample is sheared by reversing the direction of τ_α. As a result of using the pure shear assumption, the pole of the TSC moves from the point defined by p_c and q_c before shear, to the point defined by $p = p_f$ and $q = -q_f$ at failure. Thus, the principal planes in the $\overline{CK_oU}$DSS(E) test sample are rotated by 90° and $\bar{\sigma}_{1f}$ is in the direction parallel to the varves. Therefore, the $\overline{CK_oU}$DSS(E) test presumably measured $q_f(H)$. From the above discussion, it is seen that the varved inclination of 45° is selected so that $q_f(V)$ and $q_f(H)$ could be measured, assuming a pure shear state of stress during shear.

For analyses based on an assumed location of the failure envelope, a $\overline{NESE}$ for failure across the varves was selected (referred to as $\overline{NESE}$ assumption). Because of the uniqueness of NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ for failure across the varves (i.e., it is a material property), it can be used with reasonable confidence for the interpretation of $\overline{CK_oU}$PSS(E) test data on a NC inclined sample which reaches $(\tau_\alpha)_{max}$, the maximum shear stress on the inclined plane, and $(\tau_\alpha/\bar{\sigma}_{\alpha'})_{max}$ condition at the same strain. The experimental evidence supporting the uniqueness of NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ and thus $\overline{NESE}$ is given below.

All the test data [$\bar{p}_f/\bar{\sigma}_{vm}$ versus $q_f/\bar{\sigma}_{vm}$ at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$] previously presented in Sections 5.2 and 5.3 are plotted in Fig. 5.26. It also includes data from two stress-controlled, isotropically consolidated drained triaxial compression unloading

tests (CIDC(U)) on heavily OC RECOMPRESSION, Northampton samples, developing distinct failure planes across the varves observing at the end of the tests (Lacasse et al., 1972). Figure 5.26 shows the following.

1. Results from NC samples from $\overline{\text{CIUC}}$ (θ=0), $\overline{\text{CIUC}}$ (θ=90), $\overline{\text{CIUE}}$ (θ=90), $\overline{\text{CIUE}}$ (θ=0), $\overline{\text{CK}_o\text{UC}}$, $\overline{\text{CK}_o\text{UPSC}}$, $\overline{\text{CK}_o\text{UE}}$ and, to a less extent, $\overline{\text{CK}_o\text{UPSE}}$ tests lie on the same NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ for failure across the varves. Furthermore, the same $\overline{\text{NESE}}$ at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ holds for OC samples from the above types of tests.
2. Thus $\overline{\text{NESE}}$ at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ for failure across the varves is independent of the stress system during consolidation and during shear. This is one of the two necessary requirements; the other is the fact that the same $\overline{\text{NESE}}$ holds for both NC and OC samples.
3. Though data are limited, NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ for failure across the varves is also independent of drainage conditions (based on CIDC(U) tests) and the magnitude of $\bar{\sigma}_{vm}$ (see additional data in Chapter 6).

Since NESE at q_f when $\bar{p}_f/\bar{\sigma}_{vm} > 0.3$ from $\overline{\text{CK}_o\text{UC}}$ and $\overline{\text{CK}_o\text{UPSC}}$ tests are practically identical (and therefore, practically independent of the magnitude of $\Delta\sigma_2$), the $\overline{\text{NESE}}$ at q_f from $\overline{\text{CK}_o\text{UC}}$ tests is thought to hold for practical purposes for both NC and

OC triaxial and DSS samples. This NESE will be used* for the interpretation of DSS test data at $(\tau_\alpha)_{max}$ from the $\overline{CK_oUDSS(C)}$ tests. At $(\tau_\alpha/\bar{\sigma}_\alpha)_{max}$, NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ for failure across the varves will be used.

The suitability of the pure shear assumption can be evaluated by comparing, for both vertical and horizontal loading, $q_f/\bar{\sigma}_{vc}$ values estimated from the pure shear assumption to those from the $\overline{NESE}$ assumption and to those from the triaxial tests, keeping in mind that $q_f/\bar{\sigma}_{vc}$ from the $\overline{CK_oUE}$ test should be the lowest value (since $\Delta\sigma_1 = \Delta\sigma_2 > \Delta\sigma_3$ in TE tests).

Experimental Procedure

The only portion of the special DSS experimental procedure which differs from that presented in Section 3.5 is the method of consolidation. In this case, normal and shear stresses were applied to inclined samples. Upon using assumption (1) (the state of stress during consolidation), the magnitude of $\tau_{\alpha c}$ and $\bar{\sigma}_{\alpha c}$ were computed as follows:

$$\bar{\sigma}_{\alpha c} = \bar{\sigma}_{vc}(\cos^2\theta + K_o \sin^2\theta) \qquad \text{Eq. 5.4}$$

$$\tau_{\alpha c} = \bar{\sigma}_{vc}(1 - K_o) \sin\theta\cos\theta \qquad \text{Eq. 5.5}$$

For $\theta = 45°$; $\bar{\sigma}_{vc} = \bar{\sigma}_{\alpha c} + \tau_{\alpha c}$ Eq. 5.6 (Eq. 5.4+Eq. 5.5)

$$\bar{\sigma}_{hc} = \bar{\sigma}_{\alpha c} - \tau_{\alpha c}. \qquad \text{Eq. 5.7 (Eq. 5.4-Eq. 5.5)}$$

Since both $\bar{\sigma}_{vm}$ and K_o of East Windsor varved clay were known in advance, it is possible to consolidate so that the "assumed"

*Since the principal planes are not significantly rotated in the $\overline{CK_oUDSS(C)}$ test, the independence of $\overline{NESE}$ from the magnitude of $\Delta\sigma_2$ is the major requirement.

K_o consolidation could be maintained in every load step. For the purpose of preventing the sample from failing at low stresses, only $\bar{\sigma}_{\alpha c}$ was applied during the first three load increments (i.e., $\tau_{\alpha c} = 0$). $\tau_{\alpha c}$ is applied to the samples by using dead weight. Table 5.7 shows the load increments (and the computed values of $\bar{\sigma}_{vc}$ and $\bar{\sigma}_{hc}$) that were applied to the samples from the $\overline{CK_oU}$DSS(C) and $\overline{CK_oU}$DSS(E) tests. It should be noted that small load increments were used to prevent excessive deformation in the sample.

Experimental Results

The normalized stress-strain curves from $\overline{CK_oU}$DSS(C) and $\overline{CK_oU}$DSS(E) tests on normally consolidated inclined samples are shown in Fig. 5.27. The $\overline{CK_oU}$DSS(C) test sample reached $\tau_{\alpha max}$, presumably equivalent to the q_f condition, at a fairly low strain and $(\tau_\alpha/\bar{\sigma}_\alpha)_{max}$, presumably equivalent to the $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ condition, at a much larger strain. The $\overline{CK_oU}$DSS(E) test sample, on the other hand, reached the $\tau_{\alpha max}$ and $(\tau_\alpha/\bar{\sigma}_\alpha)_{max}$ condition at the same very large shear strain. Hence, the $\overline{CK_oU}$DSS(C) test behaved like a $\overline{CK_oU}$C test and the $\overline{CK_oU}$DSS(E) test had a behavior similar to that of a $\overline{CK_oU}$E test. Therefore, this similar behavior suggests that the uses of NESE at q_f for $(\tau_\alpha)_{max}$ condition and of NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ for $(\tau_\alpha/\bar{\sigma}_\alpha)_{max}$ are appropriate.

Figure 5.28 shows Mohr cirlces representing the state of stress at $q_f/\bar{\sigma}_{vc}$ from the two tests determined from the $\overline{NESE}$ assumption. Values of $q_f/\bar{\sigma}_{vc}$ and $\bar{p}_f/\bar{\sigma}_{vc}$ from $\overline{CK_oU}$C, $\overline{CK_oU}$E, $\overline{CK_oU}$PSC, and $\overline{CK_oU}$PSE are also plotted for comparison. The points defined by $q_f/\bar{\sigma}_{vc}$ and $\bar{p}_f/\bar{\sigma}_{vc}$ from $\overline{CK_oU}$C and $\overline{CK_oU}$E tests

tests fall almost exactly on the Mohr circles from the $\overline{CK_oU}$DSS(C) and $\overline{CK_oU}$DSS(E) tests respectively determined from $\overline{NESE}$ assumption.

The pore pressure parameter, A_f, for the DSS tests was estimated by assuming a value of K_o ($K_o = 0.68$) for normally consolidated samples. Table 5.8 shows how the values of A_f are obtained. Table 5.6 presents the comparisons of $q_f/\bar{\sigma}_{vc}$ (or $\tau_{ff}/\bar{\sigma}_{vc}$), γ_f, A_f, and $\bar{\phi}$ at the q_f and $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ condition from the triaxial, plane strain, and special DSS tests (determined from $\overline{NESE}$ assumption). These data show that for both vertical and horizontal loading:

1. $q_f/\bar{\sigma}_{vc}$ values from the DSS and triaxial tests are identical.
2. A_f at q_f and at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ from the DSS are roughly the same as those from triaxial tests, especially in the horizontal loading.
3. $q_f/\bar{\sigma}_{vc}$ values from the plane strain tests are higher than those from the DSS tests; A_f values are also lower.
4. $\bar{\phi}$ (assuming $\bar{c} = 0$) at q_f from the DSS and triaxial tests are equal (since $\overline{NESE}$ and q_f are identical). However, $\bar{\phi}$ from NC $\overline{CK_oU}$PSC test is larger than that from NC $\overline{CK_oU}$C test, and $\bar{\phi}$ from NC $\overline{CK_oU}$PSE test is lower than that from NC $\overline{CK_oU}$E test.

β angles determined from Fig. 5.28 are 9° from the $\overline{CK_oU}$DSS(C) test and 111° from the $\overline{CK_oU}$DSS(E) test. The small deviation in β from 0° and 90° respectively indicates that q_f values determined from $\overline{CK_oU}$DSS(C) and $\overline{CK_oU}$DSS(E) tests could logically be

compared with those from $\overline{CK_oU}$ triaxial and $\overline{CK_oU}$ plane strain test.

The agreement between the special DSS and $\overline{CK_oU}$ triaxial data for both vertical and horizontal loading does not mean that the states of stress in the DSS and triaxial test are identical and that the σ_2 has no effect on these soil parameters for the following two reasons:

1. $\bar{\sigma}_{vc}$ from the DSS test may not be correct. The complementary shear and normal stress cannot develop on the vertical sides of the specimen, especially at the edges of the specimen. Consequently, the "computed" $\bar{\sigma}_{vc}$ may be too large compared to that computed from considering no development of shear stress on the vertical sides of the sample, and thus, $q_f/\bar{\sigma}_{vc}$ "measured" from DSS tests might be too small.
2. The use of 45° varved inclination is somewhat approximate, though it generally yields a larger $(\tau_\alpha)_{max}$ compared to using $\theta = 45 + \bar{\phi}/2$ for vertical loading and using $\theta = 45 - \bar{\phi}/2$ for horizontal loading. (See data from two homogeneous clays reported in Chapter 2.)

As a result, the agreements between A_f and q_f from the special $\overline{CK_oU}$DSS and triaxial tests may be fortuitous.

For both vertical and horizontal loading, the decrease in $q_f/\bar{\sigma}_{vc}$ from $\overline{CK_oU}$ plane strain to special $\overline{CK_oU}$DSS test could partly be the result of:

1. $q_f/\bar{\sigma}_{vc}$ from the plane strain test, especially that from the $\overline{CK_oU}$PSE test, might be too high because of the friction problem in the plane strain device;
2. $q_f/\bar{\sigma}_{vc}$ measured from the special DSS tests is too low because of problems in the interpretation of DSS test data.

Mohr circles representing the state of stresses at q_f obtained from the pure shear assumption are compared to the previous results and shown in Fig. 5.29. Although $q_f/\bar{\sigma}_{vc}$ derived from the $\overline{NESE}$ assumption may be too low for both vertical and horizontal loading, $q_f/\bar{\sigma}_{vc}$ obtained from using the pure shear assumption is even lower. (Note that the same $\bar{\sigma}_{vc}$ was used for computing $q_f/\bar{\sigma}_{vc}$ values for both assumptions.) For horizontal loading, the pure shear assumption gives a $q_f/\bar{\sigma}_{vc}$ equal to 0.19 compared to 0.21 from the $\overline{CK_oU}$E test. Since the latter value represents the minimum strength, the pure shear assumption is not reasonable.

$E_u/\bar{\sigma}_{vc}$ values at various applied shear stress levels (the applied shear stress level being equal to $\Delta q/\Delta q_f$ for triaxial and plane strain tests and equal to $\Delta\tau_\alpha/(\Delta\tau_\alpha)_{max}$ for the special DSS test) from $\overline{CK_oU}$ triaxial, plane strain, and special DSS tests shown in Fig. 5.23 are based on the assumption that the soil is a linear isotropic elastic material. It is seen that only test data from $\overline{CK_oU}$E and $\overline{CK_oU}$DSS(E) tests show good agreement with respect to modulus. Such agreement, however, may be

fortuitous. For vertical loading, the special DSS test gave a much lower value of modulus than that from the triaxial compression test.

In summary, the major findings from this section are as follows:

1. The agreements in $q_f/\bar{\sigma}_{vc}$, A_f and $\bar{\phi}$ ($\bar{c}$=0) of NC soil from the $\overline{CK_oU}$ triaxial and DSS tests, for both vertical and horizontal loading, are probably fortuitous.
2. The use of $\overline{NESE}$, especially at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ (since it is a material property), for determining the state of stress in DSS tests is apparently fairly reasonable, at least compared to the pure shear assumption. In general, it is thought that $\overline{NESE}$ at q_f, if not equal to that at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$, is dependent upon the stress system. For the interpretation of the $\overline{CK_oU}$DSS(C) test, it is fortunate that NESE at q_f from $\overline{CK_oU}$ vertical loading tests appears to be little affected by the magnitude of $\Delta\sigma_2$ and that the same NESE at q_f holds for both NC and OC samples. (Thus, NESE at q_f, when $\bar{p}_f/\bar{\sigma}_{vm} > 0.3$ from $\overline{CK_oU}$C tests is thought to hold for NC $\overline{CK_oU}$DSS(C) test samples.) These requirements are sufficient for an interpretation of $\overline{CK_oU}$DSS(C) test data, since the principal planes are not expected to be significantly rotated.

5.3.5 Summary and Discussion

Tables 5.9 and 5.10 summarize the general effect of $\Delta\sigma_2$ presented in Sections 5.3.2, 5.3.3, and 5.3.4. The important points that need to be further mentioned are as follows:

1. For both vertical and horizontal loading, the agreement of $q_f/\bar{\sigma}_{vc}$ and A_f from $\overline{CK_oU}$ triaxial and special DSS (θ=45°) tests are probably fortuitous. This is due mostly to inherent problems in interpreting DSS test data, especially those arising from the assumption for computing $\bar{\sigma}_{vc}$.
2. The difference in the values of $q_f/\bar{\sigma}_{vc}$ from the $\overline{CK_oUPSE}$ and $\overline{CK_oUE}$ tests on NC soil is somewhat amplified by inherent problems (friction forces) in the plane strain device. This error is, however, less significant when test data from $\overline{CK_oUC}$ and $\overline{CK_oUPSC}$ tests are compared. Despite the friction problem in the plane strain device, for both vertical and horizontal loading, the trends observed from the plane strain and triaxial tests are similar to those observed from other homogeneous clays (see Chapter 2), i.e., the effect of σ_2 is more pronounced in the extension than in the compression test.
3. The behavior observed from the $\overline{CIU}$ tests is probably not mainly due to effect of the intermediate principal stress. Because of using horizontal (θ=90) samples, the effect of inherent anisotropy (defined

in Chapter 2) is included. One of the directions of $\Delta\sigma_2$ acting on the horizontal sample is perpendicular to the varves instead of being, in every direction, parallel to the varves, as for the vertical sample. Although for both vertical and horizontal loading, the trends of q_f from NC $\overline{CIU}$ tests are similar to those observed from comparing $\overline{CK_oU}$ plane strain and triaxial test data, these trends might equally be explained by using different sample orientations.

5.4 EFFECT OF TESTING METHOD FOR SHEARING PARALLEL TO VARVES

5.4.1 Introduction and Method of Investigation

For shearing across the varves, the effects of the stress system during consolidation and the magnitude of the intermediate principal stress on NSP were studied in Sections 5.2 and 5.3. In this section, the effect of stress system on NSP for shearing parallel to the varves is studied. The influence is shown by comparing results from two types of tests, namely: $\overline{CIUC}$ tests on inclined samples and $\overline{CK_oU}$DSS tests on vertical samples.

The stress system differences between $\overline{CIUC}$ and $\overline{CK_oU}$DSS tests are as follows:

1. For the stress system during consolidation, the samples from $\overline{CIUC}$ tests are isotropically consolidated, while those from DSS tests are K_o consolidated.

(Note that one direction of the consolidation stress ($\bar{\sigma}_c$) in the $\overline{CIUC}$ test acts at 45° to the varve structure, while $\bar{\sigma}_{hc}$ in the $\overline{CK_oU}$DSS test always acts parallel to the varve structure.)

2. For the stress system during shear, $\overline{CIUC}$ test samples are sheared in triaxial compression, i.e., $\Delta\sigma_1 > \Delta\sigma_2 = \Delta\sigma_3$. In the DSS test, the state of stress during shear is close to that of the plane strain, i.e., $\Delta\sigma_1 > \Delta\sigma_2 > \Delta\sigma_3$, and principal planes are continuously rotated during shear

Any difference between NSP obtained from DSS and $\overline{CIUC}$ tests (besides the inherent problem of interpreting DSS test data) could be the result of:

1. The influence of initial shear stress (K_o consolidation);
2. The effect of intermediate principal stress;
3. The effect of rotation of principal planes; and
4. The difference in the relative direction of σ_2 with respect to the varve structure. One of the $\Delta\sigma_2$ directions in the $\overline{CIUC}$ tests on inclined samples is 45° to the varve structure instead of parallel to the varves as in the DSS test on vertical samples.

These factors could affect q_f though their effects on the pore pressure response and/or on the failure envelope.

The direct simple shear test has a problem with respect to interpretation of the data. However, the study in Section 5.3.4 showed that a $\overline{NESE}$ could be used to determine with reasonable

accuracy the state of stress at failure in DSS tests. Thus, value of NESE at q_f and $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ for failure parallel to the varves are needed (since DSS samples at OCR's between 1 and 4 do not reach $\tau_{h_{max}}$ and $(\tau_h/\bar{\sigma}_v)_{max}$ at the same strain). The correct selection of a $\overline{NESE}$ requires that it be a material property. Based on the undrained test results from shearing across the varves, this is only true for the NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ or for NESE at q_f when it is identical to that at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$. NESE at q_f, when it is not identical to that at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$, is independent upon the stress system. Therefore, the location of the NESE at q_f for a conventional DSS test is not known for sure. For interpreting thest data at $\tau_{h_{max}}$, the $\overline{NESE}$ at q_f for the appropriate range of $\bar{p}_f/\bar{\sigma}_{vm}$ from $\overline{CIUC}$ (θ=45) tests is first used. The sensitivity of q_f to variations in the NESE is then studied.

For interpretation of DSS test data at $(\tau_h/\bar{\sigma}_v)_{max}$, the uniqueness of NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ for failure parallel to the varves should be established. For this purpose, NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ from CDDS tests (the envelope being determined assuming $\tau_h=\tau_{ff}$) and from $\overline{CIUC}$ tests on inclined samples are compared. It can probably be assumed that, apart from the difference in the drainage conditions, the stress system in a CDDS test is similar to that in DSS tests, though the latter has a more uniform stress distribution.

5.4.2 Results from CDDS and $\overline{CIUC}$ Tests

NSP from $\overline{CIUC}$ tests are summarized in Table 5.11. The test data from CDDS tests on samples from Northampton (Lacasse et al., 1972) where failures take place in the clay layer are

presented in Table 5.12. q_f and $\bar{p}_f$ were computed by assuming that the horizontal plane in the direct shear test is the failure plane. Consequently, q_f is equal to $\tau_h/\cos\bar{\phi}$ and $\bar{p}_f$ is equal to $\bar{\sigma}_{vc} + \tau_h \tan\bar{\phi}$. The $\bar{\phi}$ value in Table 5.12 are those reported in Lacasse et al. (1972).

For shearing parallel to the varves, the samples could fail either in a clay or a silt layer. However, for the range of OCR's under study ($1 \leq OCR \leq 20$), observations of $\overline{CIUC}$ ($\theta=45$) test samples at the end of the tests indicated that the failure in these samples took place within a clay layer. (Hence, the failures in the DSS samples are also assumed to be within the clay layer.) Consequently, the NESE from CDDS test samples which have failure within the clay layer is compared to that from $\overline{CIUC}$ tests.

Figure 5.30 shows the NESE at q_f from $\overline{CIUC}$ tests and NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ from CDDS and $\overline{CIUC}$ tests. Figure 5.31 shows NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ of both NC and OC samples from CDDS and $\overline{CIUC}$ ($\theta=45$) tests at high stress. It can be seen that NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ from these tests are identical (Fig. 5.31). Thus, provided the assumed state of stress at failure in the CDDS test is correct, NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ (Fig. 5.31) for failure within the clay portion of both OC and NC samples is independent of:

1. The stress system during consolidation;
2. The stress system during shear;
3. The drainage conditions;
4. The magnitude of $\bar{\sigma}_{vm}$ (samples from CDDS tests have $\bar{\sigma}_{vm}$ different among themselves and from $\overline{CIUC}$ ($\theta=45$)

test samples).

Consequently, it is a material property. Therefore, it will be used to calculate the state of stress at $(\tau_h/\bar{\sigma}_v)_{max}$ in direct simple shear tests.

For interpreting the DSS test data at $\tau_{h_{max}}$, both NC and OC, the NESE obtained from $\overline{CIUC}$ (θ=45) tests at OCR's equal to 1 and 4 will first be used. This is line D in Fig. 5.30. It is assumed to be linear for $\bar{p}_f/\bar{\sigma}_{vm} \geq 0.3$, based on data from $\overline{CIUC}$ (θ=0) tests at OCR = 1,2, and 4 (Fig. 5.8).

5.4.3 Results from $\overline{CK_oU}$DSS Tests and Their Interpretation

Three representative normalized stress-strain curves from $\overline{CK_oU}$DSS tests having OCR's of one, two, and four, are shown in Fig. 5.32. The determination of the state of stress at $\tau_{h_{max}}$ of these samples is shown in Figs. 5.33, 5.34, and 5.35. Mohr-circles representing the state of stress at q_f of inclined samples from $\overline{CIUC}$ tests are also shown for comparison. The corresponding plot at $(\tau_h/\bar{\sigma}_v)_{max}$ are shown in Fig. 5.36. Table 5.13 presents a summary of the results from Figs. 5.33 to 5.36. These results show that:

1. At $\tau_{h_{max}}$, as the OCR increases, $\tau_{h_{max}}/\tau_{ff}$ increases from 0.9 to unity at an OCR of four;
2. At $(\tau_h/\bar{\sigma}_v)_{max}$, τ_{ff} is equal to τ_h;
3. The horizontal plane in the DSS test is not a failure plane (this being true at both the q_f and $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ conditions);
4. At $\tau_{h_{max}}$, β increases with increasing OCR from about 30° to 37° versus 45° if $\tau_h = q_f$; and

5. At $(\tau_h/\bar{\sigma}_v)_{max}$, β is approximately constant ($37° \pm 2°$) and is thus independent of OCR.

The angle β corresponds to the computed rotation of principal stresses. Since the principal stresses are continuously rotating during shear, the amount of rotation will depend upon the magnitude of shear strain. Hence, samples that fail at larger strains should have a larger β. This only applies to the $\tau_{h_{max}}$ condition where in γ_f increases with increasing OCR (Fig. 5.27). Consequently, β follows the same general trend. At $(\tau_h/\bar{\sigma}_v)_{max}$, γ is quite variable, but also very large and thus OCR has little effect on β.

The sensitivity of q_f values to the location of $\overline{NESE}$ is evaluated and presented in Fig. 5.38. Three locations of $\overline{NESE}$ (arbitrarily selected X and Y $\overline{NESE}$ and that from assuming a horizontal failure plane) were used to compute $q_f/\bar{\sigma}_{vc}$, $\tau_{ff}/\bar{\sigma}_{vc}$ and β for DSS samples at OCR's of one and two. These computed values are compared with those from using the $\overline{CIUC}$ (θ=45) $\overline{NESE}$, also presented in Fig. 5.38. As shown in Fig. 5.38, it is seen that:

1. The differences in q_f are within 15%, the lowest value of q_f being computed from the lower limit of $\overline{NESE}$ (i.e., that assuming a horizontal failure plane). q_f values from $\overline{NESE}$ X and Y are within 10% of those determined from the $\overline{CIUC}$ (θ=45) $\overline{NESE}$.
2. The values of β vary with different assumed states of stress in the DSS test. The fact that $\bar{\phi}(\bar{c}=0)$ for

NC soil determined from this horizontal failure plane assumption is unrealistically low ($\bar{\phi}$=15°) and that the computed β value is significantly different from the X, Y, and $\overline{CIUC}$ (θ=45) $\overline{NESE}$ shows that the use of a horizontal failure plane assumption is not suitable.

3. The small variations in q_f and β determined from using $\overline{NESE}$ X and $\overline{NESE}$ Y suggest that the use of $\overline{NESE}$ from $\overline{CIUC}$ (θ=45) test data for interpreting the DSS test is suitable and much better than the use of horizontal failure plane assumption.

In general, τ_h is not equal to either τ_{ff} or q_f. Ladd and Edgers (1972) concluded that they would expect $\tau_{ff} \leq \tau_{h_{max}} \leq q_f$. Based on the results presented in Table 5.13, $\tau_{h_{max}}$ can be up to 12% less than τ_{ff} for samples which have OCR's of 1 and 2, and is approximately equal to τ_{ff} at an OCR of four. These data suggest that the use of $\tau_{h_{max}}$ as the value of τ_{ff} in an undrained total stress analysis will be conservative, especially at low OCR values.

In summary, the use of $\overline{NESE}$ from $\overline{CIUC}$ (θ=45) tests leads to a reasonable interpretation of DSS data at the $\tau_{h_{max}}$ and $(\tau_h/\bar{\sigma}_v)$ condition. $\tau_{h_{max}}$ values of samples at low OCR are generally lower than τ_{ff}. As the OCR increases, the ratio $\tau_{h_{max}}/\tau_{ff}$ approaches unity.

5.4.4 Comparison of Results from $\overline{CIUC}$ (θ=45) and $\overline{CK_oU}$DSS Tests

Table 5.14 presents a summary of NSP obtained from $\overline{CIUC}$

tests on inclined samples and $\overline{CK_oU}$DSS tests on vertical samples. q_f and A_f from DSS tests were determined from using $\overline{NESE}$ at q_f from $\overline{CIUC}$ (θ=45°) tests. The results in Table 5.14 show that shearing in a triaxial condition relative to a DSS condition:

1. Increased $q_f/\bar{\sigma}_{vc}$ by about 10 and 20% at OCR's of one to four, respectively;
2. Increased $\tau_{ff}/\bar{\sigma}_{vc}$ at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ by about 20±5% at both OCR;
3. Increased A_f at both q_f and $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ for normally consolidated soil, but decreased them at OCR = 4.

As previously stated, the differences between NSP from $\overline{CIUC}$ (θ=45) and $\overline{CK_oU}$DSS tests could be the result of several factors. Based on the comparison of the stress systems in these tests, it can be hypothesized that such differences are due to the rotation of principal planes, the intermediate principal stress, and the initial shear stress effects, as well as inherent problems due to the interpretation of DSS test data.

In summary, the use of SHANSEP $\overline{CIUC}$ (θ=45) test samples for measuring the in situ q_f for inclined loading will not be conservative. The analyses presented in this section also show that the use of $\overline{NESE}$ at q_f from $\overline{CIUC}$ (θ=45) tests leads to a reasonable interpretation of DSS test data, at least better than using the assumption that the horizontal plane is the failure plane. Because it is a material property, $\overline{NESE}$ at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ for failure within the clay layer can be used

with confidence for interpreting DSS test data when the $\tau_{h_{max}}$ and $(\tau_h/\bar{\sigma}_v)_{max}$ conditions occur at the same shear strain (e.g., in heavily OC SHANSEP samples or OC RECOMPRESSION samples.

5.5 GENERAL DISCUSSION

One outcome from the studies of the effect of stress system on NSP is the opportunity to investigate the possibility of using a simplified testing method for evaluating anisotropic S_u values for total stress analyses. The $\overline{CIUC}$ test is the simplest test and could be performed on vertical, inclined, and horizontal samples. Therefore, data from $\overline{CK_oUC}$, $\overline{CK_oUPSC}$, $\overline{CK_oUDSS}$, $\overline{CK_oUPSE}$, and $\overline{CK_oUE}$ tests will be compared with those obtained from $\overline{CIUC}$ tests on samples with different varve inclinations.

Figure 5.39 shows plots of $q_f/\bar{\sigma}_{vc}$ versus OCR obtained from these tests. The data from $\overline{CIUE}$ tests are also included for the sake of completeness. It can be seen that the stress system during consolidation and the intermediate principal stress have practically no effect on $q_f(V)$. Therefore, $\overline{CIUC}$ tests on vertical samples could be used in practice for measuring q_f. (q_f from $\overline{CIUC}$ tests are about 10% lower than those from $\overline{CK_oUPSC}$ tests.)

$\overline{CIUC}$ tests on inclined and horizontal samples, however, can not be used in place of $\overline{CK_oUDSS}$ and $\overline{CK_oUE}$ or $\overline{CK_oUPSE}$ tests, since q_f values from inclined and horizontal loading are significantly affected by the stress system and/or sample

orientation. The combined effects cause q_f(H) from $\overline{CIUC}$ (θ=90) tests to be up to 35% higher than those from $\overline{CK_oUE}$ (θ=0) tests and about 17% higher than those from $\overline{CK_oUPSE}$ tests. The differences are rather large and, hence, $\overline{CIUC}$ (θ=90) tests are not suitable to use in practice for measuring the horizontal undrained shear strength. For inclined loading, q_f from $\overline{CIUC}$ (θ=45) tests can be 10 to 20% higher than that from DSS tests (Fig. 5.39). Such a difference is also somewhat too large. Thus, in practice, $\overline{CIUC}$ tests on inclined samples overestimate the strength for the measurement of S_u with inclined failure surface.

The above conclusions are based on the test results from SHANSEP samples. For RECOMPRESSION samples (see data in Chapter 6), $\overline{CIUC}$ tests yield a reasonable value of q_f(V), but q_f(H) is too low at the in situ OCR of 4 because of disturbance due to separation of the varves. On the other hand, q_f(45°) from a RECOMPRESSION $\overline{CIUC}$ test is practically equal to q_f from SHANSEP $\overline{CK_oUDSS}$ test, though this may be fortuitous.

5.6 SUMMARY AND CONCLUSIONS

The principal findings from the study of the effect of stress system on NSP with different loading directions using the SHANSEP method of consolidation are as follows:

1. For vertical loading, the stress system during consolidation and the intermediate principal stress have little effect on q_f(V). However,

they affect A_f, γ_f, $E_u/\bar{\sigma}_{vc}$, $\bar{\phi}$ ($\bar{c}$=0) of NC samples, and generally NESE at q_f, though NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ is not affected. (See Section 5.2.5 for a detailed summary of the effect of stress system during consolidation and Table 5.9 for the effect of σ_2.)

2. For horizontal loading, the stress system during consolidation and the intermediate principal stress affect q_f, A_f, γ_f, $E_u/\bar{\sigma}_{vc}$, $\bar{\phi}$ ($\bar{c}$=0) of NC samples, but have no effect on NESE at q_f (same envelope as that at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$). (See Section 5.2.5 for a detailed summary of the effect of the stress system during consolidation and Table 5.10 for the effect of σ_2.)
3. Limited data show that the stress systems during consolidation and shear affect NSP from inclined loading more than from vertical loading.
4. The stress system during consolidation and the intermediate principal stress have more effect on the horizontal undrained shear strength behavior than on the vertical undrained shear strength behavior.
5. For vertical and horizontal loading, the NC test data from $\overline{CK_oU}$ triaxial and plane strain tests, show trends similar to those observed with homogeneous natural clays (reviewed in Chapter 2). Due to inherent problems in interpreting DSS test data, especially the assumption used for estimating

$\bar{\sigma}_{vc}$ value, q_f, A_f, and $\bar{\phi}$ ($\bar{c}$=0) for NC samples from the special DSS tests on inclined samples may be fortuitously identical to those from $\overline{CK_oU}$ triaxial tests.

6. The use of strength data from $\overline{CK_oU}$ triaxial tests rather than plane strain tests for the evaluation of aniostropic S_u values for total stress stability analyses in plane strain loading should be conservative. Also, the use of $\tau_{h_{max}}$ from DSS (θ=0) tests as τ_{ff} seems to be conservative as well.
7. $\overline{CIUC}$ tests on samples cut at θ=45° and 90° will yield unconservative values of q_f for undrained total stress stability analyses, though q_f(V) from $\overline{CIUC}$ tests are slightly lower than those from $\overline{CK_oU}$PSC tests.
8. Reasonable estimates of q_f(V) and q_f(H) for PS conditions can be obtained from $\overline{CIUC}$ and $\overline{CIUE}$ tests on vertical samples.

The results in this chapter also provide insight into the basic understanding of the anisotropic behavior of varved clay, as presented below.

1. There are four possible failure mechanisms in varved clays, namely: failure by developing distinct failure planes across the varves, failure by bulging, failure within the clay layer, and failure within the silt layer.

2. Despite failure by bulging in $\overline{CIUC}$ (θ=0) test samples, their NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ is identical to that from $\overline{CK_oUC}$ test samples which develop distinct failure planes across the varves. This indicates that the basic failure mechanism (i.e., failure across the varves) is the same even though bulging occurs.

3. The NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ for failure across the varves and for failure within the clay layer are shown to be independent of:

   a. The stress system during consolidation;

   b. The stress system during shear;

   c. The magnitude of $\bar{\sigma}_{vm}$ (see additional data in Chapter 6);

   d. The drainage conditions (limited data); and

   e. The OCR of the sample.

Therefore, these envelopes are considered to be material properties. The values of $\bar{\phi}$ and $\bar{c}/\bar{\sigma}_{vm}$ of these envelopes are given below:

Type of Failure	$\bar{\phi}°$*	$\bar{c}/\bar{\sigma}_{vm}$
Across the varves	30	0.017
Within the clay layer	22	0.027

5. The NESE at q_f, if not equal to that at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ (e.g., for vertical and inclined loading), is dependent upon the stress system. For vertical

*Note that $\bar{\phi}$ for failure across the varves is much larger than that for failure within the clay layer.

loading, the NESE at q_f, when $\bar{p}_f/\bar{\sigma}_{vm} > 0.3$ from $\overline{CK_oUC}$ tests, is lower than that from $\overline{CIUC}$ tests (Fig. 5.8).

Results in Sections 5.3.4 and 5.4.3 suggest that appropriate values of NESE at q_f and $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ can be used to interpret the results of DSS tests, and that is much better than using the pure shear assumption or assuming a horizontal failure plane.

EXPERIMENTAL PROGRAM FOR INVESTIGATION THE EFFECT OF STRESS SYSTEM ON SHANSEP NSP

Stress in kg/cm^2

Variable	Type of Test	θ°	Soil Location[1]	OCR	$\bar{\sigma}_{vm}$(L)
K_o versus Isotropic Consolidation	$\overline{CIUC}$	0	E,A[2]	1,2,4	4.0
	$\overline{CK_oUC}$	0	E,A[2]	1,2,4	3.6 to 4.0
	$\overline{CIUE}$	0	E,A[2]	1,2,4	4.0
	$\overline{CK_oUE}$	0	E,A[2]	1,2,4	3.6
Intermediate Principal Stress	$\overline{CIUC}$	0	E	1,2,4	4.0
	$\overline{CIUE}$	90	E	1,2,4	4.0
	$\overline{CIUC}$	90	E	1,2,4	4.0
	$\overline{CIUE}$	0	E	1,2,4	4.0
	$\overline{CK_oUC}$	0	A	1	
	$\overline{CK_oUPSC}$	0	N	1	2.99 to 4.0
	$\overline{CK_oUDSS(C)}$	45	A	1	
	$\overline{CK_oUE}$	0	E	1	
	$\overline{CK_oUPSE}$	0	N	1	3.6 to 4.0
	$\overline{CK_oUDSS(E)}$	45	E	1	
Effect of Testing Method for Shearing Parallel to the Varves	$\overline{CIUC}$	45	E	1,4,20	4.0
	$\overline{CK_oUDSS}$	0	A,N,E	1,2,4	3.0 to 4.0
	CDDS	0	N	1,1.83,3.4	3.4 to 6.0

Notes: (1) E = East Windsor; A = Amherst; N = Northampton
(2) Only soil from East Windsor that are tested at indicated OCR.

Table 5.1

COMPARISON OF NSP FROM $\overline{CIUC}$ AND $\overline{CK_oUC}$ TESTS ON CONNECTICUT VALLEY VARVED CLAY (ONLY EAST WINDSOR AND AMHERST DATA)

Type of Test	OCR	Data at q_f					Data at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ (1)				
		$q_f/\bar{\sigma}_{vc}$	A_f	ε_f	$\bar{\phi}°$ (2)	$\bar{c}/\bar{\sigma}_{vm}$ (2)	A_f	ε_f	$\bar{\phi}°$ (1)	$\bar{c}/\bar{\sigma}_{vm}$ (1)	$\tau_{ff}/\bar{\sigma}_{vc}$
$\overline{CK_oUC}$	1	0.25±0.005	1.40±0.25	0.85±0.20	21.1±0.7	0	3.74	15.5	31.7	0	0.18
$\overline{CIUC}$		0.245±0.005	1.41± .02	4.7 ±0.3	27 ±1.5	0	1.51	16.8	33.0	0	0.21
$\overline{CK_oUC}$	2	0.40±0.005	0.50±0.05	2.8 ±0.04	14.5	0.088	0.94	12.6	30.0	0.017	0.31
$\overline{CIUC}$		0.42	0.68	7.4	22.5	0.060	0.74	12.2	30.0	0.017	0.36
$\overline{CK_oUC}$	4	0.67±0.01	0.25±0.03	7.3 ±1.0	14.5	0.088	0.26±0.03	7.3±1.0	30.0	0.017	0.61
$\overline{CIUC}$		0.68	0.24	11.2	22.5	0.060	0.26	13.6	30.0	0.017	0.60

Note: (1) Only data at OCR's of one and two from Amherst reached $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$.

(2) $\bar{\phi}$ for NC sample was computed from assuming $\bar{c} = 0$.
$\bar{\phi}$ and $\bar{c}/\bar{\sigma}_{vm}$ are normalized Mohr-Coulomb effective strength parameters.

Table 5.2

COMPARISON OF NSP FROM $\overline{CIUE}$ AND $\overline{CK_oUE}$ TESTS ON CONNECTICUT VALLEY VARVED CLAY

Test No.	Type of Test	OCR	$\frac{q_f(H)}{\bar{\sigma}_{vc}}$	τ_{ff} at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$	A_f		ϵ_f, %		$\bar{\phi}°$	$\frac{\bar{c}}{\bar{\sigma}_{vm}}$	$\bar{\phi}°$	$\frac{\bar{c}}{\bar{\sigma}_{vm}}$	Type of Failure
					$(\sigma_1-\sigma_3)_{max}$	$(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$	$(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$	$(\sigma_1-\sigma_3)_{max}$	$(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$		$(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$		
EKE-1	$\overline{CK_oUE}$	1	.210	0.180	1.070	1.070	9.4	9.4	32.5	--	32.5	--	AV
EIE-1-1	$\overline{CIUE}$	1	.250	0.215	1.495	1.495	9.9	9.9	30.5	--	30.5	--	AV
AKE-1	$\overline{CK_oUE}$	2	.380	0.350	0.710	0.710	12.5	12.5	30	.017	30	.017	AV
EIE-1-2	$\overline{CIUE}$	2	.460	0.420	0.645	0.645	9.8	9.8	30	.017	30	.017	AV
EKE-3	$\overline{CK_oUE}$	4	.650	0.595	0.400	0.400	11.5	11.5	30	.017	30	.017	AV
EIE-1-3	$\overline{CIUE}$	4	.750	0.685	0.300	0.300	10.7	10.7	30	.017	30	.017	AV

Notes: (1) Developing distinct failure plane across the varves = AV

(2) $\bar{\phi}$ and $\bar{c}/\bar{\sigma}_{vm}$ are normalized Mohr-Coulomb effective stress strength parameters.

Table 5.3

COMPARISON OF NSP FROM $\overline{CIUE}$ (θ=90) AND $\overline{CIUC}$ (θ=0) TESTS ON CONNECTICUT VALLEY VARVED CLAY (VERTICAL LOADING)

Test No.	(1) Type of Test	$\theta°$	OCR	$q_f/\bar{\sigma}_{vc}$	(4) ε_f %	$\tau_{ff}/\bar{\sigma}_{vc}$ at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$	Pore Pressure Parameters at q_f A_f	$a^{(f)}$	$\bar{\phi}°$	$\bar{c}/\bar{\sigma}_{vm}$	$\bar{\phi}°$	$\bar{c}/\bar{\sigma}_{vm}$	$E/\bar{\sigma}_{vc}$ at ε = 1/2%	(3) Type of Failure
									$(\sigma_1-\sigma_3)_{max}$		$(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$			
EIE-3-1	$\overline{CIUE}$	90	1	0.270	6.2	0.220	1.490	1.750	34.5	0	34.5	0	78	AV
AIC-1-1	$\overline{CIUC}$	0	1	0.254	4.5	0.210	1.420	2.310	28.1	0	33.5	0	70	B
EIE-3-2	$\overline{CIUE}$	90	2	0.450	6.0	0.410	0.750	0.180	30.0	0.017	30.0	0.017	110	AV
EIC-1-2	$\overline{CIUC}$	0	2	0.420	7.4	0.370	0.675	0.725	22.5	0.060	30.0	0.017	90	B
EIE-3-3	$\overline{CIUE}$	90	4	0.700	4.4	0.640	0.410	-0.550	30.0	0.017	30.0	0.017	140	AV
EIC-1-3	$\overline{CIUC}$	0	4	0.68	11.3	0.610	0.240	-0.200	22.5	0.060	30.0	0.017	110	B

Notes:
(1) W_N = 56.2 - 59% for $\overline{CIUE}$ test samples; and W_N = 56.0 - 58.2% for $\overline{CIUC}$ test samples.
(2) $\Delta U = \Delta\sigma_{oct} + a\Delta\tau_{oct}$.
(3) AV = Across the Varves
B = Bulging
(4) At q_f condition.

Table 5.4

COMPARISON OF NSP FROM $\overline{CIUE}$ (θ=0) AND $\overline{CIUC}$ (θ= 90) TESTS ON CONNECTICUT VALLEY VARVED CLAY (HORIZONTAL LOADING)

Test No.	(1) Type of Test	OCR	$\frac{q_f(H)}{\bar{\sigma}_{vc}}$	(4) ε_f %	$\tau_{ff}/\bar{\sigma}_{vc}$ at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$	Pore Pressure Parameters at q_f		$\bar{\phi}°$	$\bar{c}/\bar{\sigma}_{vm}$	$\bar{\phi}°$	$\bar{c}/\bar{\sigma}_{vm}$	$\theta°$	$E/\bar{\sigma}_{vc}$ at ε = 1/2%	(3) Type at Failure
						A_f	a	$(\sigma_1-\sigma_3)_{max}$		$(\bar{\sigma}_1-\bar{\sigma}_3)_{max}$				
EIE-1-1	$\overline{CIUE}$	1	0.250	9.9	0.215	1.495	1.760	30.5	--	30.5	--	0	56	AV
EIC-3-1	$\overline{CIUC}$	1	0.290	9.3	0.255	1.180	1.800	28.5	--	28.5	--	90	64	AV
EIE-1-2	$\overline{CIUE}$	2	0.460	9.8	0.420	0.645	-0.050	30.0	0.017	30.0	0.017	0	75	AV
EIC-3-2	$\overline{CIUC}$	2	0.510	6.5	0.465	0.400	-0.140	30.0	0.017	30.0	0.017	90	110	AV
EIE-1-3	$\overline{CIUE}$	4	0.750	10.7	0.685	0.300	-0.780	30.0	0.017	30.0	0.017	0	144	AV
EIC-3-3	$\overline{CIUC}$	4	0.785	7.4	0.720	0.128	-0.435	30.0	0.017	30.0	0.017	90	170	AV

Notes: (1) W_N = 56.0 - 58.2% for $\overline{CIUE}$ samples; and W_N = 54.0 - 60.1% for $\overline{CIUC}$ samples.

(2) $\Delta u = \Delta\sigma_{oct} + a\Delta\tau_{oct}$

(3) AV = Across the Varves

(4) At q_f condition.

Table 5.5

COMPARISON OF NSP FROM $\overline{CK_oU}$ TRIAXIAL, PLANE STRAIN, AND SPECIAL DSS TESTS ON NORMALLY CONSOLIDATED SAMPLES

Test No.	Stress System	Type of Test	$\beta°$	$\frac{\bar{\sigma}_{vm}(L)}{\bar{\sigma}_{vm}}$	$\bar{\sigma}_{vc}$ kg/cm^2	At $(\sigma_1-\sigma_3)_{max}$ (2)				At $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ (2)			
						$q_f/\bar{\sigma}_{vc}$	A_f	$\bar{\phi}°$	γ_f % (3)	$\frac{\tau_{ff}}{\bar{\sigma}_{vc}}$	A_f	$\bar{\phi}°$	γ_f % (3)
AKC-1	Triaxial	$\overline{CK_oUC}$	0	1.88	3.0	0.25	1.32	21.0	1.5	0.18	3.70	31.7	23.2
NPC-1	Plane Strain	$\overline{CK_oUPSE}$	0	1.16	4.05	0.28	1.09	23.5	0.76	0.19	3.77	33	7.8
ESDS-1	DSS	$\overline{CK_oUDSS(C)}$	9	1.67	4.01	0.25	1.16	21.0	3.6	0.18	4.05	33	13.0
EKE-1	Triaxial	$\overline{CK_oUE}$	90	1.5	3.6	0.21	1.07	31.0	22.8	0.18	1.07	31	22.8
NPE-1	Plane Strain	$\overline{CK_oUPSE}$	90	1.14	4.00	0.25	0.85	27.0	8.0	0.22	0.85	27	8.0
ESDS-2	DSS	$\overline{CK_oUDSS(E)}$	111	1.67	4.01	0.21	1.09	31.0	20.0	0.18	1.09	31	20.0

Notes: (1) $\bar{\sigma}_{vc} = \bar{\sigma}_{\alpha o} + \tau_{\alpha o}$ and $S_u = q_f$.

(2) At $\tau_{\alpha max}$ and $\left(\tau_\alpha/\bar{\sigma}_\alpha\right)_{max}$ for DSS tests.

(3) $\gamma_f = 1.5$ for triaxial and $\gamma_f = 2.0\ \varepsilon_f$ plane strain tests.

Table 5.6

LOAD INCREMENTS USED IN $\overline{CK_oUDSS}$(C) and $\overline{CK_oUDSS}$(E) TESTS ON NORMALLY CONSOLIDATED INCLINED SAMPLES

$$\bar{\sigma}_{\alpha c} = \bar{\sigma}_{vo}(\cos^2\theta + K_o \sin^2\theta)$$

$$\tau_{\alpha c} = \bar{\sigma}_{vc}(1 - K_o)\sin\theta\cos\theta$$

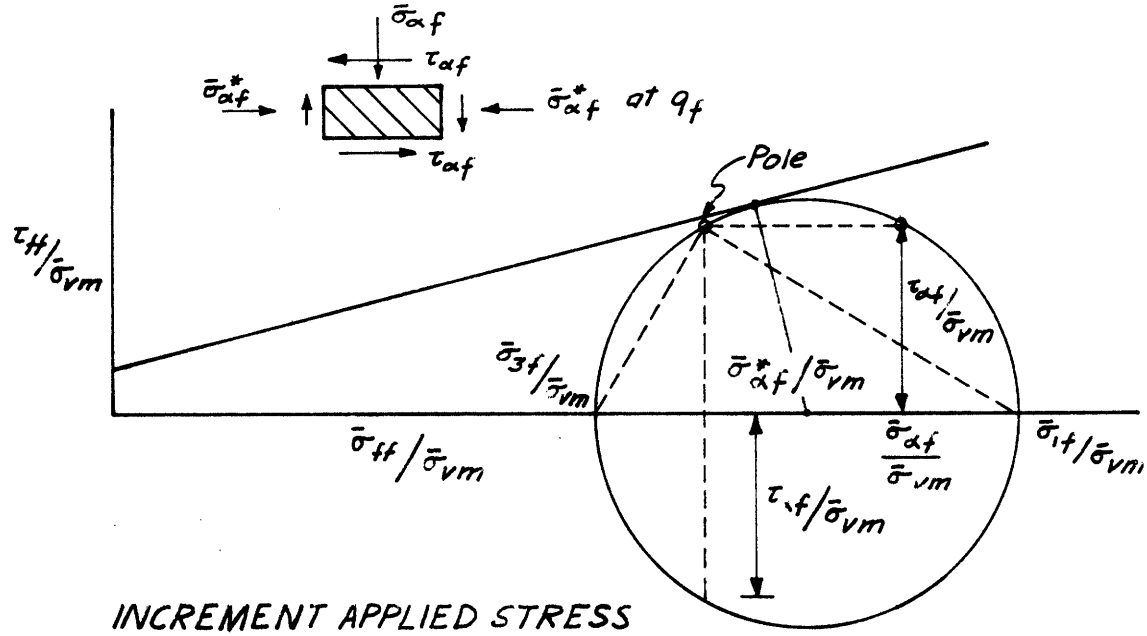
Stress in kg/cm^2

OCR	K_o	$\bar{\sigma}_{\alpha c}$	$\tau_{\alpha c}$	$\bar{\sigma}_{vc}$	$\bar{\sigma}_{hc}$
23	1.0	0.104	0	0.104	0.104
11	1.0	0.211	0	0.211	0.211
4.6	1.0	0.520	0	0.520	0.520
2.36	0.91	0.971	0.049	1.016	0.927
1.61	0.80	1.342	0.146	1.488	1.196
1.133	0.74	1.838	0.281	2.119	1.557
1.0	0.68	2.097	0.399	2.496	1.698
1.0	0.68	2.505	0.479	2.984	2.026
1.0	0.68	3.020	0.575	3.595	2.445
1.0	0.68	3.395	0.639	4.034	2.756

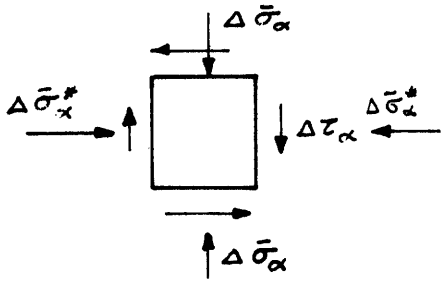
For $\theta = 45°$: $\bar{\sigma}_{vc} = \bar{\sigma}_{\alpha c} + \tau_{\alpha c}$ and $\bar{\sigma}_{hc} = \bar{\sigma}_{\alpha c} - \tau_{\alpha c}$

Table 5.7

ESTIMATION OF A_f FROM DSS TEST SAMPLE ($\theta = 45$) USING NESE ASSUMPTION



INCREMENT APPLIED STRESS



$$A_f = \frac{\Delta u_f - \Delta\sigma_{3f}}{\Delta\sigma_{1f} - \Delta\sigma_{3f}} \qquad \text{Eq. 5.8}$$

$\bar{\sigma}_{\alpha c}$ *from equation 5.4*

$$\Delta\bar{\sigma}_{\alpha f} = \bar{\sigma}_{\alpha f} - \bar{\sigma}_{\alpha c} = 0 \qquad \text{Eq. 5.9}$$

$$\Delta\bar{\sigma}^*_{\alpha f} = \bar{\sigma}^*_{\alpha f} - \bar{\sigma}_{\alpha c} \qquad \text{Eq. 5.10}$$

$$\Delta\tau_{\alpha f} = \tau_{\alpha f} - \tau_{\alpha c} \qquad \text{Eq. 5.11}$$

Assuming two dimensional applied stress system

$$\Delta\sigma_{1f} = \frac{\Delta\bar{\sigma}^*_\alpha}{2} + \sqrt{\left(\frac{\Delta\bar{\sigma}^*_\alpha}{2}\right)^2 + (\Delta\tau_{\alpha f})^2} \qquad \text{Eq. 5.12}$$

$$\Delta\sigma_{3f} = \frac{\Delta\bar{\sigma}^*_\alpha}{2} - \sqrt{\left(\frac{\Delta\bar{\sigma}^*_\alpha}{2}\right)^2 + (\Delta\tau_{\alpha f})^2} \qquad \text{Eq. 5.13}$$

$\Delta u_f = \sigma_{\alpha f} - \bar{\sigma}_{\alpha c}$ *in DSS test*

TABLE 5.8

SUMMARY OF THE EFFECT OF $\Delta\sigma_2$ ON VERTICAL LOADING

Type of Test	Condition of $\Delta\sigma_2$	Data at q_f					$E_u/\bar{\sigma}_{vc}$ at the (6) same ASL	Date at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$		
					$\overline{NESE}$ (3)				$\overline{NESE}$	
		$q_f/\bar{\sigma}_{vc}$	A_f	$\bar{\phi}$ $(\bar{c}=0)$	$\bar{\phi}$	$\bar{c}/\bar{\sigma}_{vm}$		$\bar{\phi}$ $(\bar{c}=0)$	$\bar{\phi}$	$\bar{c}/\bar{\sigma}_{vm}$
$\overline{CIUE}$ (θ=90°) versus $\overline{CIUC}$ (θ=0)	↑ increase $\Delta\sigma_2$	↑ (1) ($\approx$5%)	↑	↑	↑	↓	slightly ↑	slightly ↑	(2) X	X
$\overline{CK_oUPSC}$ versus $\overline{CK_oUC}$	↑ increase $\Delta\sigma_2$	↑ ($\approx$12%)	↓	↑	(4) X*	(4) X*	↑	↑	X	X
$\overline{CK_oUDSS}$ (5) versus $\overline{CK_oUC}$	↑ increase $\Delta\sigma_2$	X	X*	X	X	X	↓	X	X	X

Notes:
(1) ↑ direction of the increases in soil properties from bottom to the top of the arrow.
(2) X= no change.
(3) When not equal to that at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$.
(4) X* = practically insignificant.
(5) Interpreting by using $\overline{NESE}$ from $\overline{CK_oUC}$ tests.
(6) Applied stress level.

Table 5.9

SUMMARY OF THE EFFECT OF $\Delta\sigma_2$ ON HORIZONTAL LOADING

Type of Test	Condition of $\Delta\sigma_2$	Data at q_f					$E_u/\bar{\sigma}_{vc}$ at the same ASL(4)	Remarks
		$q_f/\bar{\sigma}_{vc}$	A_f	$\bar{\phi}$ ($\bar{c}$=0)	$\bar{\phi}$	$\bar{c}/\bar{\sigma}_{vm}$		
$\overline{CIUC}$ (θ=0) versus $\overline{CIUE}$ (θ=90)	↑increase $\Delta\sigma_2$	↓ (1) (10±5%)	↑	↑	(2) X	X	slightly ↓	Same $\overline{NESE}$ at q_f and $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$
$\overline{CK_oUE}$ versus $\overline{CK_oUPSE}$	↑increase $\Delta\sigma_2$	↓ (≈20%)	↑	↑	X	X	↓	Same $\overline{NESE}$ at q_f and $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$
$\overline{CK_oUE}$ versus $\overline{CK_oUDSS(E)}$[5]	↑increase $\Delta\sigma_2$	X	X*	X	X	X	(3) X*	Same $\overline{NESE}$ at q_f and $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$

Notes: (1) ↑ direction of the increases in soil properties from bottom to the top of the arrow.
(2) X = no change.
(3) X*= practically insignificant
(4) Applied shear stress level.
(5) Interpreting using NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ for failure across the varves.

Table 5.10

SUMMARY OF NSP FROM $\overline{CIUC}$ TESTS ON INCLINED SAMPLES ($\theta = 45°$)

Test No.	Soil Location	OCR	$\bar{\sigma}_{vm}$	$\frac{\bar{\sigma}_{vm}\ (L)}{\bar{\sigma}_{vm}}$	At $(\sigma_1-\sigma_3)_{max}$				At $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$			
					$\frac{q_f}{\bar{\sigma}_{vc}}$	$\frac{\bar{p}_f}{\bar{\sigma}_{vc}}$	A_f	ε_f %	$q/\bar{\sigma}_{vc}$	$\bar{p}/\bar{\sigma}_{vc}$	A_f	ε_f %
AIC-2-1	Amherst	1	2.1	1.90	0.215	0.517	1.63	5.8	0.210	0.465	1.77	9.8
EIC-2-1	East Windsor	1	2.4	1.67	0.205	0.550	1.64	5.4	0.190	0.435	2.09	10.3
EIC-2-3		4	2.4	1.67	0.575	1.345	0.20	4.9	0.550	1.255	0.27	8.7
EIC-2-4		20	2.4	1.67	1.330	2.550	-0.08	4.8	1.330	2.550	-0.08	4.8

Table 5.11

RESULTS FROM DRAINED DIRECT SHEAR TEST ON NORTHAMPTON VARVED CLAY FOR FAILURE WITHIN CLAY LAYER (LADD AND WISSA, 1970 AND LACASSE ET AL., 1972)

Test No.	$\bar{\sigma}_{vm}$ TSF	$\bar{\sigma}_{vc}$ TSF	$\frac{\tau_h}{\bar{\sigma}_{vc}}$	$\bar{\phi}$ (1)	$q/\bar{\sigma}_{vc}$ (2)	$\frac{q_f}{\bar{\sigma}_{vm}}$	$\frac{\bar{p}_f}{\bar{\sigma}_{vm}}$ (3)
NCD-5	3.65	2.00	0.380	19.5°	0.410	0.220	0.620
NCD-6	3.4	1.01	0.495	19.5°	0.525	0.160	0.350
NCD-1	4.0 (4)	4.0	0.385	21.0°	0.410	0.410	1.150
NCD-2	5.0 (4)	5.0	0.390	21.3°	0.420	0.420	1.150
NCD-3	5.14 (4)	5.14	0.450	24.1°	0.490	0.490	1.220
NDC-4	6.00 (4)	6.00	0.340	18.8°	0.360	0.360	1.120

Notes: (1) From Lacasse et al. (1972) for overconsolidated samples:
$\bar{\phi} = \tan^{-1} \frac{\tau_h}{\bar{\sigma}_{vc}}$ for normally consolidated samples.

(2) $q_f/\bar{\sigma}_{vc} = (\tau_h/\bar{\sigma}_{vc})(1/\cos \bar{\phi})$

(3) $\bar{p}_f/\bar{\sigma}_{vm} = \tau_h/\bar{\sigma}_{vm} \tan\bar{\phi} + \bar{\sigma}_{vc}/\bar{\sigma}_{vm}$

(4) Laboratory maximum past pressure ($\bar{\sigma}_{vm}(L)$)

Table 5.12

SUMMARY OF THE TEST RESULTS FROM $\overline{CK_oUDSS}$ TESTS

OCR	At $\tau_{h_{max}}$						At $\tau_h/\bar{\sigma}_{v_{max}}$					
	β°	$\frac{\tau_h}{\bar{\sigma}_{vc}}$	$\frac{\tau_{ff}}{\bar{\sigma}_{vc}}$ (1)	A_f	$\frac{q_f}{\bar{\sigma}_{vc}}$ (1)	γ_f	β°	$\frac{\tau_h}{\bar{\sigma}_{vc}}$	$\frac{\tau_{ff}}{\bar{\sigma}_{vc}}$	A_f	$\frac{q}{\bar{\sigma}_{vc}}$ (2)	γ
1	29°	0.165	0.185	1.475	0.190	6.7	39°	0.160	0.155	1.640	0.165	14.0
2	31°	0.300	0.315	0.820	0.330	7.5	34°	0.285	0.285	1.105	0.310	12.8
4	37.5°	0.460	0.460	0.435	0.480	8.0	37°	0.435	0.430	0.520	0.460	14.1

Notes: (1) Determined by using $\overline{NESE}$ at q_f from $\overline{CIUC}$ tests on inclined samples.

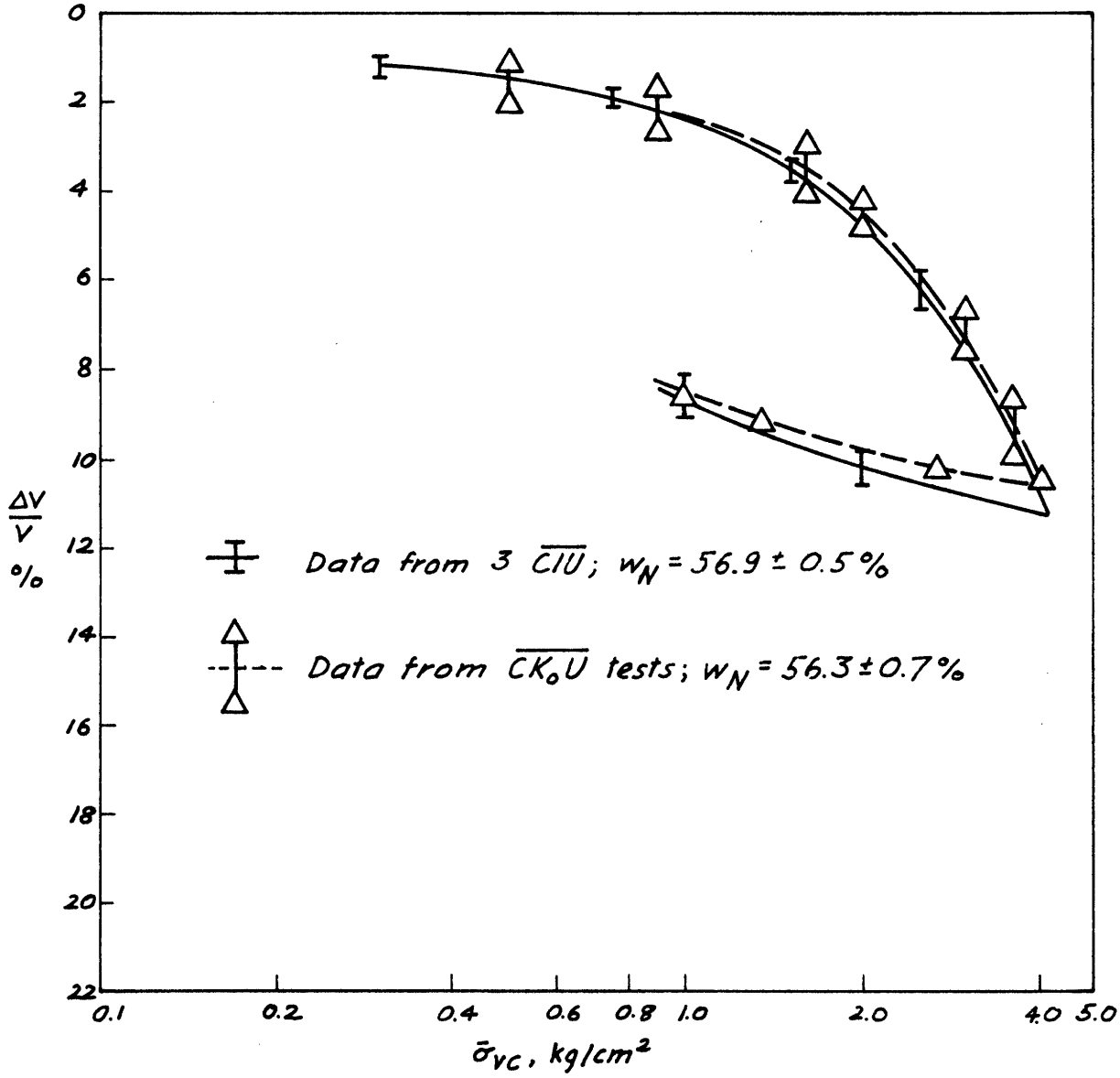
(2) Determined by using $\overline{NESE}$ at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ from $\overline{CIUC}$ tests on inclined samples and CDDS tests.

Table 5.13

COMPARISON OF TYPICAL VALUES OF $q_f/\bar{\sigma}_{vc}$, $\tau_{ff}/\bar{\sigma}_{vc}$ at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ AND A_f FROM $\overline{CIUC}$ TESTS ON INCLINED SAMPLES AND $\overline{CK_oUDSS}$ TESTS

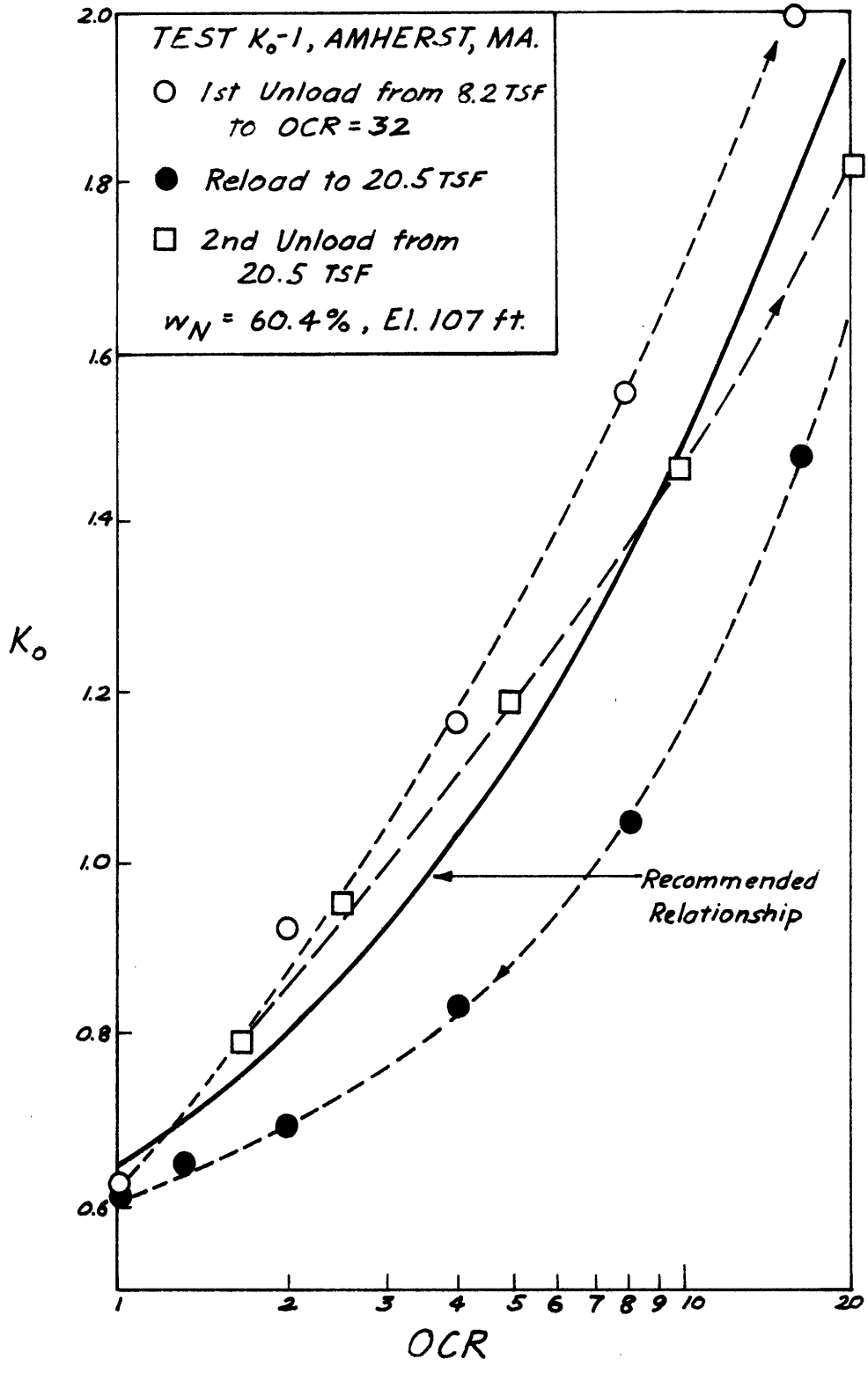
Test No.	Types of Tests	OCR	$q_f/\bar{\sigma}_{vc}$	$\tau_{ff}@(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$	A_f	
					$(\sigma_1-\sigma_3)_{max}$	$(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$
EIC-2-1	$\overline{CIUC}$	1	0.21	0.17	1.61	2.09
ADS - 1	$\overline{CK_oUDSS}$		0.19	0.15	1.47	1.64
EIC-2-2	$\overline{CIUC}$	4	0.57	0.55	0.20	0.27
EDS - 2	$\overline{CK_oUDSS}$		0.48	0.43	0.43	0.52

Table 5.14



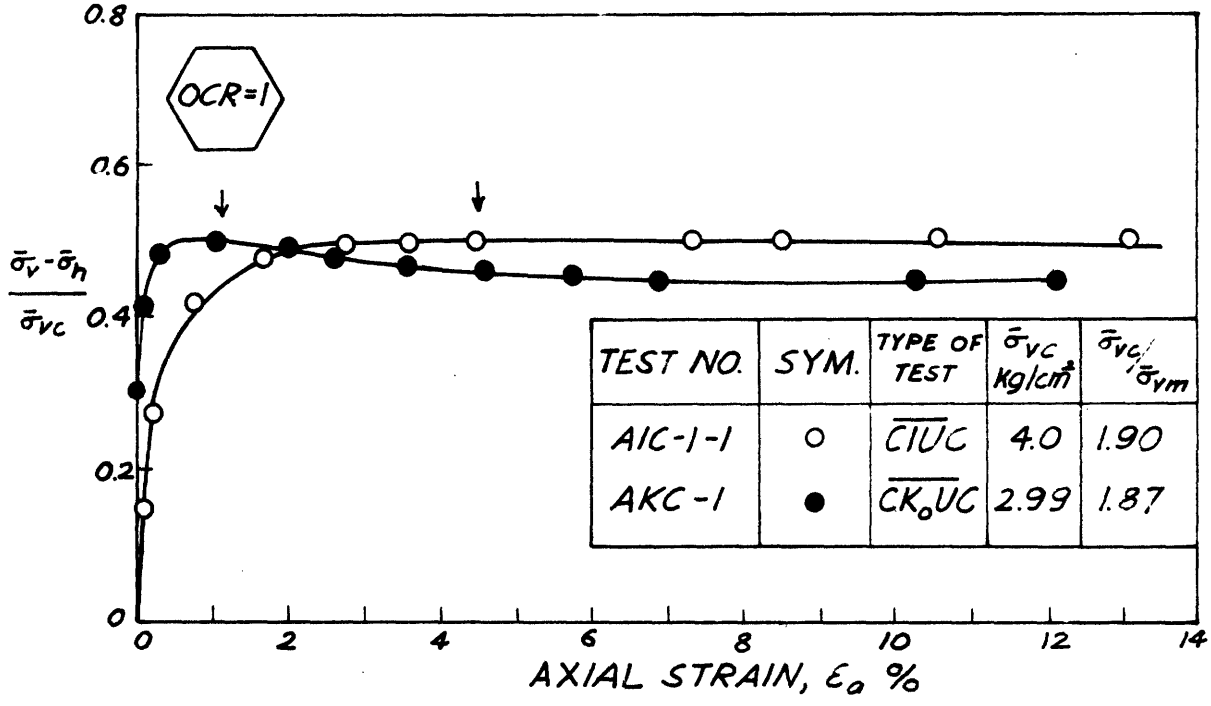
VOLUME CHANGE DATA FROM $\overline{CIU}$ AND $\overline{CK_oU}$ TESTS ON HORIZONTAL VARVE ($\theta = 0$) SAMPLES FROM EAST WINDSOR

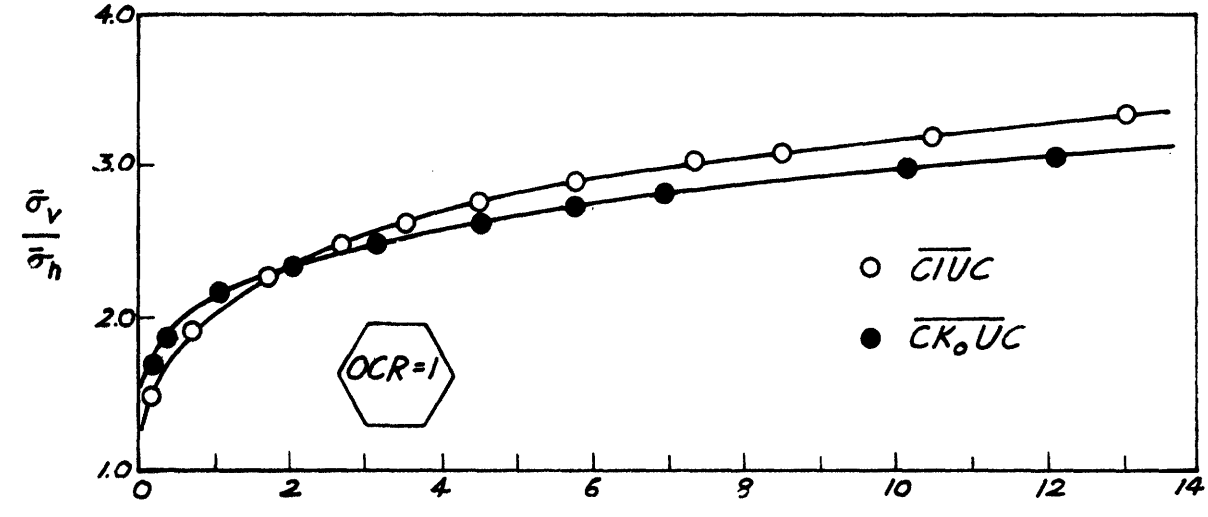
FIGURE 5.1



K_o VERSUS OCR FOR CONNECTICUT VALLEY VARVED CLAYS

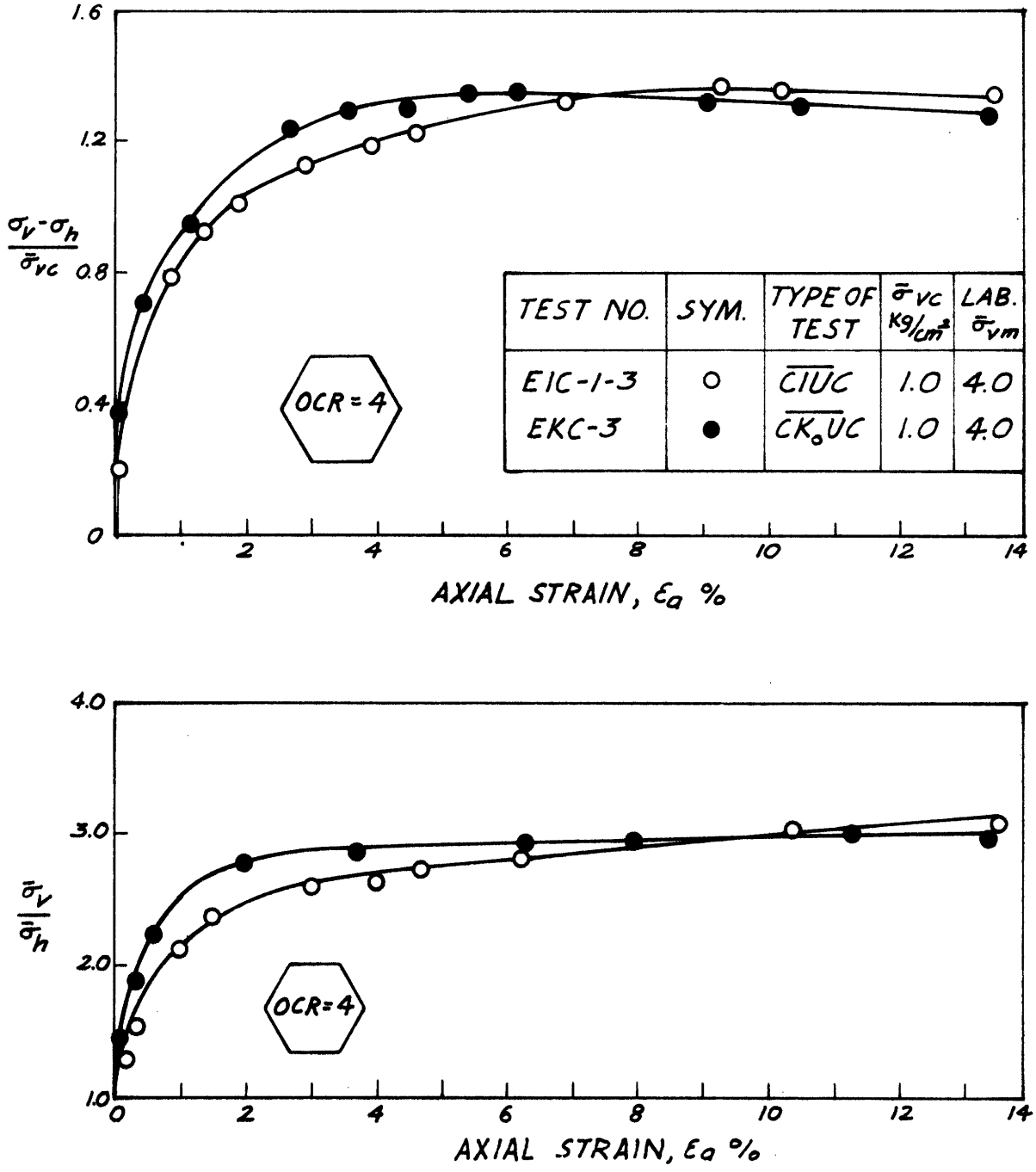
FIGURE 5.2





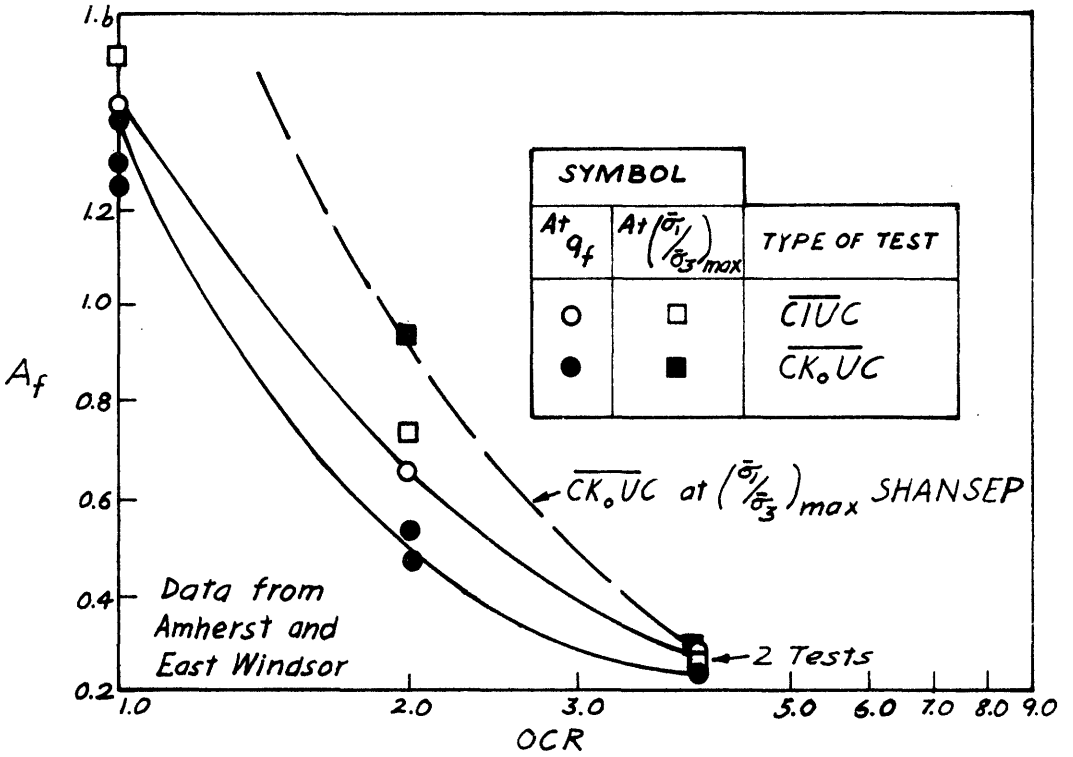
NORMALIZED STRESS-STRAIN DATA FROM $\overline{CIUC}$ AND $\overline{CK_oUC}$ TESTS ON NORMALLY CONSOLIDATED AMHERST VARVED CLAY

FIGURE 5.3

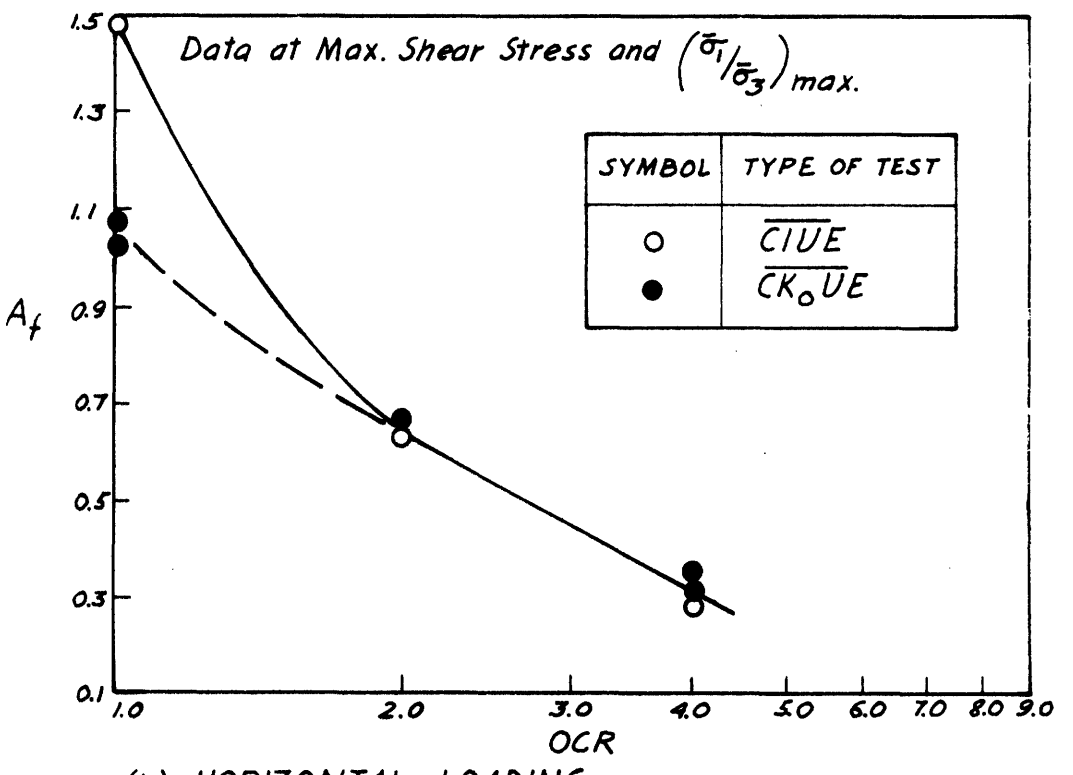


NORMALIZED STRESS-STRAIN CURVES $\overline{CIUC}$ AND $\overline{CK_oUC}$ TESTS ON OVERCONSOLIDATED (OCR = 4) EAST WINDSOR VARVED CLAY

FIGURE 5.4

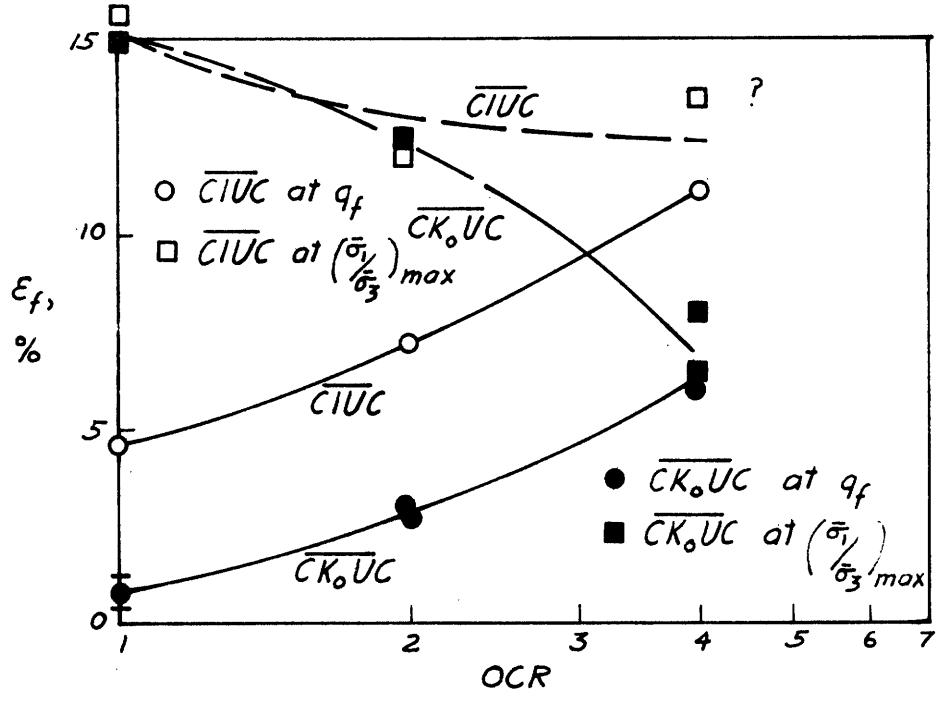


(a) VERTICAL LOADING

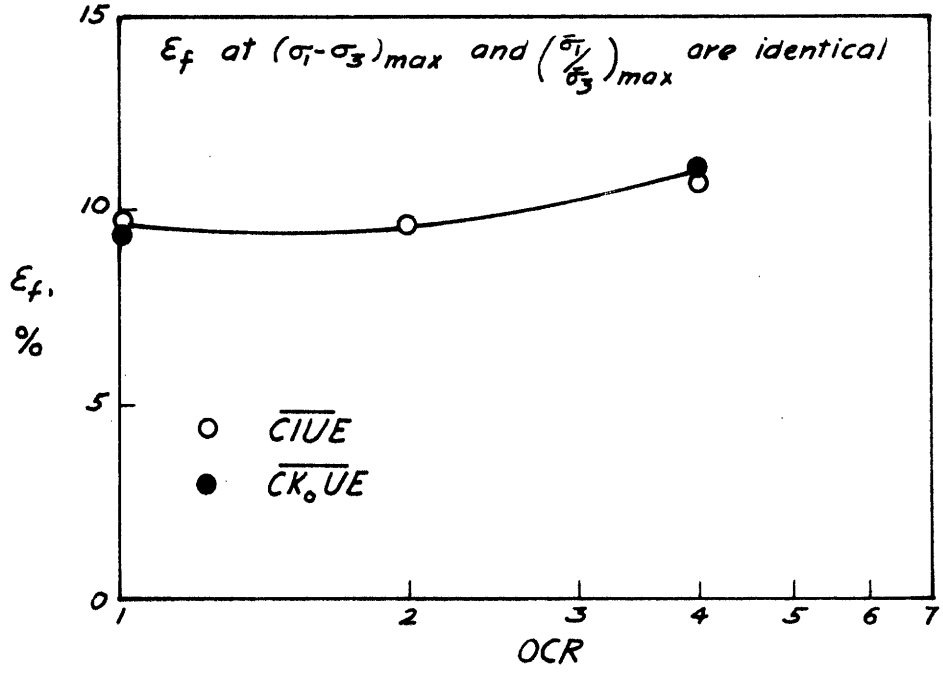


(b) HORIZONTAL LOADING

A_f VERSUS OCR FROM $\overline{CIUC}$, $\overline{CK_oUC}$, $\overline{CIUE}$, AND $\overline{CK_oUE}$ TESTS ON CONNECTICUT VALLEY VARVED CLAYS FIGURE 5.5



(a) VERTICAL LOADING



(b) HORIZONTAL LOADING

ε_f VERSUS OCR FROM $\overline{CIUC}$, $\overline{CK_oUC}$, $\overline{CIUE}$, AND $\overline{CK_oUE}$ TESTS ON CONNECTICUT VALLEY VARVED CLAYS

FIGURE 5.6

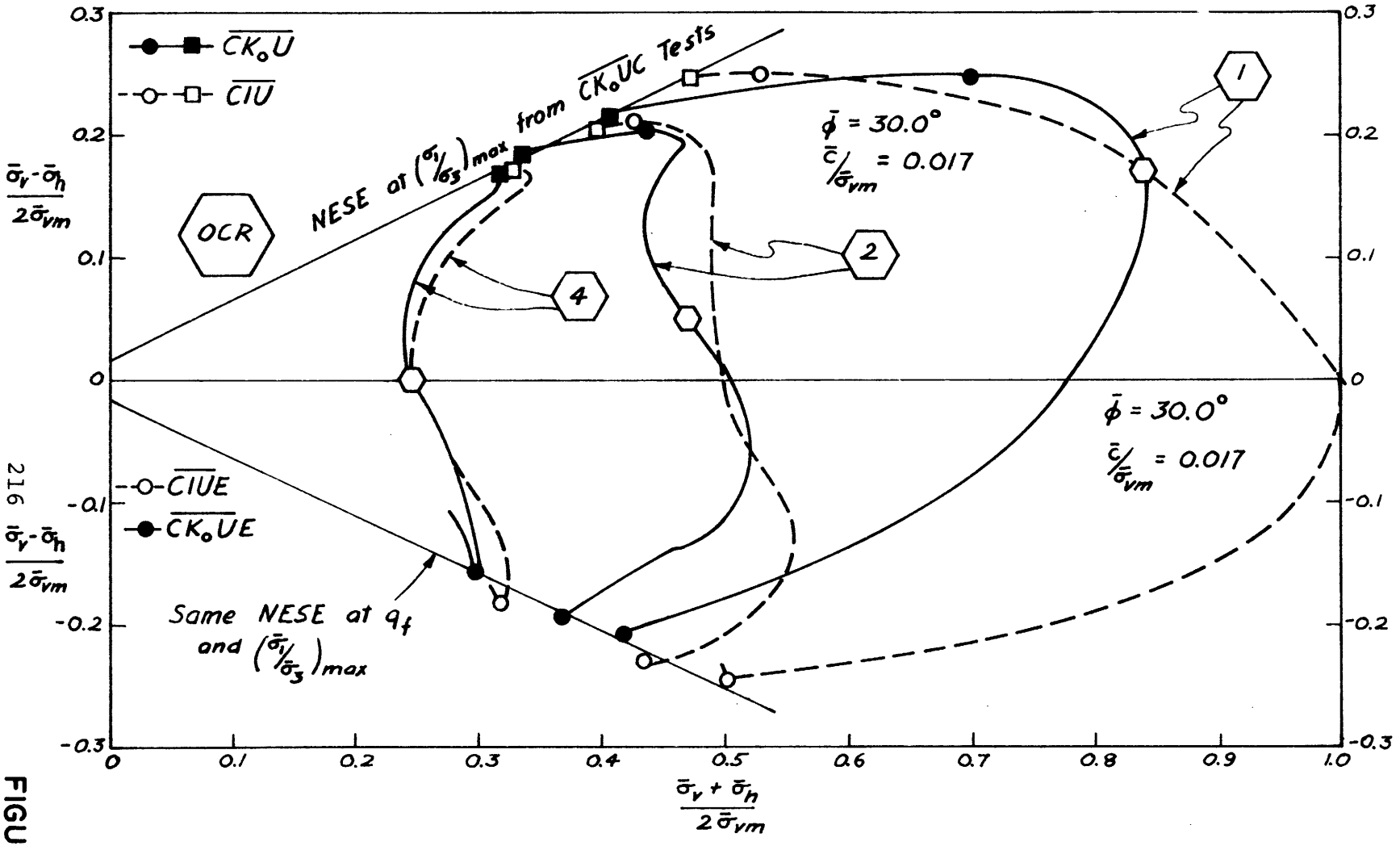


FIGURE 5.7 TYPICAL NORMALIZED EFFECTIVE STRESS PATHS FOR $\overline{CIU}$ AND $\overline{CK_oU}$ TRIAXIAL COMPRESSION AND EXTENSION TESTS ON NORMALLY CONSOLIDATED AND OVERCONSOLIDATED CONNECTICUT VALLEY VARVED CLAYS

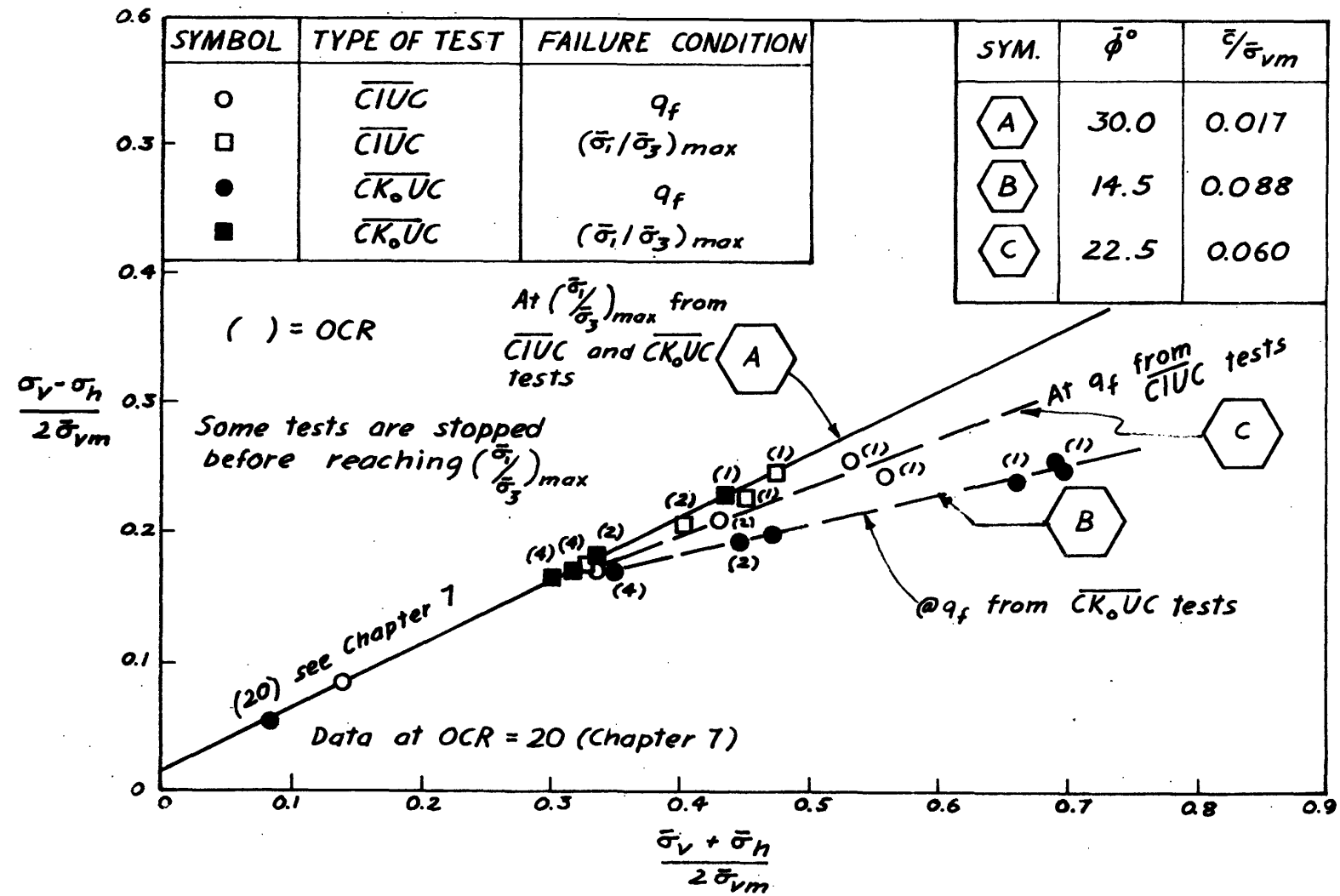
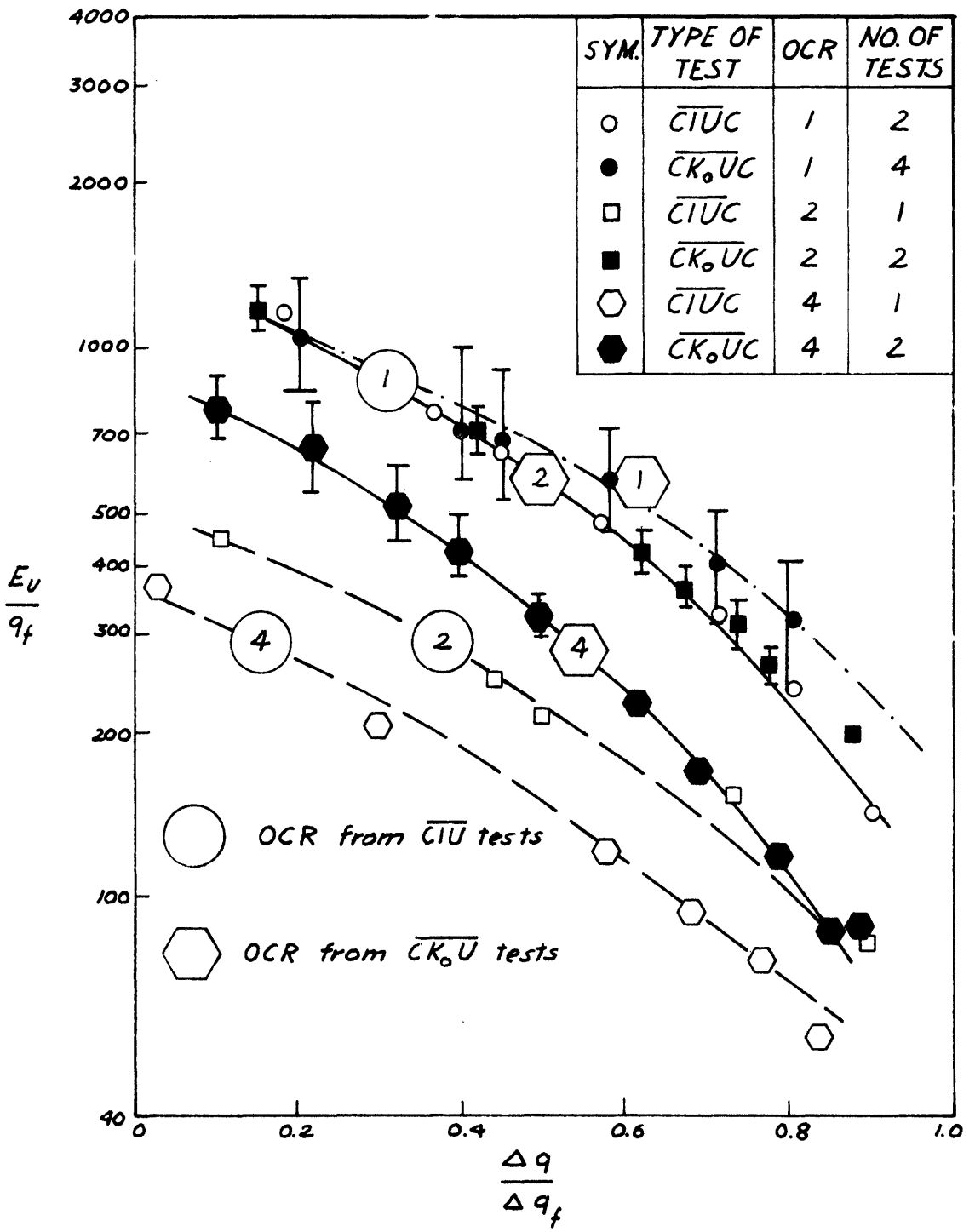
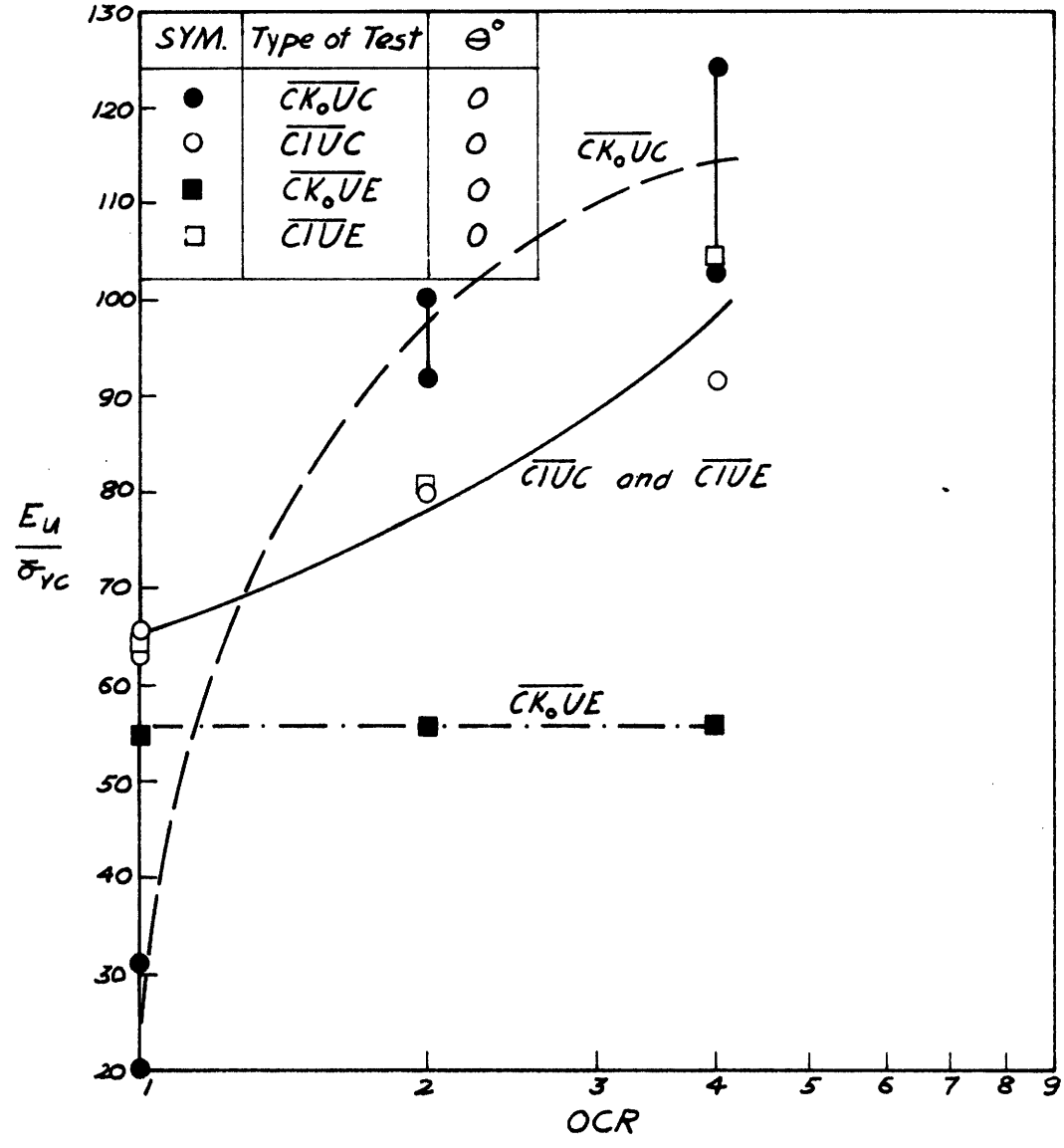


FIGURE 5.8 NORMALIZED EFFECTIVE STRESS ENVELOPES FROM $\overline{CIUC}$ AND $\overline{CK_oUC}$ TESTS ON CONNECTICUT VALLEY VARVED CLAYS



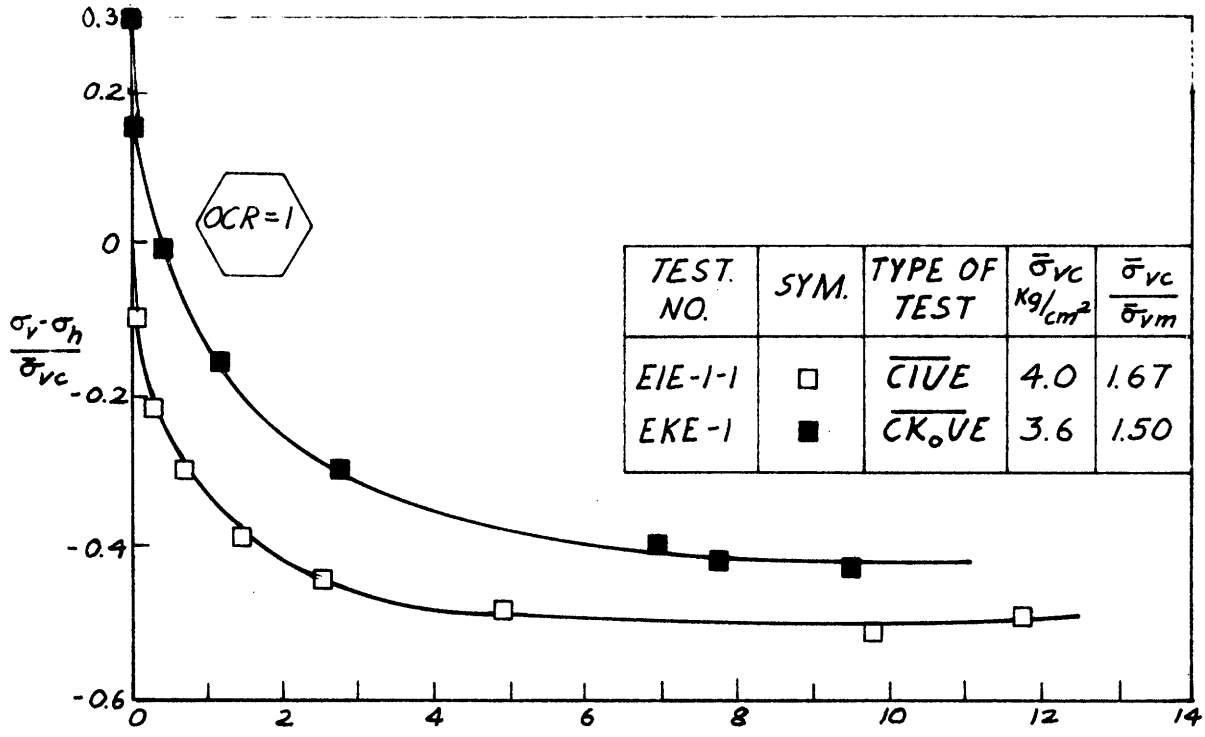
$\frac{E_u}{q_f}$ VERSUS $\frac{\Delta q}{\Delta q_f}$ FROM $\overline{CIU}C$ AND $\overline{CK_oU}C$ TESTS ON SAMPLES FROM CONNECTICUT VALLEY

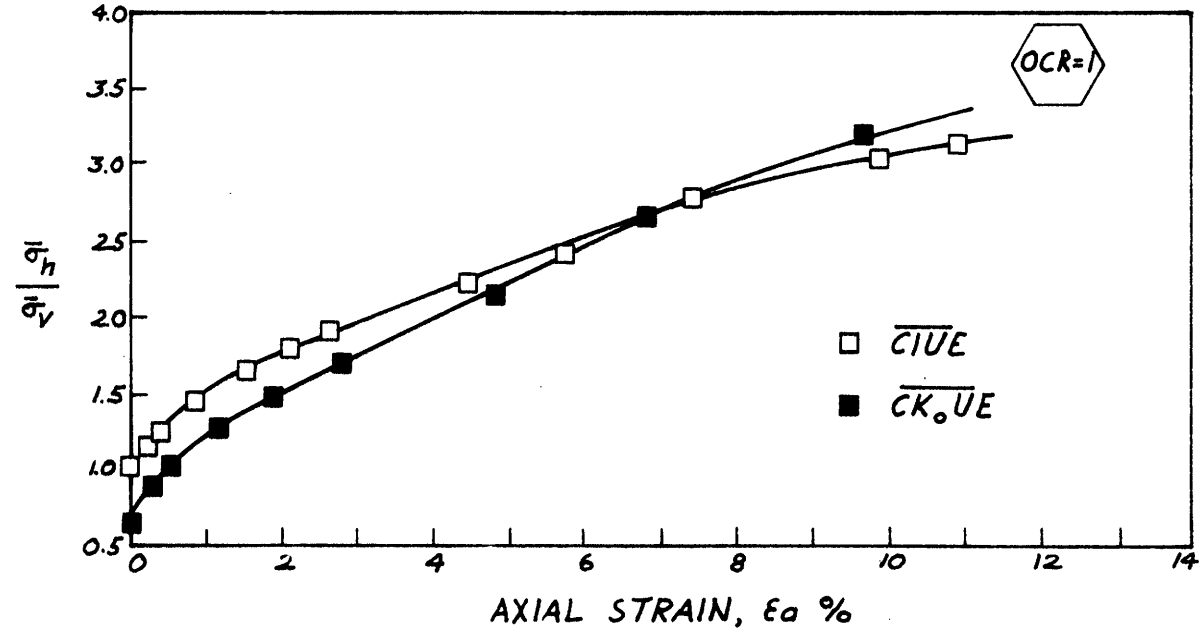
FIGURE 5.9



$\frac{E_u}{\bar{\sigma}_{VC}}$ AT $\varepsilon = 0.2\%$ VERSUS OCR FROM $\overline{CIUC}$, $\overline{CK_oUC}$, $\overline{CIUE}$, AND $\overline{CK_oUE}$ TESTS ON SAMPLES FROM EAST WINDSOR

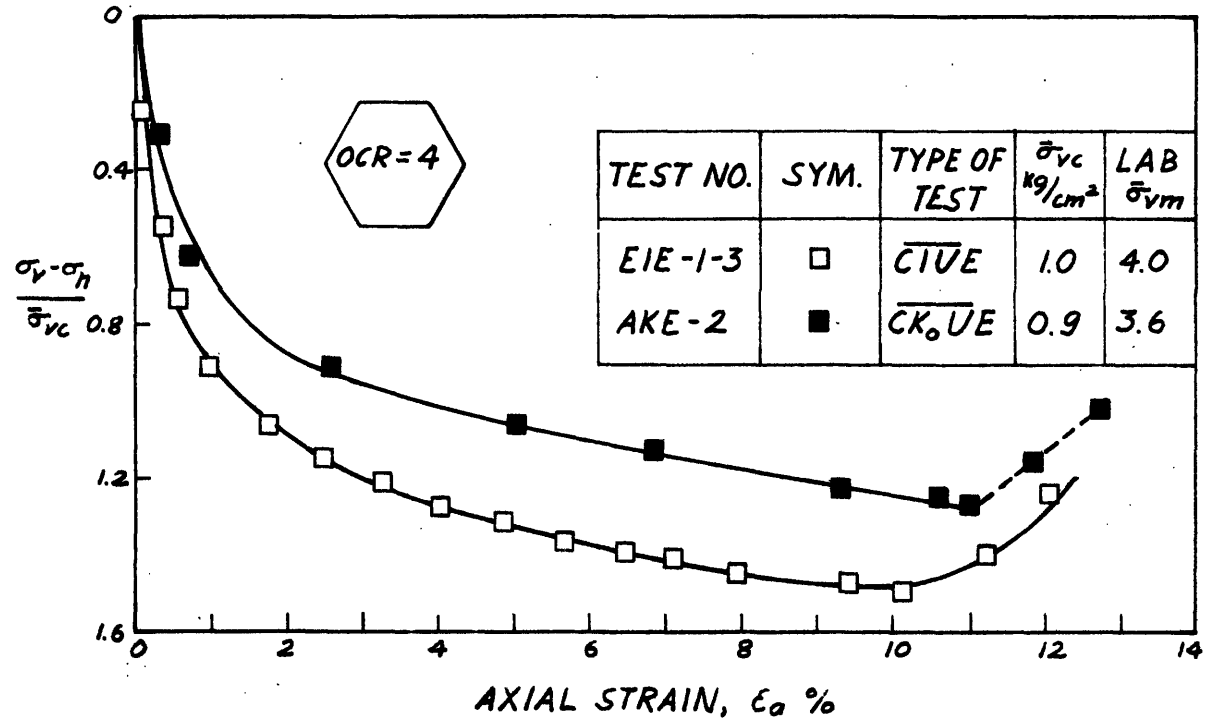
FIGURE 5.10

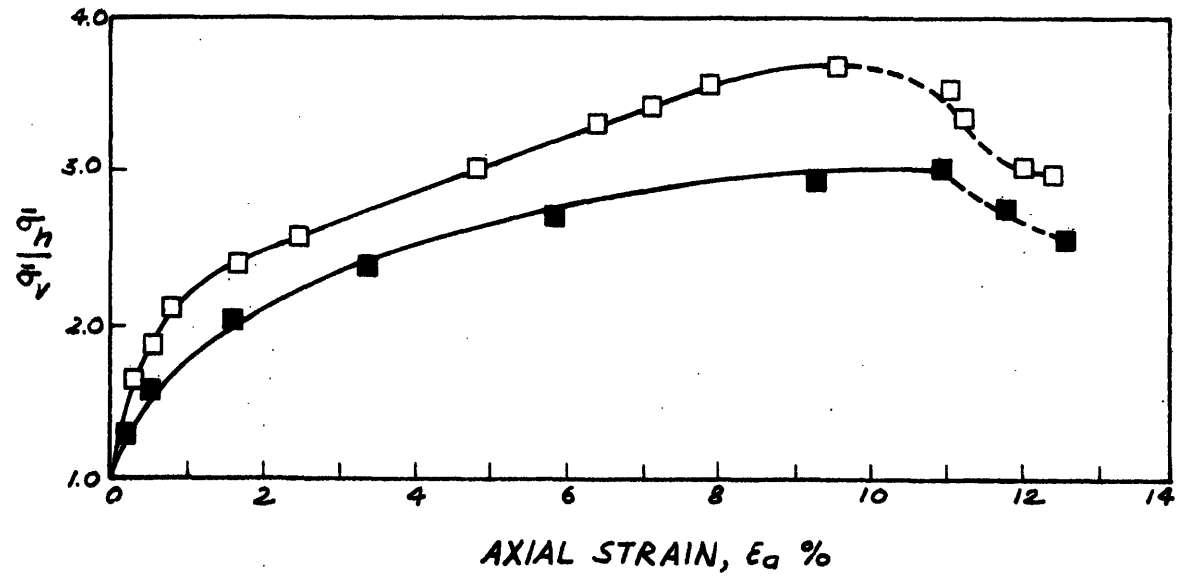




NORMALIZED STRESS-STRAIN CURVES FROM $\overline{CIUE}$ AND $\overline{CK_oUE}$ TESTS ON NORMALLY CONSOLIDATED EAST WINDSOR VARVED CLAY

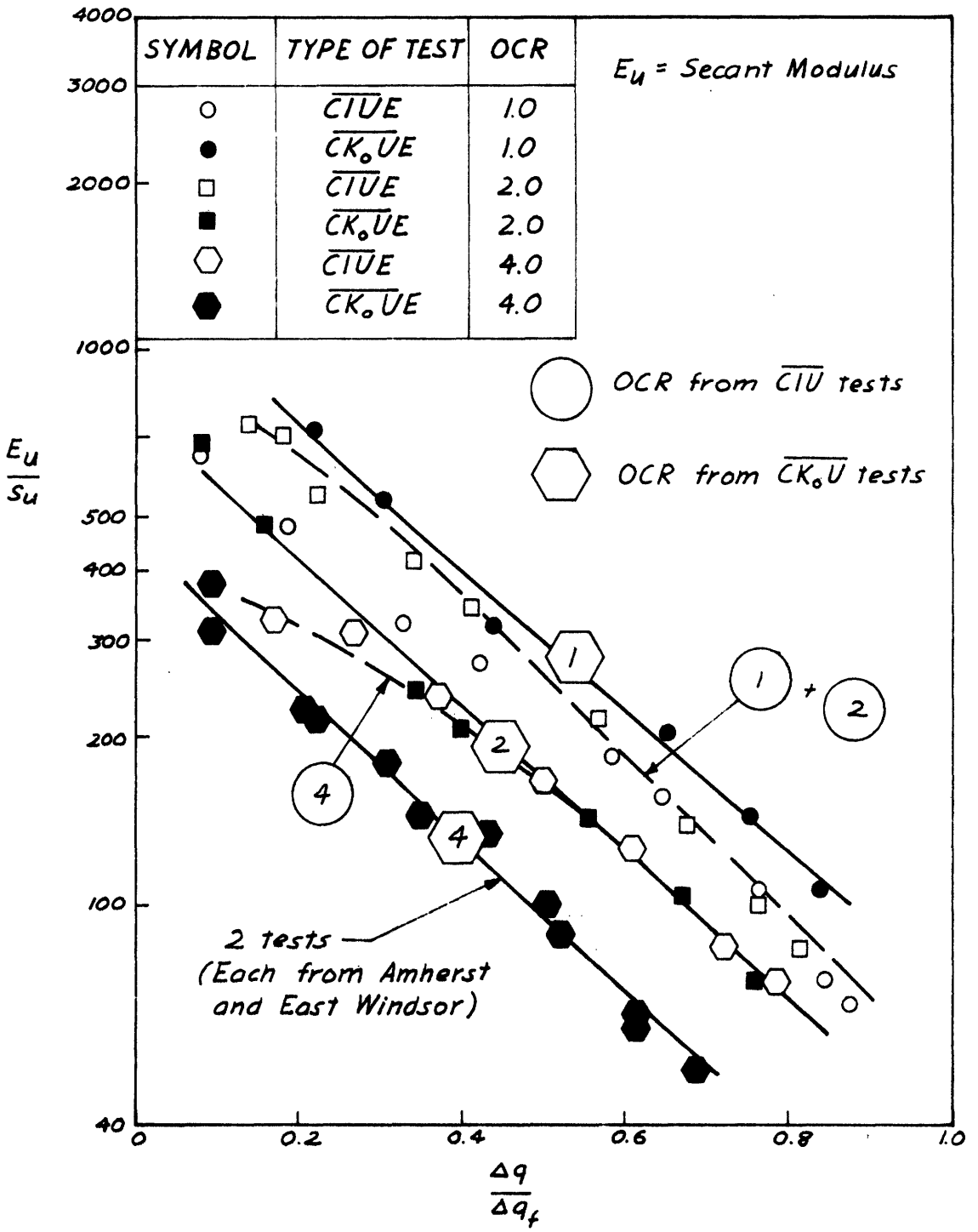
FIGURE 5.11





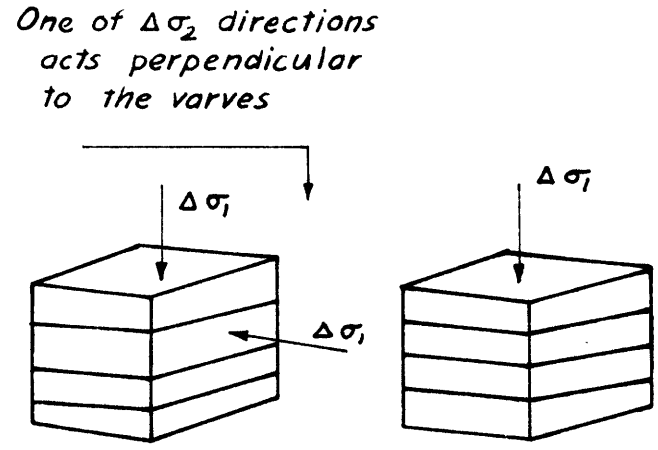
NORMALIZED STRESS-STRAIN CURVES FROM CIUE & CKoUE TESTS ON OVERCONSOLIDATED (OCR = 4.0) CONNECTICUT VALLEY VARVED CLAYS

FIGURE 5.12



$\frac{E_u}{S_u}$ VERSUS $\frac{\Delta q}{\Delta q_f}$ FROM $\overline{CIUE}$ AND $\overline{CK_oUE}$ TESTS ON SAMPLES FROM EAST WINDSOR

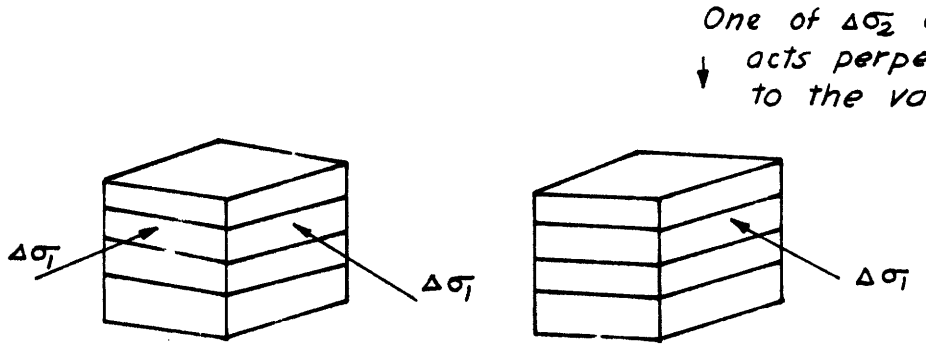
FIGURE 5.13



$\overline{CIUE}$ $(\Theta = 90)$ $\Delta\sigma_1 = \Delta\sigma_2 > \Delta\sigma_3$ $\Delta\sigma_3 = 0$

$\overline{CIUC}$ $(\Theta = 0)$ $\Delta\sigma_1 > \Delta\sigma_2 = \Delta\sigma_3$ $(\Delta\sigma_2 = \Delta\sigma_3 = 0)$

(a) VERTICAL LOADING ($\beta = 0$)



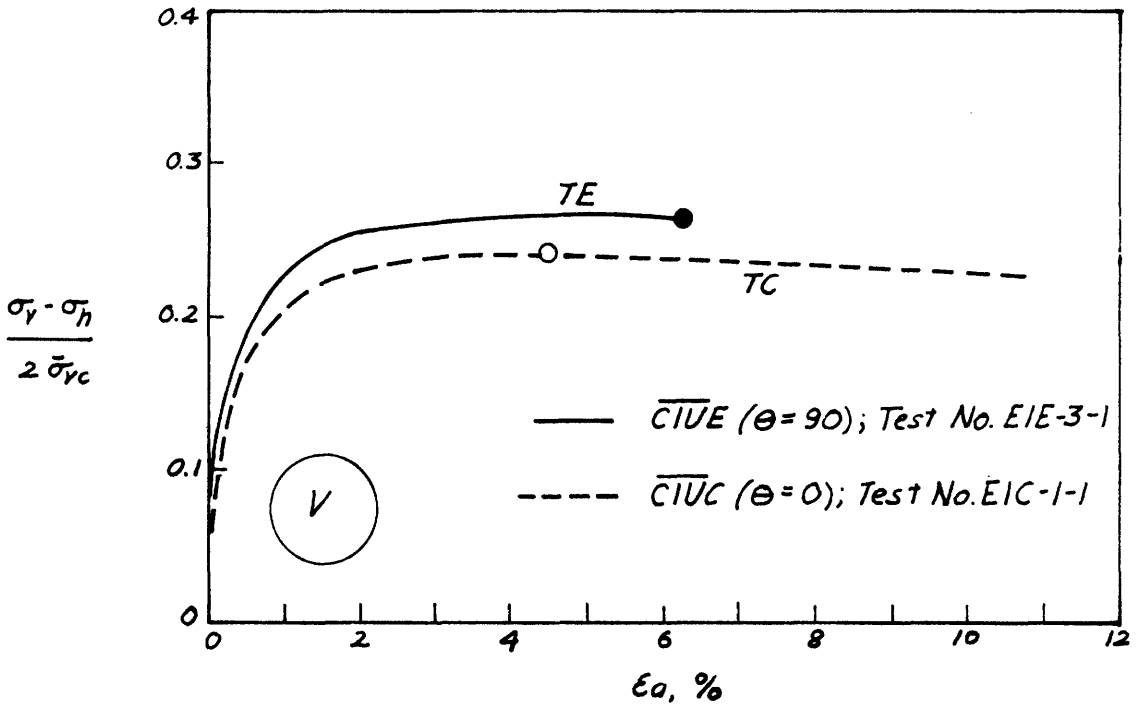
$\overline{CIUE}$ $(\Theta = 0)$ $\Delta\sigma_1 = \Delta\sigma_2 > \Delta\sigma_3$ $(\Delta\sigma_3 = 0)$

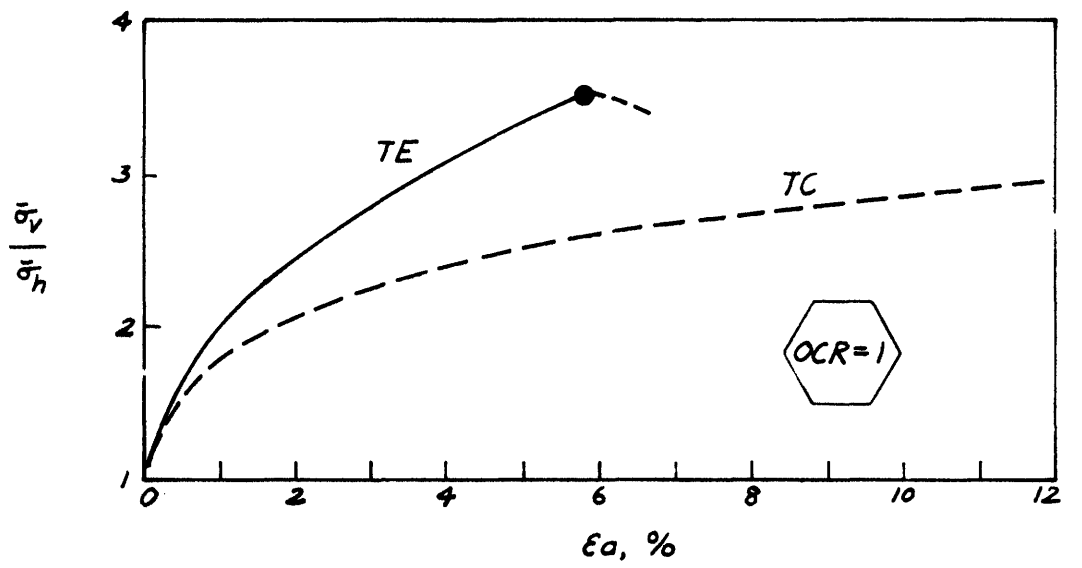
$\overline{CIUC}$ $(\Theta = 90)$ $\Delta\sigma_1 > \Delta\sigma_2 = \Delta\sigma_3$ $(\Delta\sigma_2 = \Delta\sigma_3) = 0$

(b) HORIZONTAL LOADING ($\beta = 90$)

STATES OF STRESS FOR $\overline{CIUC}$ ($\theta = 0$), $\overline{CIUE}$ ($\theta = 0$), $\overline{CIUE}$ ($\theta = 90$), AND $\overline{CIUC}$ ($\theta = 90$) TEST SAMPLES

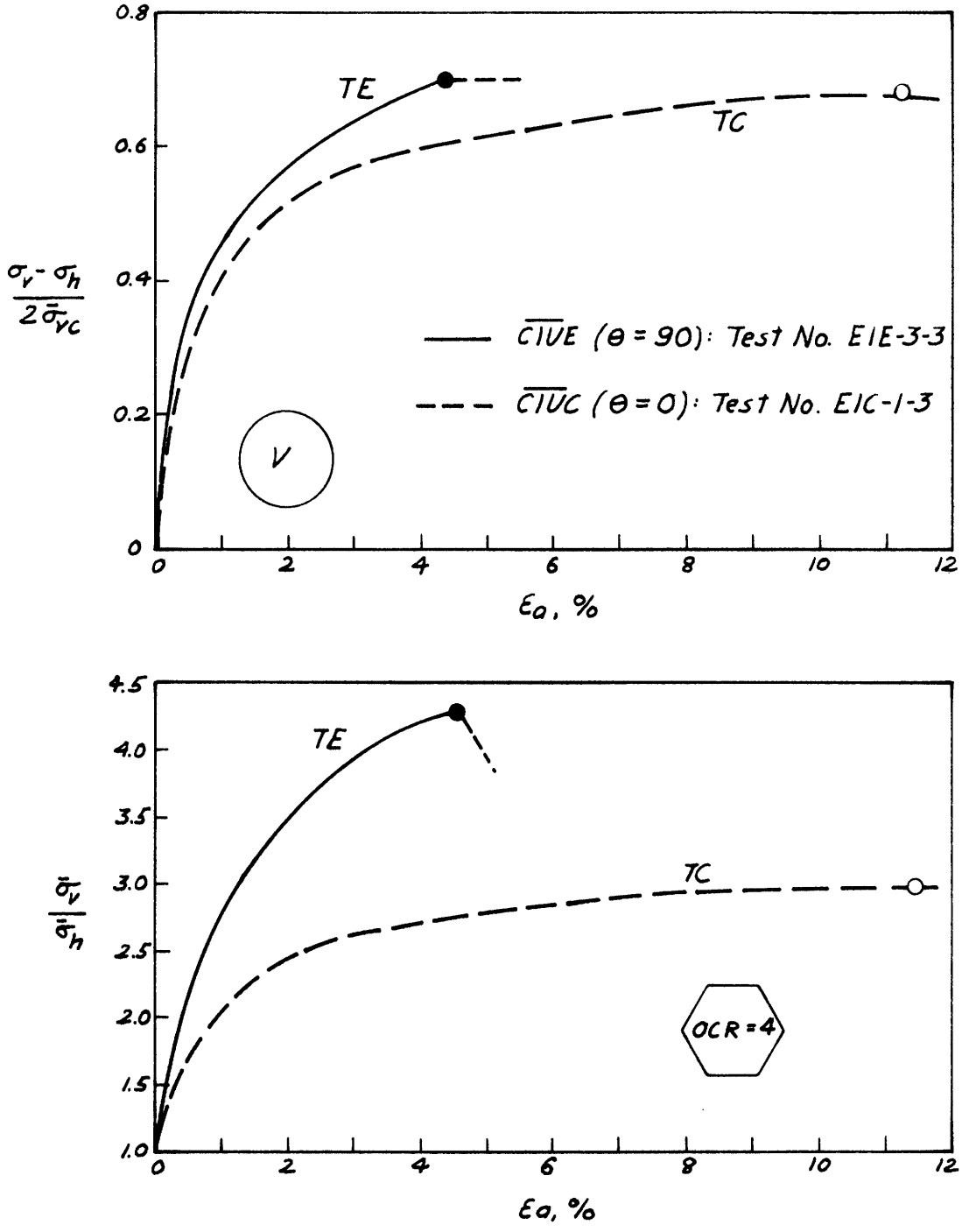
FIGURE 5.14





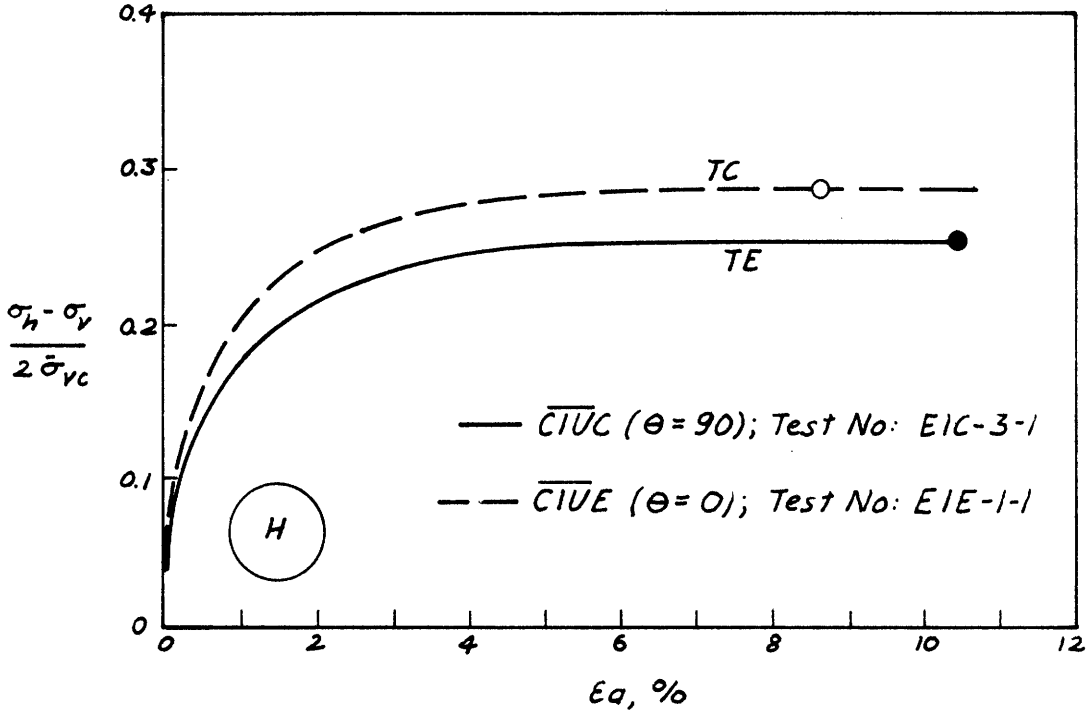
NORMALIZED STRESS-STRAIN CURVES FOR NORMALLY CONSOLIDATED $\overline{CIUE}$ (θ = 90) AND $\overline{CIUC}$ (θ = 0) TEST SAMPLES (VERTICAL LOADING)

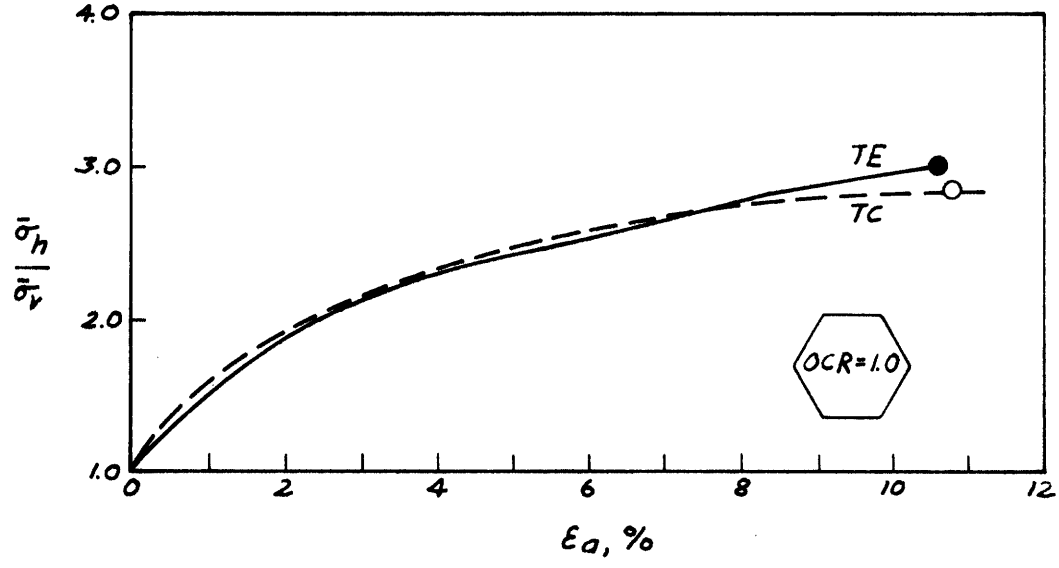
FIGURE 5.15 (a)



NORMALIZED STRESS-STRAIN CURVES FOR OVER-CONSOLIDATED $\overline{CIUE}$ ($\theta = 90$) AND $\overline{CIUC}$ ($\theta = 0$) TEST SAMPLES (OCR = 4.0)

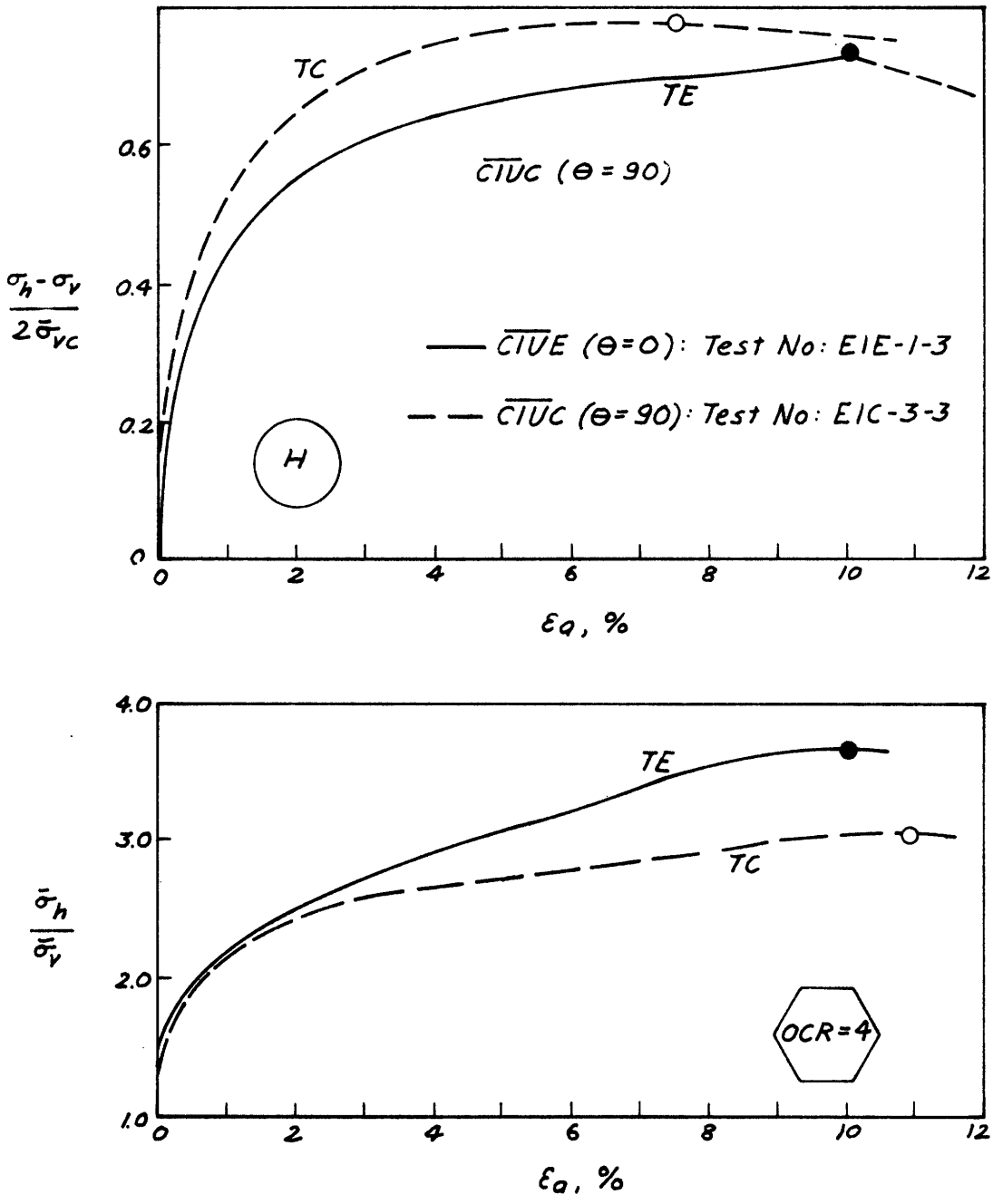
FIGURE 5.15 (b)





NORMALIZED STRESS-STRAIN CURVES FOR NORMALLY CONSOLIDATED $\overline{CIUE}$ ($\theta = 0$) AND $\overline{CIUC}$ ($\theta = 90$) TEST SAMPLES (HORIZONTAL LOADING)

FIGURE 5.16 (a)



NORMALIZED STRESS-STRAIN CURVES FOR OVERCONSOLIDATED $\overline{CIUE}$ (θ=0) AND $\overline{CIUC}$ (θ=90) TEST SAMPLES (OCR = 4.0)

FIGURE 5.16(b)

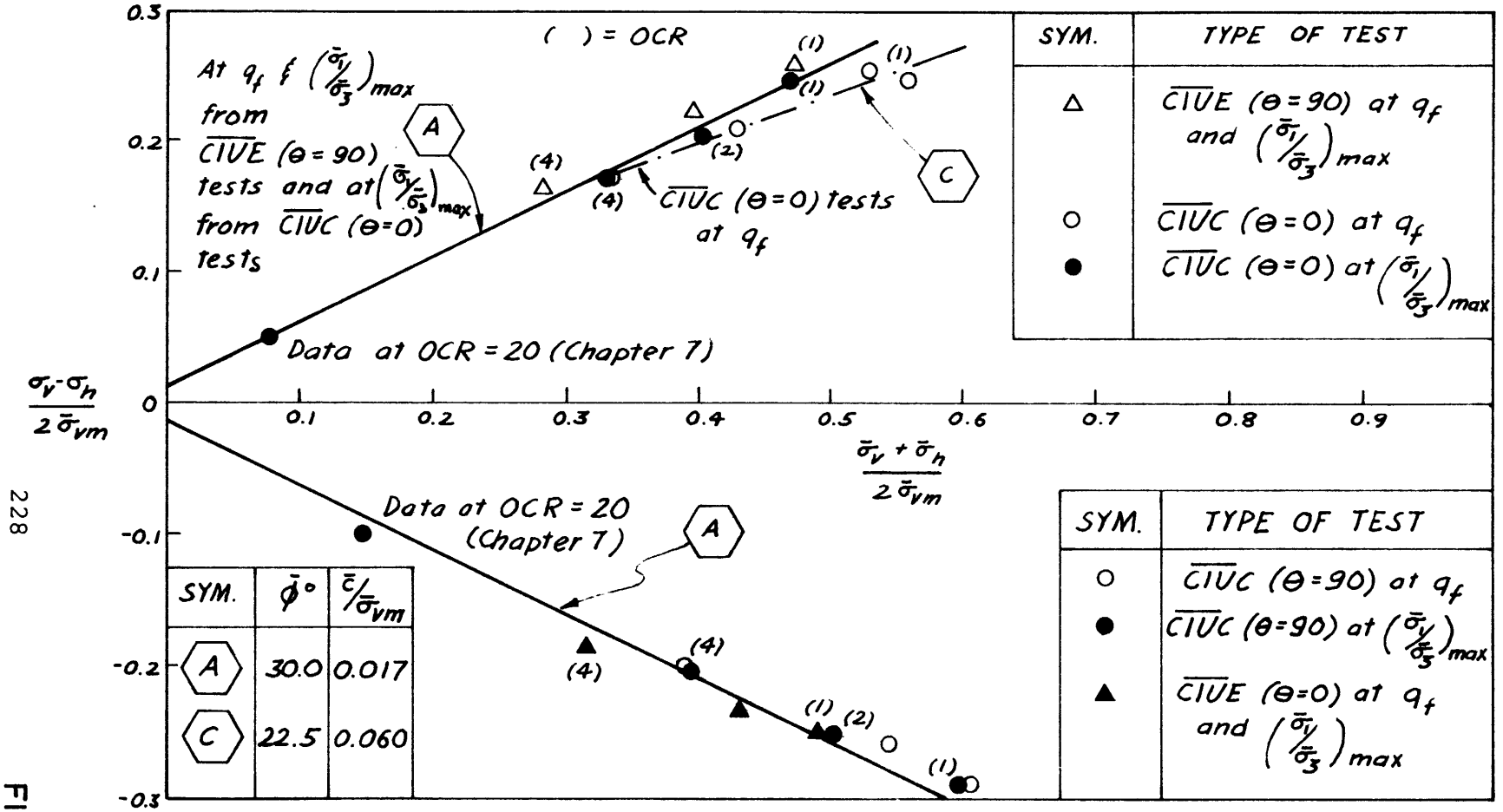


FIGURE 5.17

NORMALIZED EFFECTIVE STRESS ENVELOPES FROM $\overline{CIUC}$ AND $\overline{CIUE}$ TESTS ON VERTICAL AND HORIZONTAL SAMPLES OF CONNECTICUT VALLEY VARVED CLAYS

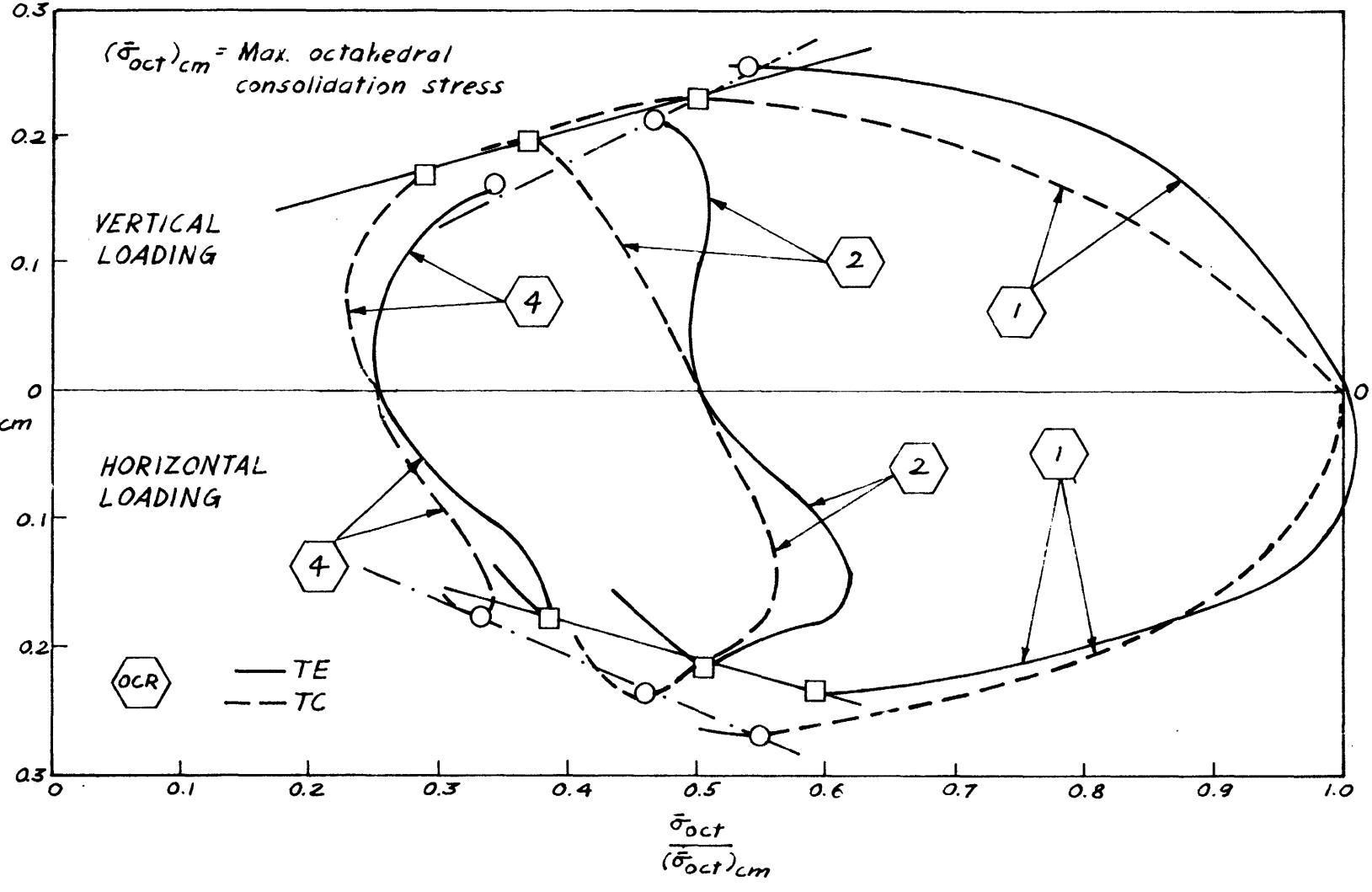


FIGURE 5.18

EFFECT OF INTERMEDIATE PRINCIPAL STRESS ON NORMALIZED OCTAHEDRAL STRESS PATHS

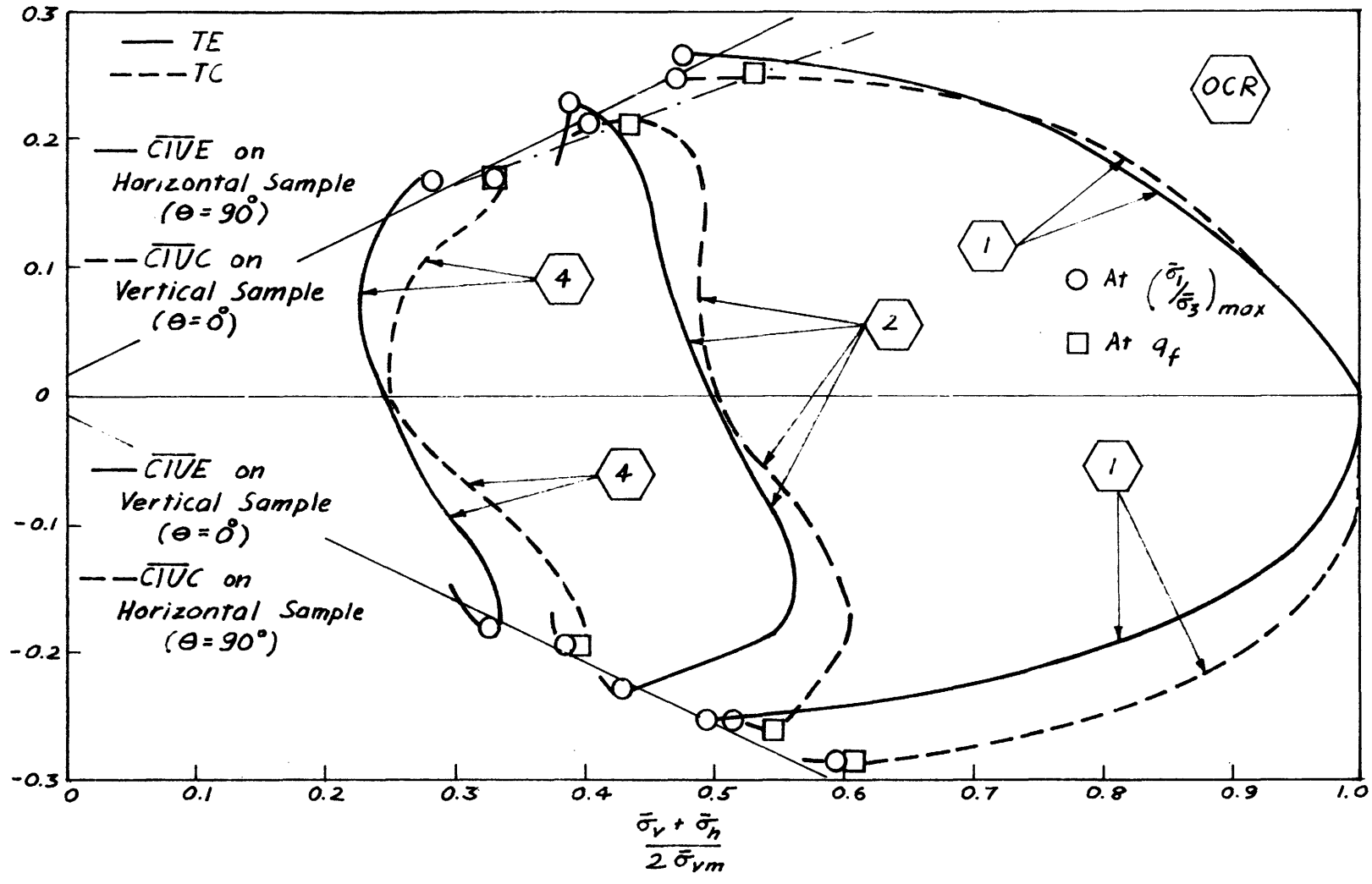
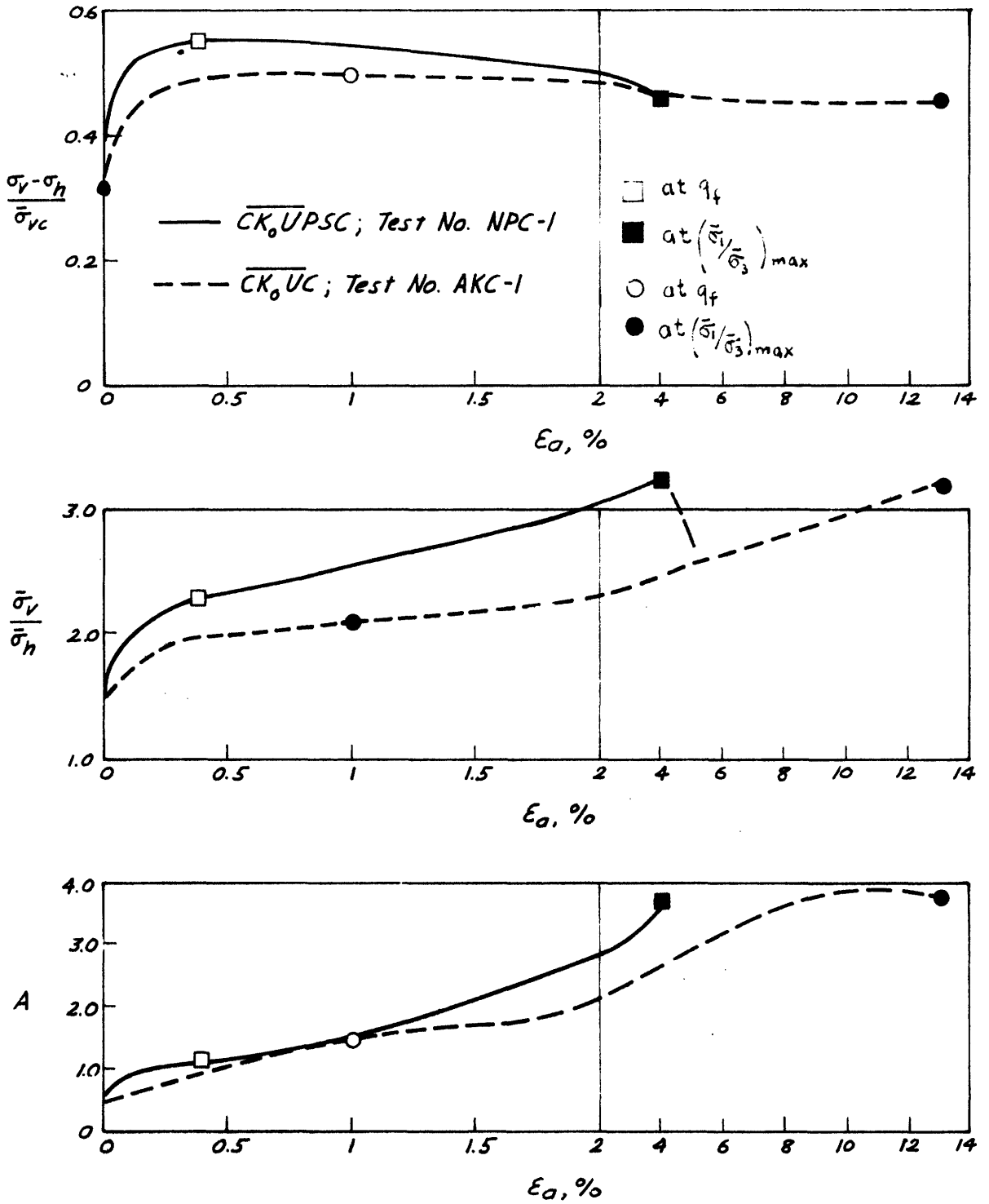
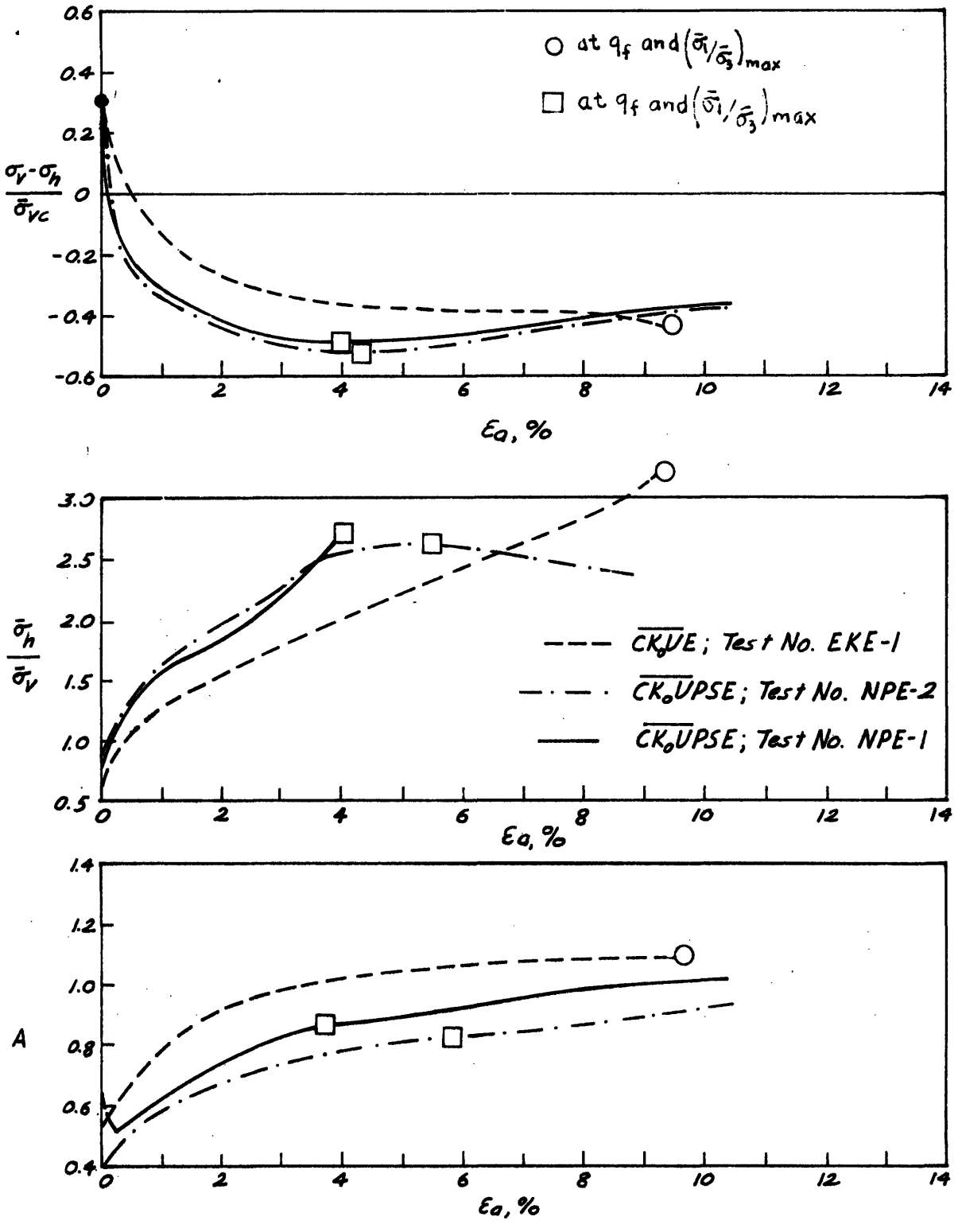


FIGURE 5.19 EFFECTS OF σ_2: NORMALIZED EFFECTIVE STRESS PATHS FROM $\overline{CIUC}$ AND $\overline{CIUE}$ TESTS ON VERTICAL AND HORIZONTAL SAMPLES OF EAST WINDSOR VARVED CLAY



NORMALIZED STRESS-STRAIN CURVES OF NORMALLY CONSOLIDATED SAMPLES FROM $\overline{CK_oU}$C AND $\overline{CK_oU}$PSC TESTS

FIGURE 5.20



NORMALIZED STRESS-STRAIN CURVES OF NORMALLY CONSOLIDATED SAMPLES FROM $\overline{CK_oU}E$ AND $\overline{CK_oU}PSE$ TESTS

FIGURE 5.21

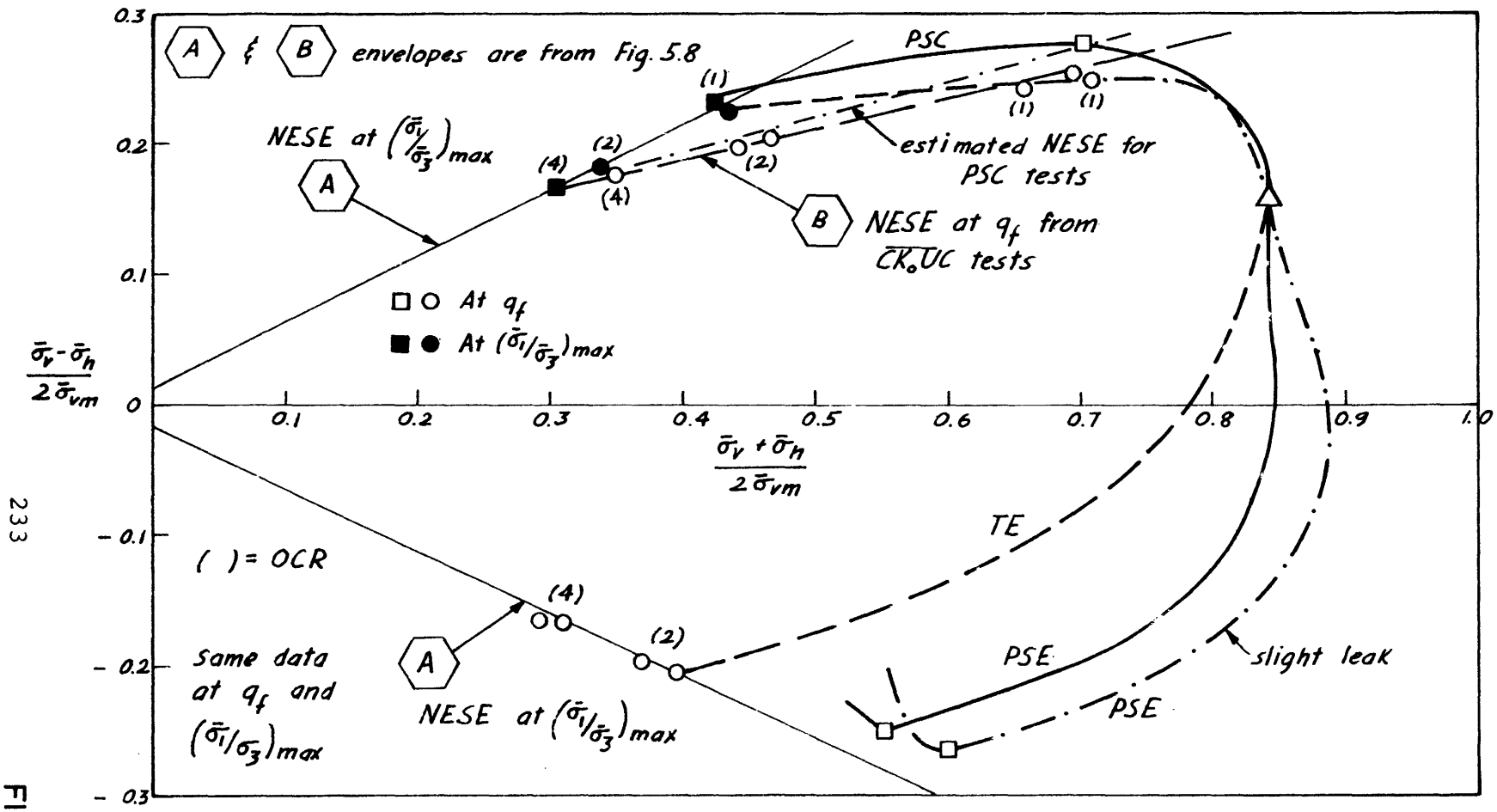
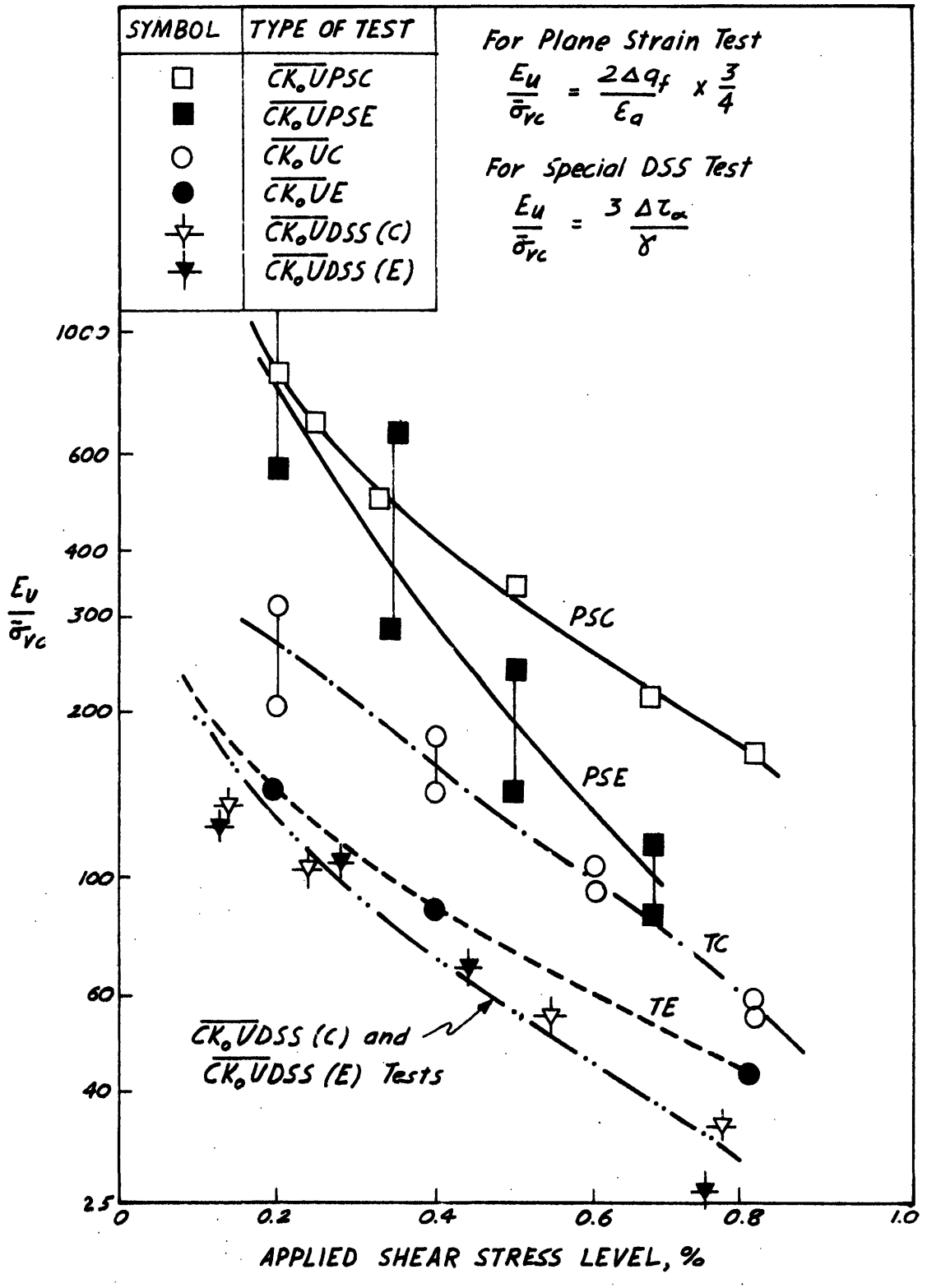


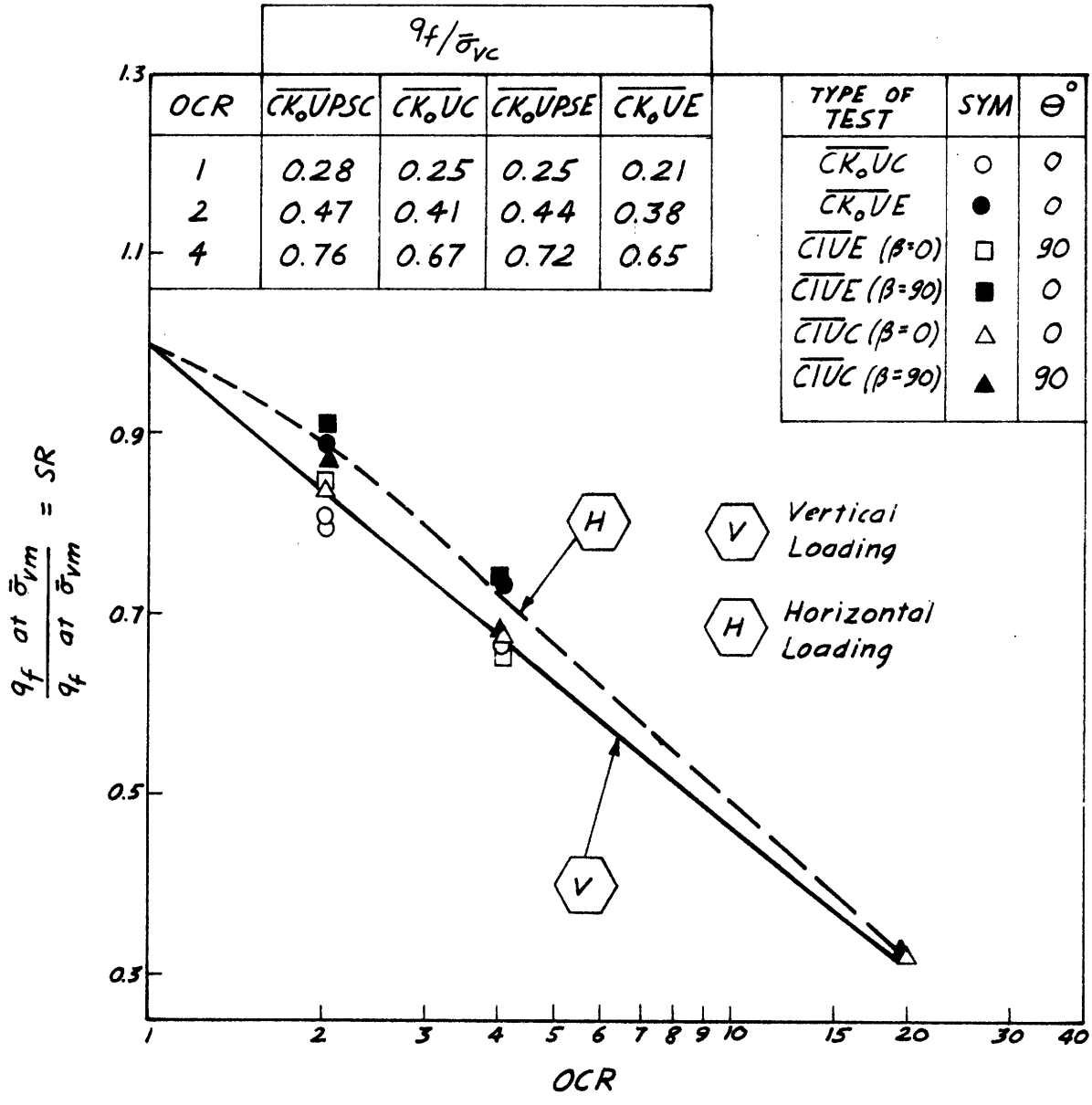
FIGURE 5.22

NORMALIZED EFFECTIVE STRESS PATHS FOR PLANE STRAIN AND TRIAXIAL SHEAR OF NORMALLY CONSOLIDATED CONNECTICUT VALLEY VARVED CLAYS



COMPARISON OF $\frac{E_u}{\bar{\sigma}_{vc}}$ FROM $\overline{CK_oU}$ PLANE STRAIN, TRIAXIAL, AND SPECIAL DSS TESTS

FIGURE 5.23



ESTIMATION OF $\frac{q_f}{\bar{\sigma}_{vc}}$ OF OVERCONSOLIDATED PLANE STRAIN HORIZONTAL AND VERTICAL LOADING SAMPLES FROM CONNECTICUT VALLEY VARVED CLAYS

FIGURE 5.24

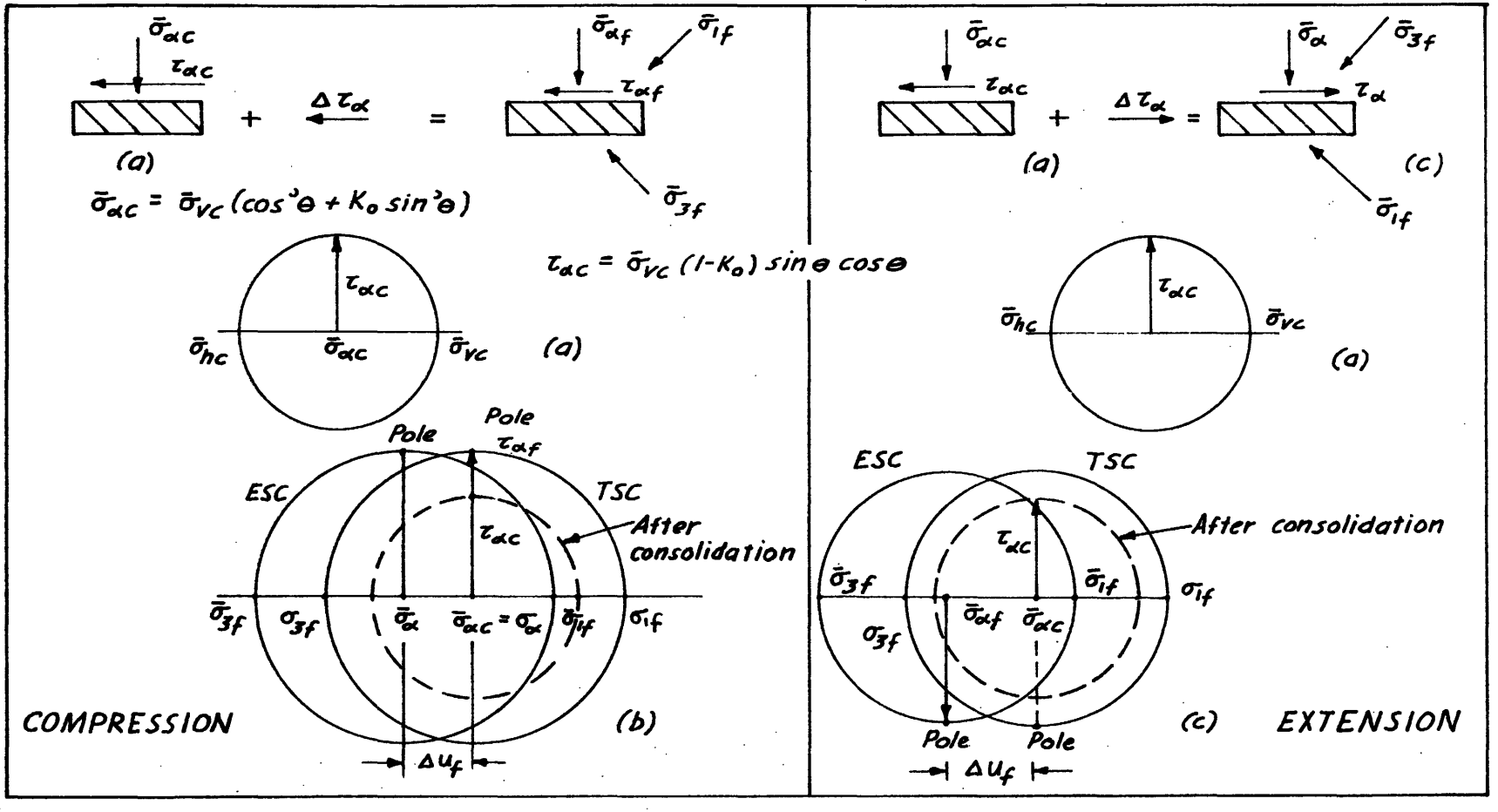


FIGURE 5.25

ASSUMED STATE OF STRESS (PURE SHEAR) FOR MEASURING s_u(V) AND s_u(H) OF INCLINED (45°) NORMALLY CONSOLIDATED SAMPLES FROM $\overline{CK_oU}$DSS(C) AND $\overline{CK_oU}$DSS(E) TESTS

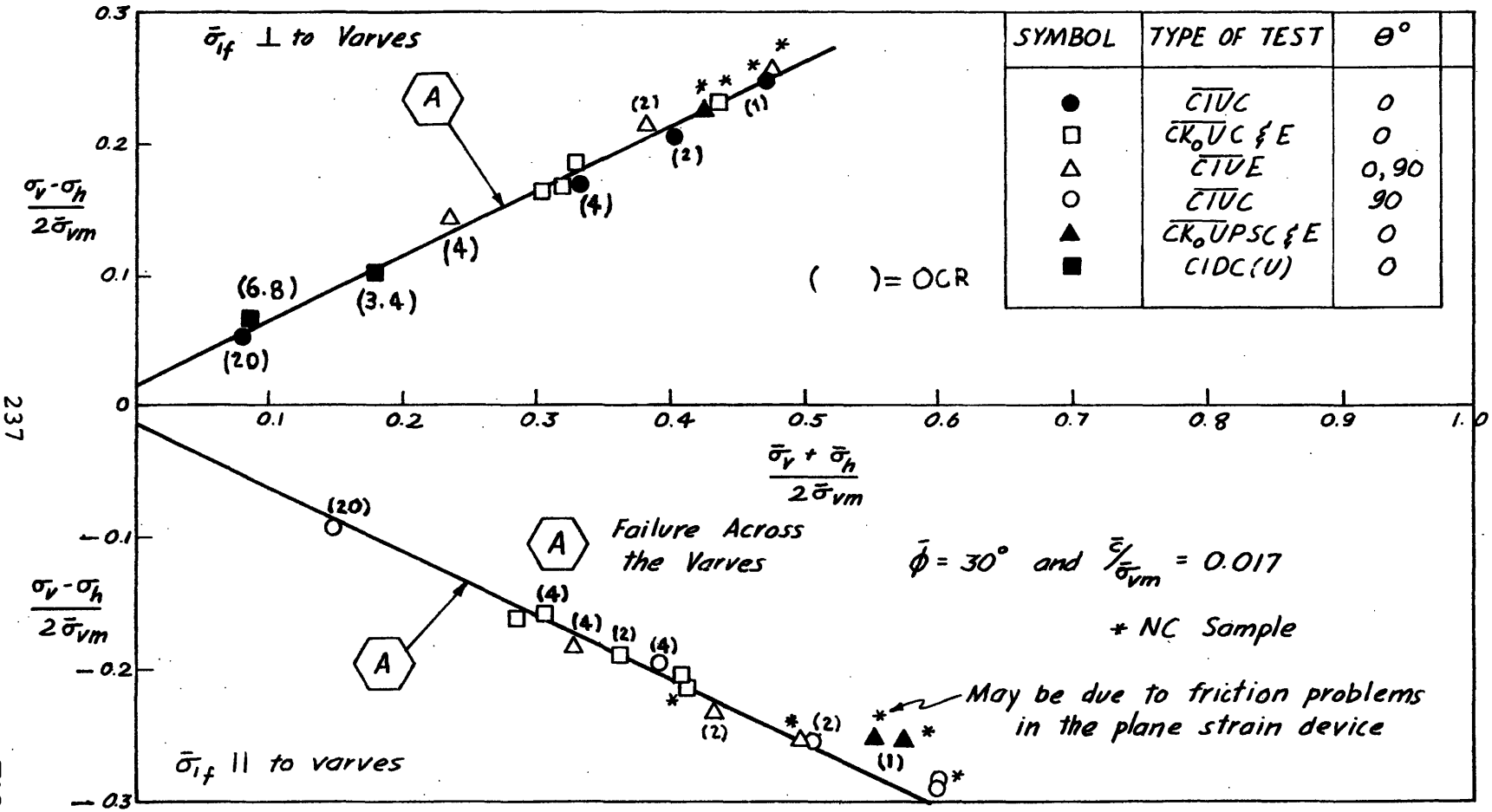
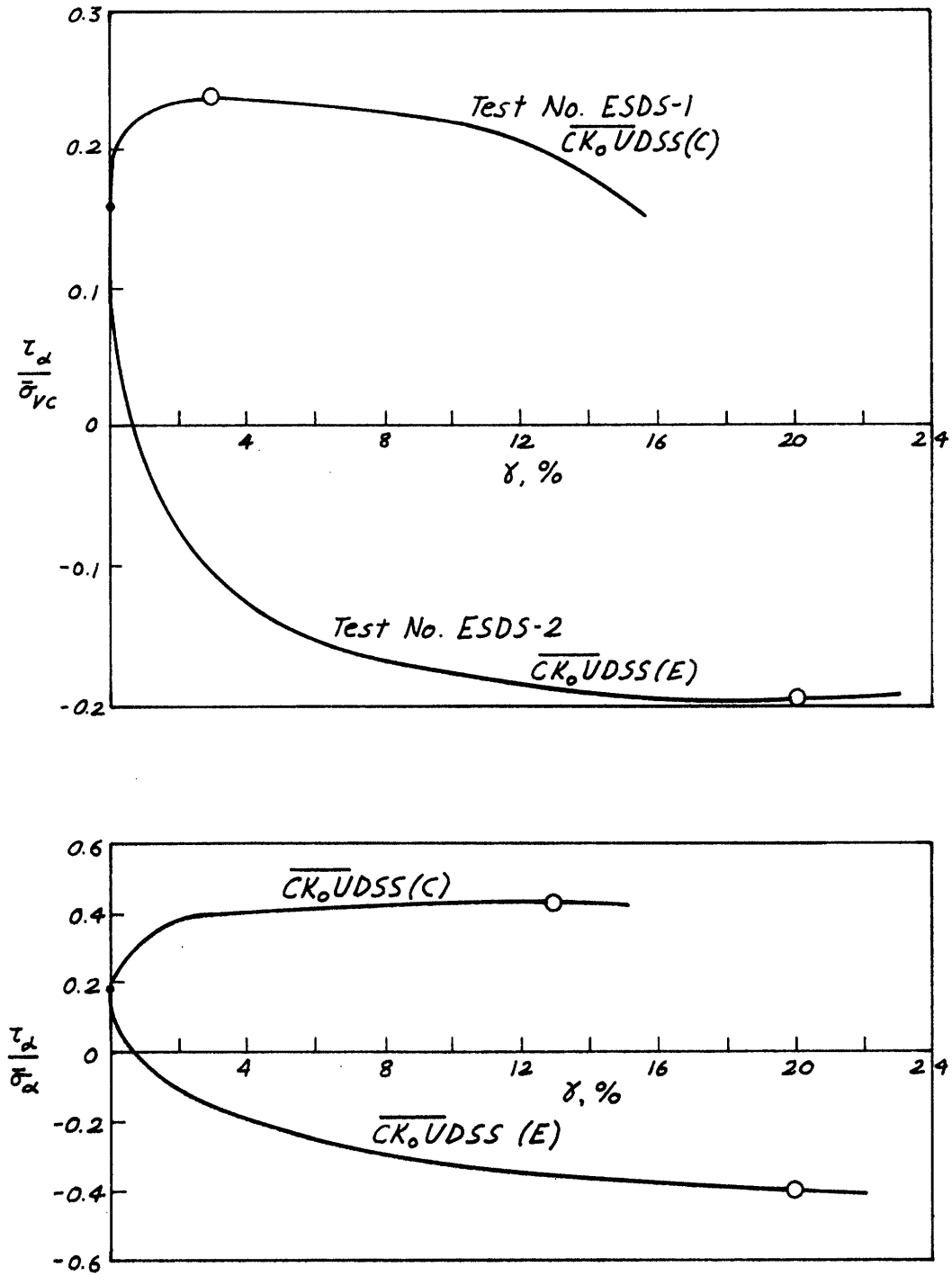


FIGURE 5.26 NESE AT $(\frac{\bar{\sigma}_1}{\bar{\sigma}_3})_{max}$ FOR FAILURE ACROSS THE VARVES



NORMALIZED STRESS-STRAIN CURVES OF NORMALLY CONSOLIDATED INCLINED SAMPLES FROM $\overline{CK_oU}$DSS(C) AND $\overline{CK_oU}$DSS(E) TESTS

FIGURE 5.27

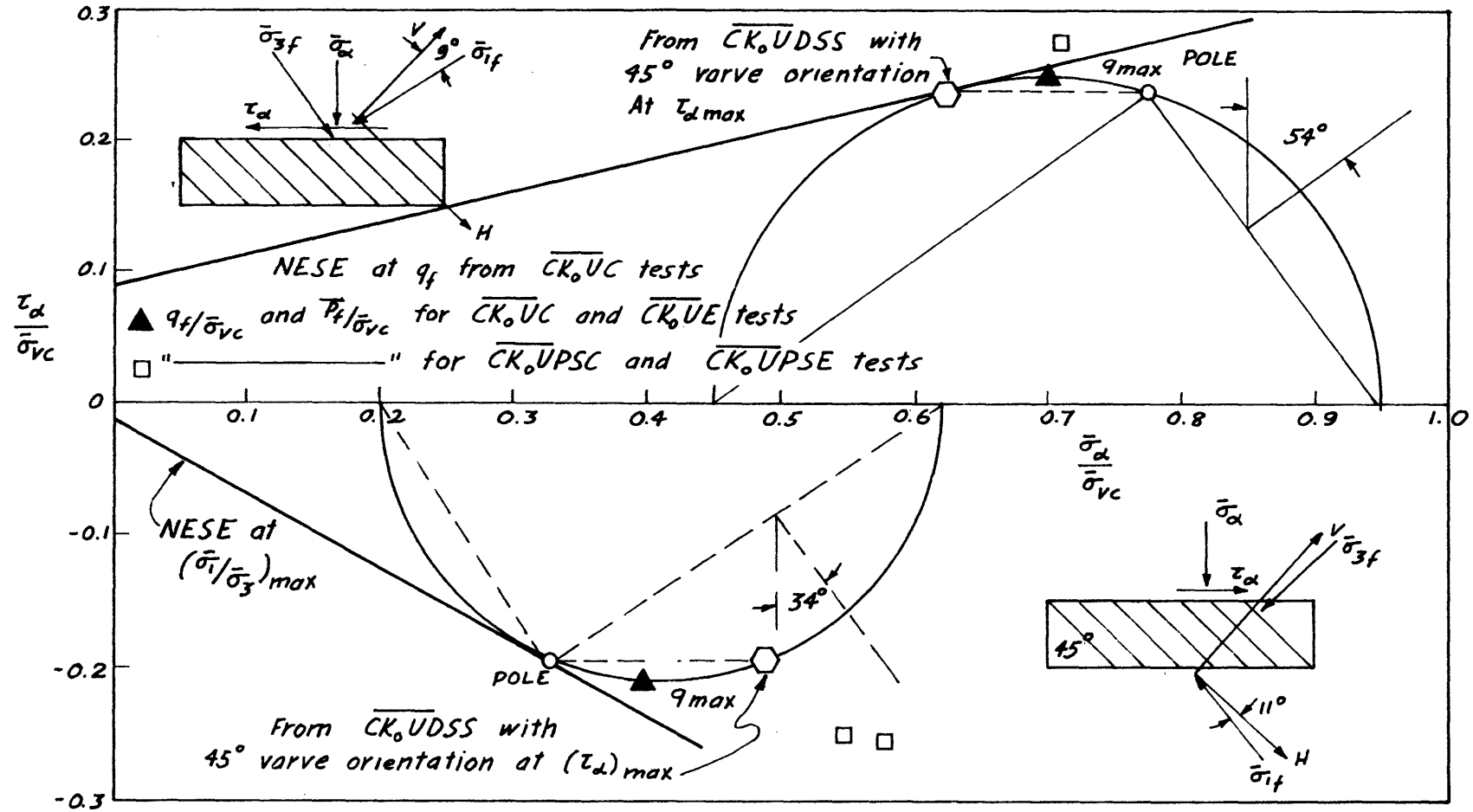
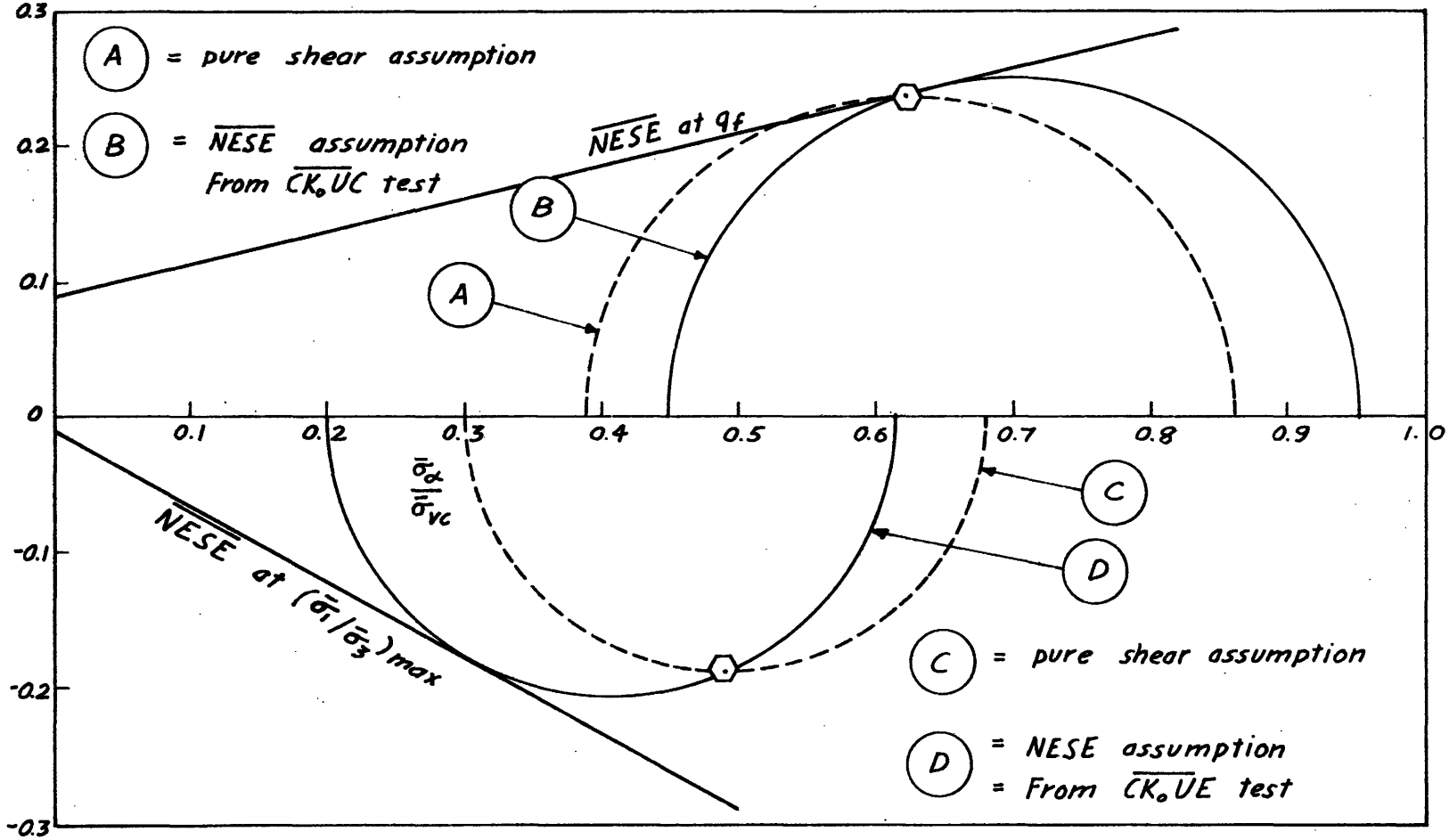


FIGURE 5.28 COMPARISON OF $\frac{q_f}{\bar{\sigma}_{vc}}$ FROM $\overline{CK_oU}$ TRIAXIAL TESTS AND $\overline{CK_oU}$DSS TEST ON INCLINED SAMPLES FROM EAST WINDSOR (USING $\overline{NESE}$ ASSUMPTION)



COMPARISON OF $\frac{q_f}{\bar{\sigma}_{vc}}$ VALUES FROM $\overline{NESE}$ ASSUMPTION AND THE PURE SHEAR ASSUMPTION

FIGURE 5.29

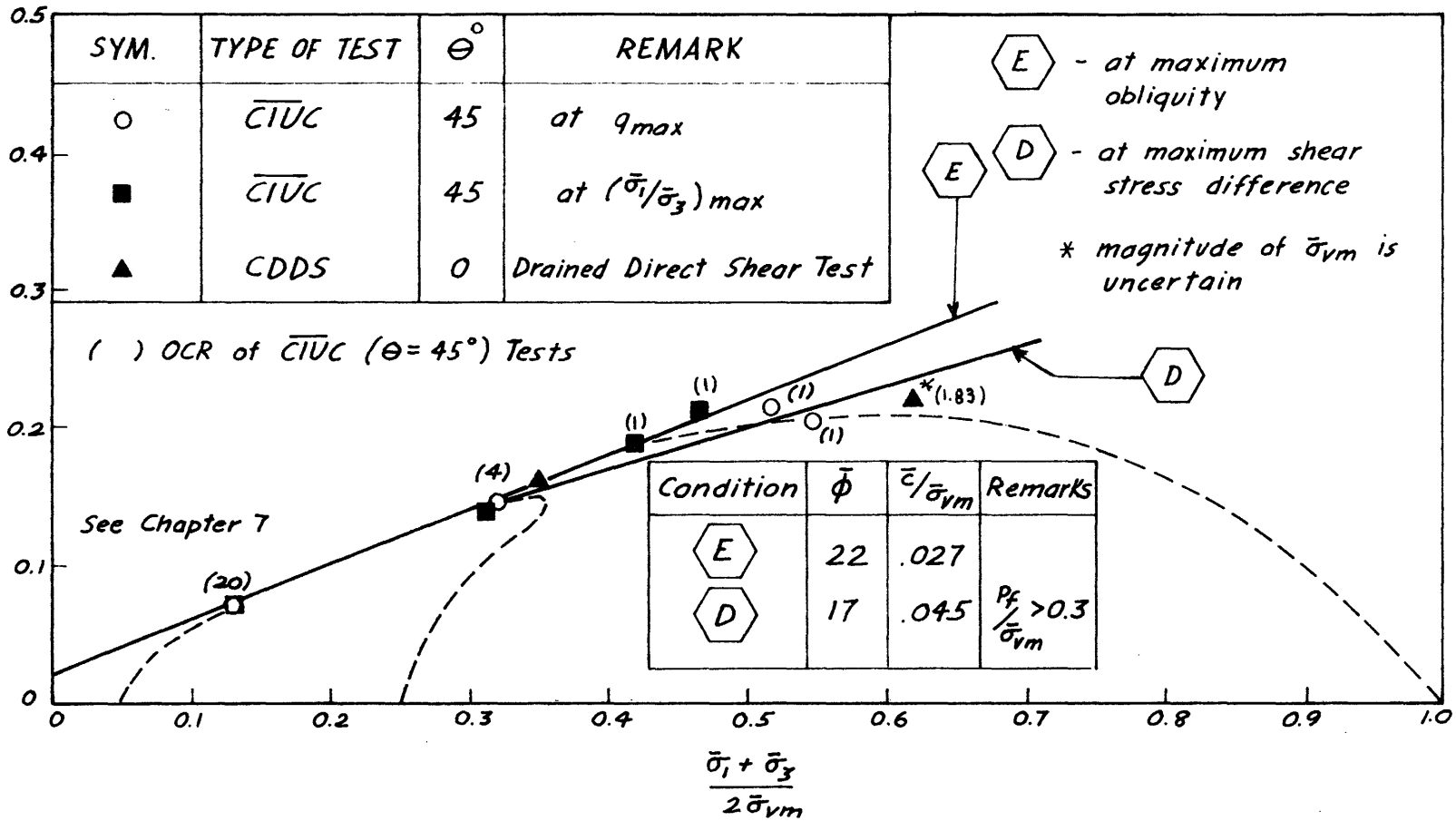
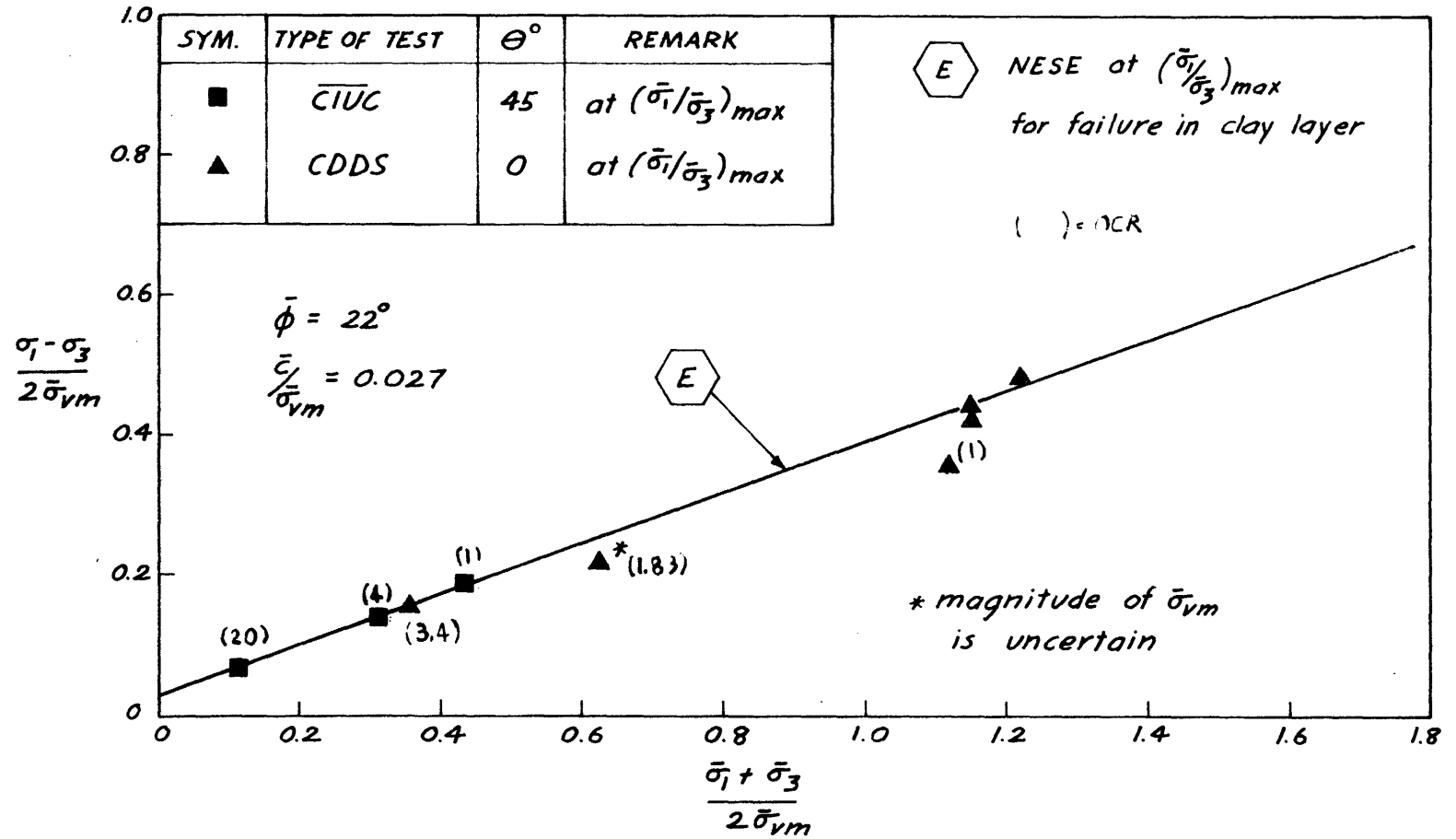
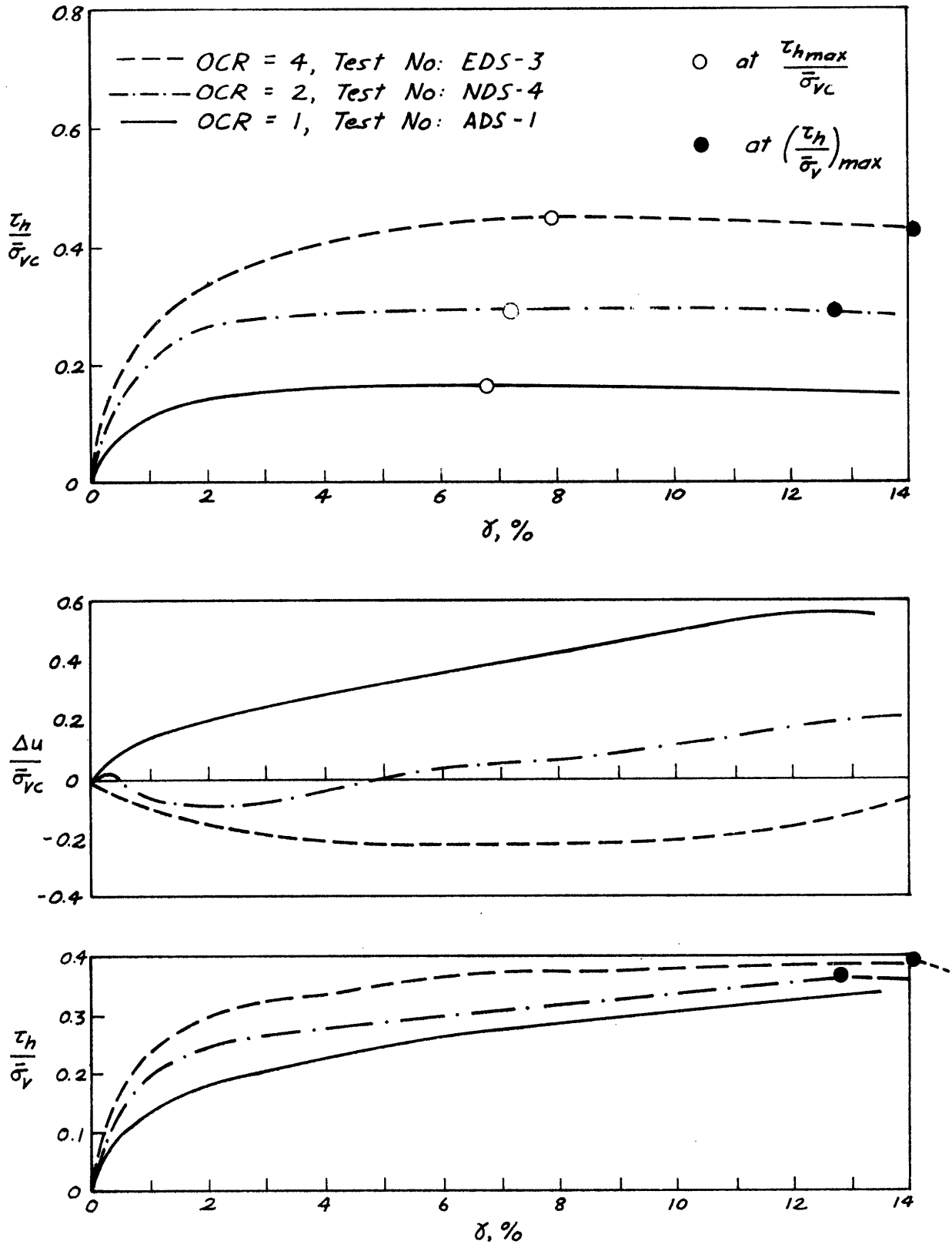


FIGURE 5.30 NESE AT q_f AND $(\frac{\bar{\sigma}_1}{\bar{\sigma}_3})_{max}$ FROM C̄IŪC TESTS ON INCLINED SAMPLES AND CDDS TESTS



NESE AT $\left(\frac{\sigma_1}{\bar{\sigma}_3}\right)_{max}$ FOR FAILURE WITHIN THE CLAY LAYER

FIGURE 5.31



TYPICAL NORMALIZED STRESS-STRAIN CURVES FROM $\overline{CK_oUDSS}$ TESTS ON NC AND OC CONNECTICUT VALLEY VARVED CLAYS FIGURE 5.32

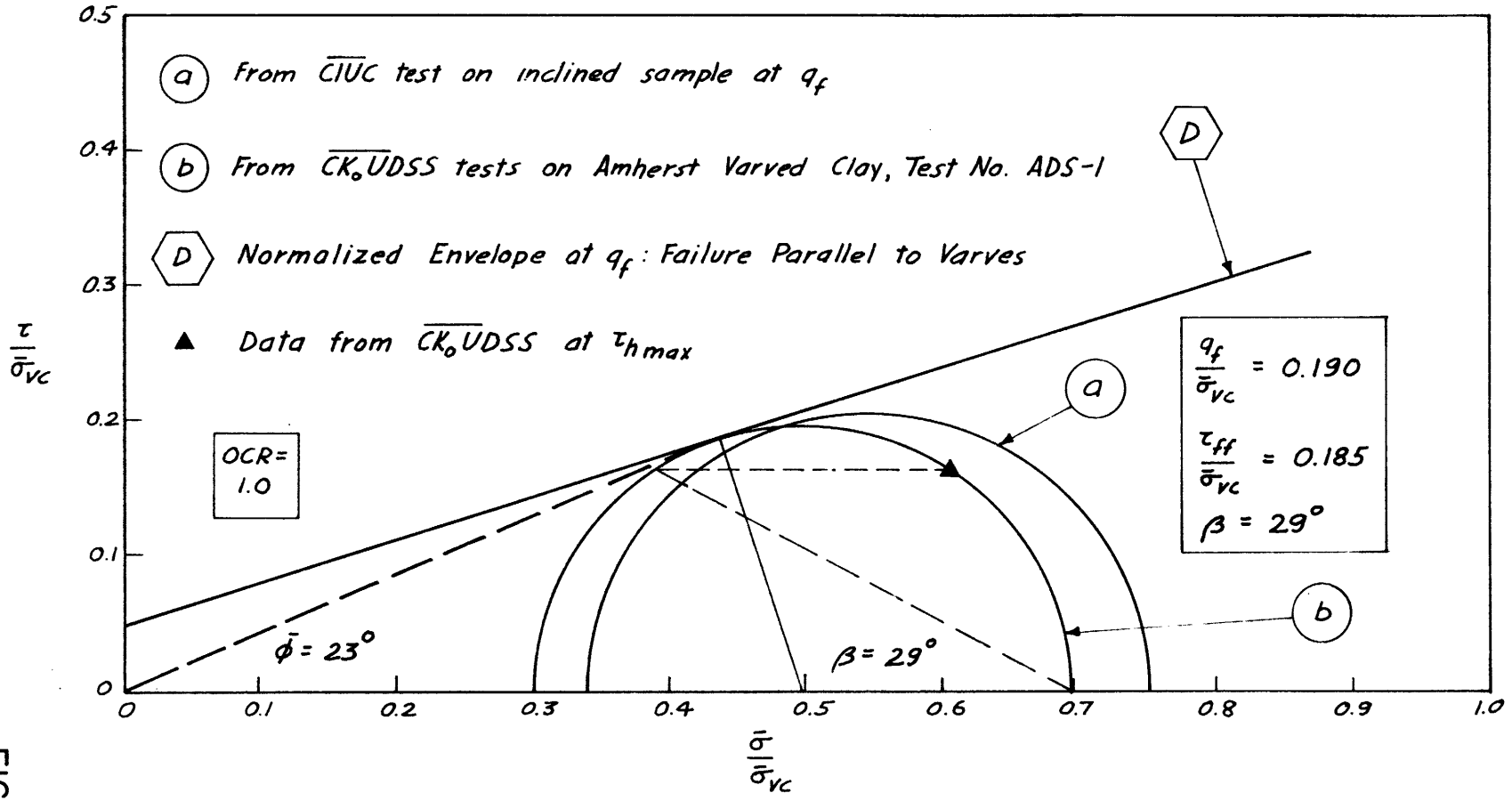
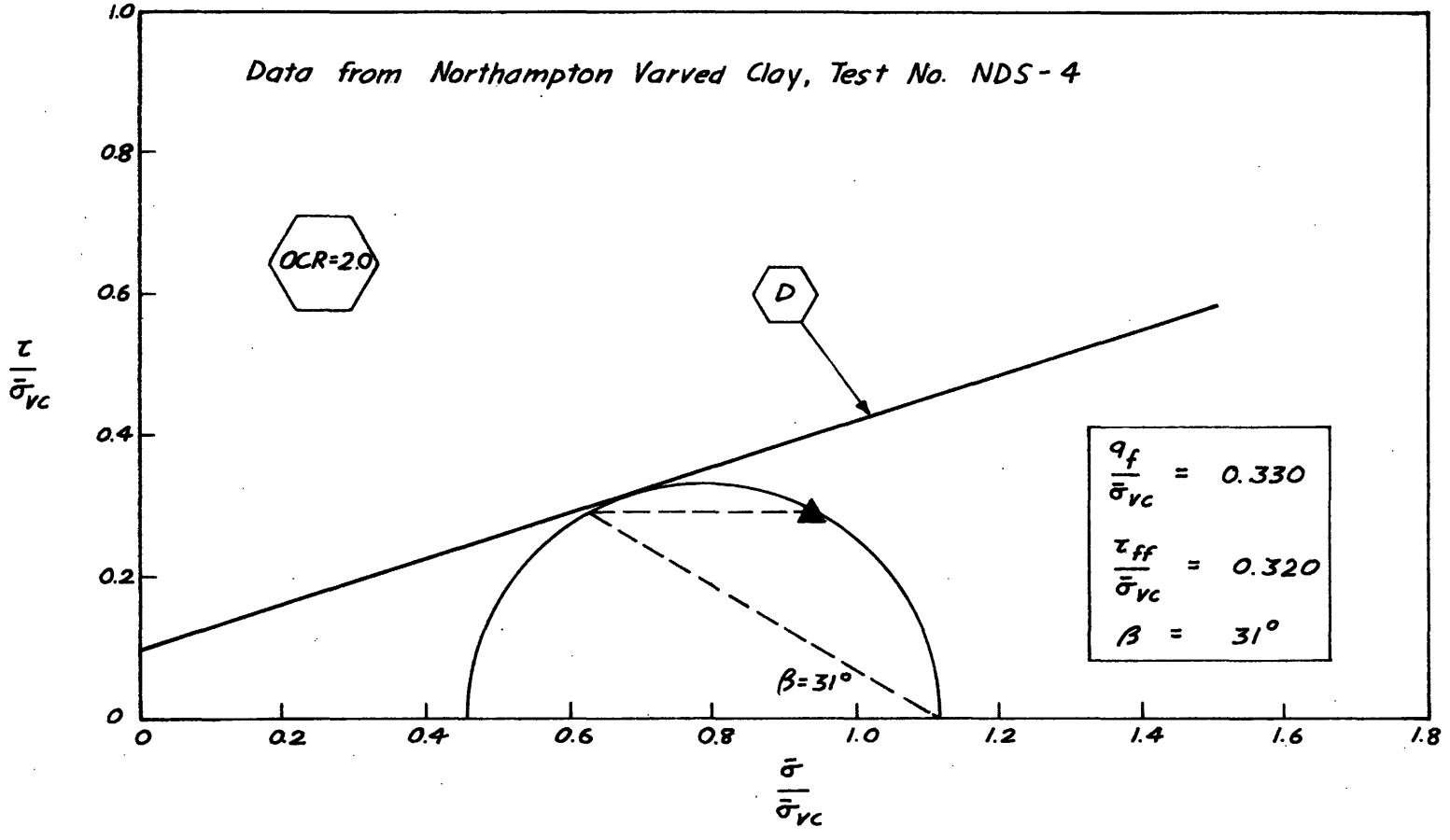


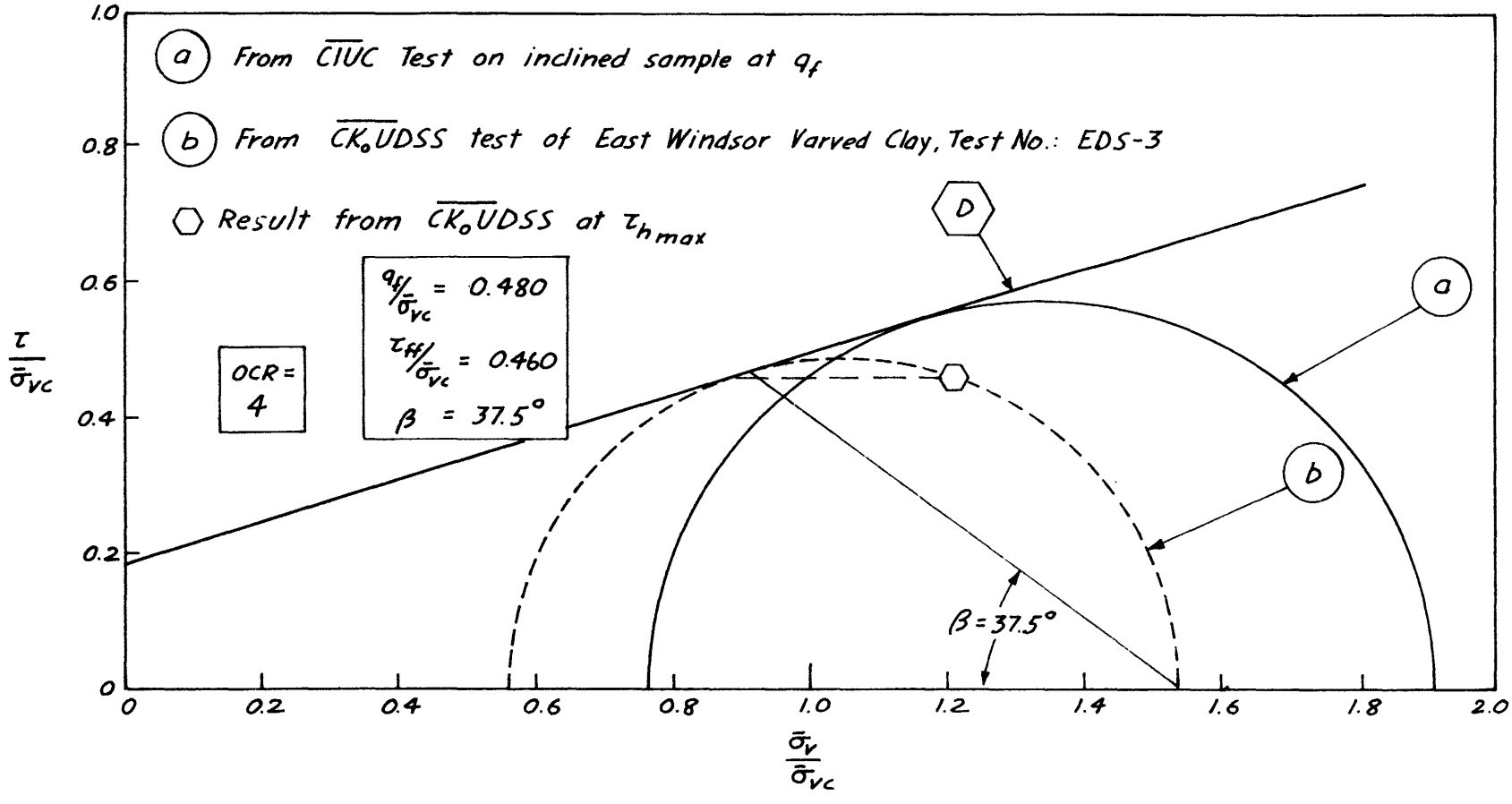
FIGURE 5.33

METHOD OF COMPUTING q_{max} FROM $\overline{CK_oUDSS}$ TEST ON NORMALLY CONSOLIDATED AMHERST VARVED CLAY
(Assumed NESE at q_f from $\overline{CIUC}$ tests on inclined samples)



METHOD OF COMPUTING q_f FROM $\overline{CK_oUDSS}$ TEST OF OVER-CONSOLIDATED (OCR = 2) NORTHAMPTON VARVED CLAY AT $\tau_{h(max)}$ (Assumed NESE at q_f from $\overline{CIUC}$ tests on inclined samples)

FIGURE 5.34



METHOD OF COMPUTING q_f FROM $\overline{CK_oU}$DSS TEST OF OVER-CONSOLIDATED (OCR = 4.0) EAST WINDSOR VARVED CLAY AT $\tau_{h(max)}$
(Assumed $\overline{NESE}$ at q_f from $\overline{CIUC}$ tests on inclined samples)

FIGURE 5.35

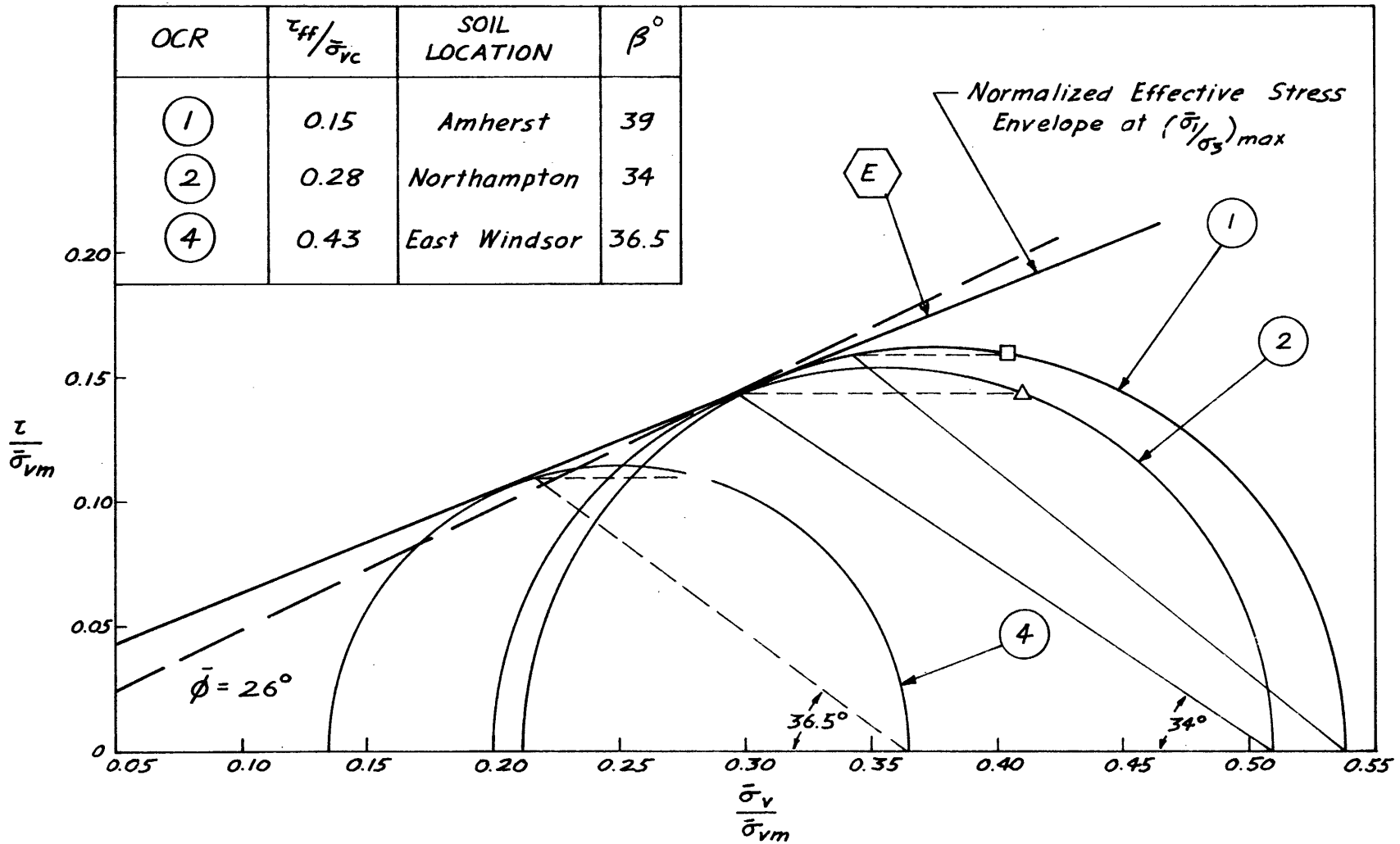
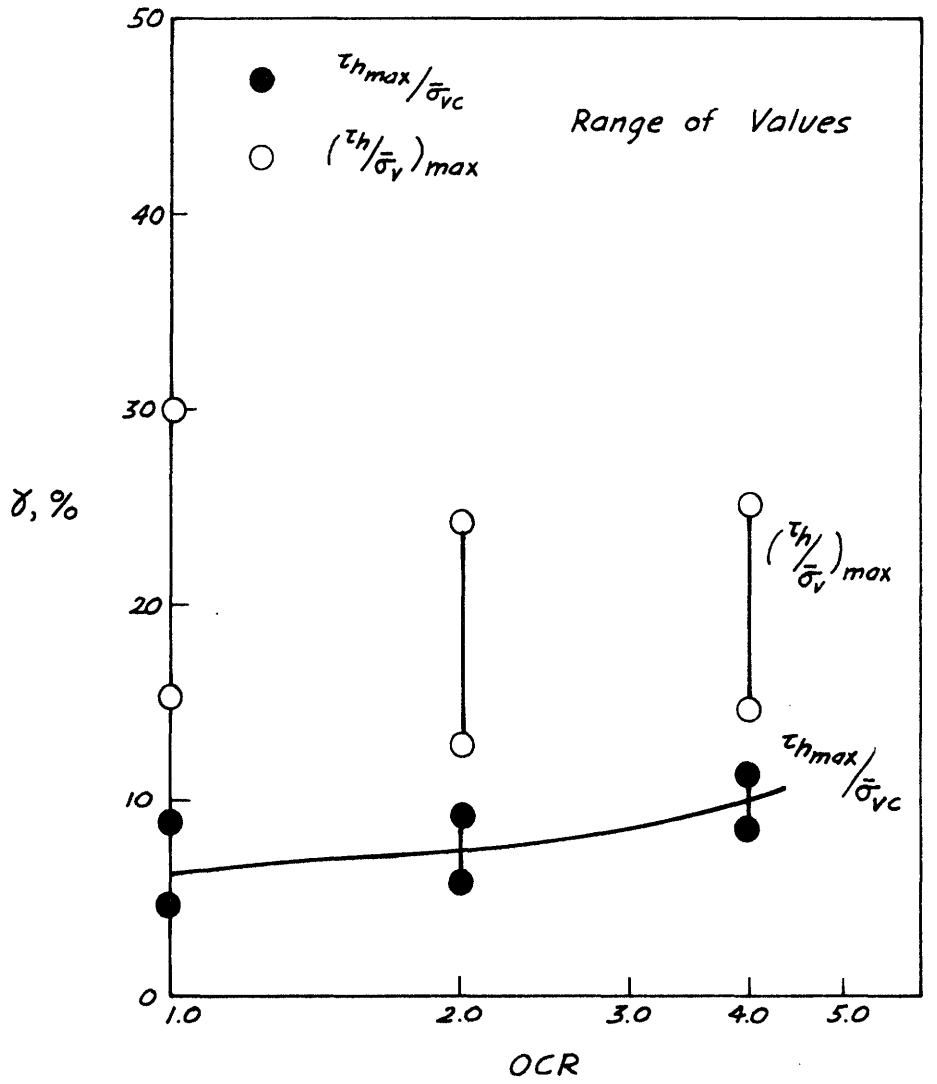


FIGURE 5.36 DETERMINATIONS OF τ_{ff} AT $(\frac{\tau_h}{\bar{\sigma}_v})_{max}$ FROM $\overline{CK_oUDSS}$ TESTS



FAILURE STRAINS AT τ_{hmax} AND $(\frac{\tau_h}{\bar{\sigma}_v})_{max}$ FROM $\overline{CK_oU}$DSS TESTS ON CONNECTICUT VALLEY VARVED CLAYS

FIGURE 5.37

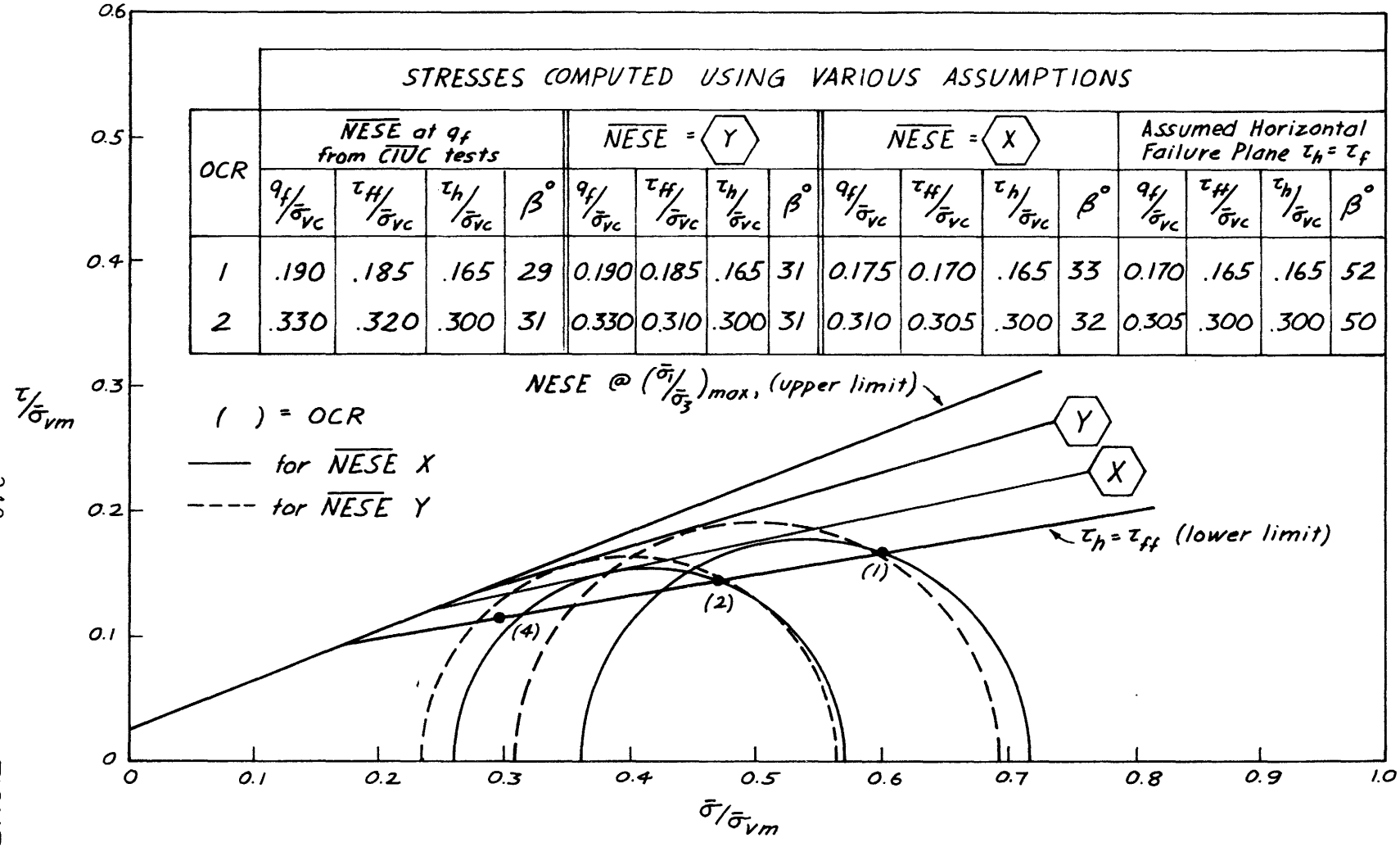


FIGURE 5.38

COMPUTATION OF RELATIVE MAGNITUDES OF $\frac{q_f}{\bar{\sigma}_{vc}}$, $\frac{\tau_{ff}}{\bar{\sigma}_{vc}}$, AND $\frac{\tau_{h_{max}}}{\bar{\sigma}_{vc}}$ BY ASSUMING TWO LOCATIONS OF $\overline{NESE}$

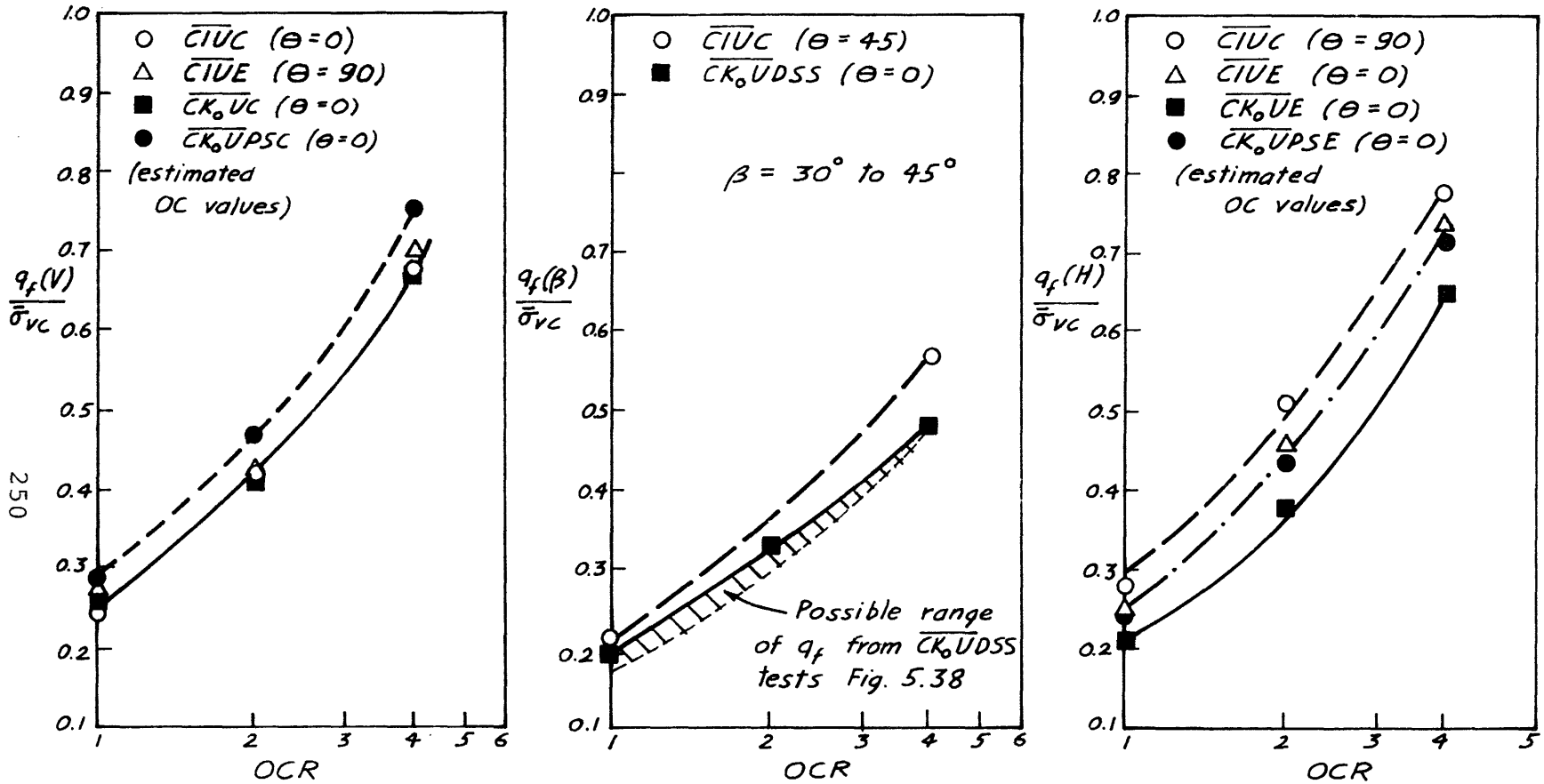


FIGURE 5.39 COMPARISON OF THE EFFECT OF STRESS SYSTEM ON $\frac{q_f}{\bar{\sigma}_{vc}}$ VS OCR FOR THREE DIFFERENT DIRECTION OF LOADINGS

6. CONSIDERATION OF SAMPLE DISTURBANCE

6.1 INTRODUCTION AND BACKGROUND

6.1.1 Purpose

Sample disturbance in homogeneous clays has been studied fairly extensively (e.g., Ladd and Lambe, 1963; Davis and Poulos, 1967; Bozozuk, 1970; and Milovic, 1970), but disturbance in varved clay has received little attention. The special problems with varved clays include formation of transverse cracks located at the interface between clay and silt portions due to sampling technique (Kenney and Chan, 1972) and separation of varves during trimming of samples (Saxena et al., 1974). In this chapter, methods for minimizing disturbance in varved clay are studied for the purpose of providing: (1) comparison of NSP ($q_f/\bar{\sigma}_{vm}$, A_f, $E_u/\bar{\sigma}_{vc}$, $\bar{\phi}$ and $\bar{c}/\bar{\sigma}_{vm}$) from these testing methods: the UUC test, the RECOMPRESSION method, and the SHANSEP method; and (2) recommended methods for measuring the in situ strength-deformation properties of varved clay.

6.1.2 Types and Causes of Sample Disturbance

There are two basic types of sample disturbance in varved clay, namely:

1. Disturbance due to the removal of the in situ stresses which caused a change from anisotropic stresses to an equivalent isotropic stress, $\bar{\sigma}_{ps}$; and
2. Disturbance due to sampling, handling, trimming, and setting up a test, all of which may cause

permanent damage to the soil structure, and with varved clays, cause separations or transverse cracks located at the interfaces between silt and clay layers. The separations are most likely to occur between the varves.

The formation of cracks or separations are the result of sampling techniques (Kenney and Chan, 1972), and trimming, especially with inclined and horizontal samples (Saxena et al., 1974). Kenney and Chan (1972) concluded that the formation of transverse cracks due to sampling of varved clay samples was primarily the result of the large friction force created between the coarser-grained soil (silt portion) and the sampling tube. The formations of cracks or separations resulting from trimming samples are difficult to avoid, even when a thin wire is used. Such an effect will be very pronounced in horizontal samples, slightly less pronounced in inclined samples, and will be negligible in vertical samples.

6.1.3 General Effect of Sample Disturbance

During sampling, the change in effective stress due to the removal of the in situ stress is unavoidable. Even for the "perfect sample," defined as a sample which undergoes only an undrained release of the in situ stresses to end up with an isotropic effective stress equal to $\bar{\sigma}_{ps}$ (Ladd and Lambe, 1963), the strength and undrained modulus of the soil are affected. As a result of structural damage (including both soil structure damage and separation of varves), and expansion (swelling) of

the sample, during sampling, extruding, trimming, and setting up of a test, the isotropic effective stress in a disturbed sample is decreased from $\bar{\sigma}_{ps}$ to $\bar{\sigma}_{s}$ (the effective stress of the sample prior to a test). Such a reduction in effective stress in turn causes an additional decrease in undrained shear strength (q_f) and Young's modulus (E). In addition, planes of weakness at the interfaces between clay and silt portions, which are created by the transverse cracks or separations (i.e., part of structural damage) may cause further decreases in undrained shear strength and modulus, especially when $\bar{\sigma}_{lf}$ acts parallel to the varves and when shearing is parallel to the varves. This would produce a directional dependence of sample disturbance.

6.1.4 Methods for Minimizing Sample Disturbance

Methods for minimizing sample disturbance to be studied are the UUC test, the RECOMPRESSION method, and the SHANSEP method. Since the cracks or separations can be closed by pressure, the UUC test (where a cell pressure is used to confine the sample) is considered as a method for minimizing sample disturbance compared to an unconfined compression test. In the RECOMPRESSION method, the sample is K_o consolidated to the in situ vertical and horizontal stresses (Bjerrum, 1973). It therefore minimizes disturbance by reapplying the field effective stresses, i.e., $\bar{\sigma}_{vc} = \bar{\sigma}_{vo}$ and $\bar{\sigma}_{hc} = \bar{\sigma}_{ho}$. Because of the resultant decrease in moisture content and any prior structural damage (soil structure damage and formations of cracks or

separations located at the interfaces between clay and silt portions), the RECOMPRESSION method may yield a sample having a different soil structure and hence different soil properties compared to the in situ soil.

Research reported by Ladd and Foott (1974) and Ladd et al. (1977) suggest that, if a soil exhibits normalized behavior when $\bar{\sigma}_{vc} > \bar{\sigma}_{vm}$, K_o consolidated SHANSEP samples (especially NC samples) should yield reasonable $q_f/\bar{\sigma}_{vc}$ values compared to those of the in situ soil. The basis of the SHANSEP method proposed by Ladd and Foott (1974) as a means for minimizing sample disturbance effects on strength-deformation properties measured in the laboratory is as follows. As indicated in Fig. 6.1(a), the laboratory curve typically approaches the in situ virgin compression curve at Point C ($\bar{\sigma}_{vc} = \bar{\sigma}_{vm}(L) \geq (1.5\text{-}2)\ \bar{\sigma}_{vm}$). Thus, $\overline{CK_oU}$ test samples consolidated to Point D should have a soil structure similar to the in situ normally consolidated clay and hence yield reasonable $q_f/\bar{\sigma}_{vc}$ values if a soil exhibits normalized behavior. For overconsolidated samples (A and B in Fig. 6.1(a)), the samples are first consolidated into the virgin compression range to obtain a normally consolidated clay (i.e., at Point D), and unloaded prior to shear (shown as Point A and B in Fig. 6.1(a)). These OC samples then should yield reasonable $q_f/\bar{\sigma}_{vc}$ values at the required OCR's.

It is seen in Fig. 6.1(a) that the SHANSEP and RECOMPRESSION samples at the same OCR have different magnitudes of $\bar{\sigma}_{vc}$, e.g., compare $\bar{\sigma}_{vc}$ of Sample A and A' in Fig. 6.1(a). Therefore, the

comparison of the measured soil parameters of these samples must be in normalized form. For the purpose of making extensive comparison of NSP from SHANSEP and RECOMPRESSION methods of consolidation, RECOMPRESSION samples considered in this study will also include those which are consolidated above $\bar{\sigma}_{vo}$ but below $\bar{\sigma}_{vm}$. NSP ($q_f/\bar{\sigma}_{vm}$, A_f, $E_u/\bar{\sigma}_{vc}$, $\bar{\phi}$ and $\bar{c}/\bar{\sigma}_{vm}$) of a RECOMPRESSION sample such as that indicated by B' (Fig. 6.1(a)) will therefore be compared with those of a SHANSEP sample which has the same OCR. Figure 6.1(b) shows a hypothetical plot of $q_f/\bar{\sigma}_{vm}$ versus $\bar{\sigma}_{vc}/\bar{\sigma}_{vm}$(L) from SHANSEP samples A, B, C, and D. Since samples C and D are normally consolidated, their $q_f/\bar{\sigma}_{vm}$ are identical if the normalized behavior concept applies. Figure 6.1(c) shows hypothetical plots of $q_f/\bar{\sigma}_{vm}$ versus $\bar{\sigma}_{vc}/\bar{\sigma}_{vm}$ from RECOMPRESSION and UUC test samples and $q_f/\bar{\sigma}_{vm}$(L) versus $\bar{\sigma}_{vc}/\bar{\sigma}_{vm}$(L) from SHANSEP samples. Because of the difference in the capabilities for minimizing disturbance effects and the use of different methods of consolidation, $q_f/\bar{\sigma}_{vm}$ of the RECOMPRESSION sample at the in situ OCR can be higher or lower than that of the SHANSEP sample. As a result of the smaller preshear effective stress $\bar{\sigma}_s$, $q_f/\bar{\sigma}_{vm}$ of the UUC test sample will be lower than those from the SHANSEP and RECOMPRESSION samples at the in situ OCR.

This chapter compares data obtained from the SHANSEP and RECOMPRESSION methods in an attempt to ascertain their relative advantages and disadvantages. Results from both isotropic and K_o consolidated tests are included since $\overline{CIUC}$ tests on samples cut at different inclinations have been used in practice.

6.1.5 Experimental Program

Table 6.1 presents the experimental program used in the study. Since sample disturbance can affect the undrained shear strength behavior differently for various loading directions, the investigation is made for three loading directions: vertical, inclined (45°), and horizontal. NSP ($q_f/\bar{\sigma}_{vm}$, A_f, $\bar{\phi}$, $\bar{c}/\bar{\sigma}_{vm}$ and $E_u/\bar{\sigma}_{vc}$) of K_o consolidated OC SHANSEP samples, RECOMPRESSION OC samples (K_o and isotropically consolidated), and UUC test samples are compared. High quality block samples (in situ OCR $\tilde{>}$ 4) from East Windsor were primarily used for the UUC and RECOMPRESSION test programs, supplemented by some data from 5 in. diameter tube samples from Amherst and Northampton. Results from SHANSEP samples (presented in Chapters 4 and 5) are used for comparisons shown in Table 6.1.

It should be noted that K_o consolidated RECOMPRESSION East Windsor samples at an OCR of four have K_o equal to unity. For simplicity, these samples were, therefore, isotropically consolidated in the laboratory prior to shear.

Data on overconsolidated samples with θ = 0, 45° and 90° are used to study the method of consolidation in $\overline{CIUC}$ tests.

6.2 EFFECT OF METHODS FOR MINIMIZING DISTURBANCE ON UNDRAINED STRENGTH BEHAVIOR

It is emphasized that the soil samples used in the study, especially those from block samples, are of much better quality than normally encountered in practice. Thus, results of this

study may not apply to the poorer quality "undisturbed" samples obtained from 3 in. diameter tube samples or obtained from 5 in. diameter tube samples with poor sampling techniques.

6.2.1 Behavior from Vertical Loading

The upper half of Table 6.2 summarizes test data from $\overline{CK_oUC}$ tests on SHANSEP and RECOMPRESSION samples. Figures 6.2 through 6.5 present plots of test data from the UUC test, and from the $\overline{CU}$ tests at OCR's of two and four. These data show the following results.

1. At OCR's of two and four, $\overline{CK_oUC}$ tests on SHANSEP and RECOMPRESSION samples at the same OCR yield essentially the same $q_f/\bar{\sigma}_{vc}$ within experimental accuracy. However, RECOMPRESSION samples have a higher NESE at q_f, and a higher pore pressure parameter A than the SHANSEP samples (Figs. 6.2 to 6.5). Therefore, the behavior in terms of effective stress is really quite different. Furthermore, the stress-strain behavior is also somewhat different (Figs. 6.2 and 6.3). RECOMPRESSION samples have a lower ε_f and higher $E_u/\bar{\sigma}_{vc}$ at high strain, though $E_u/\bar{\sigma}_{vc}$ at low strain of both SHANSEP and RECOMPRESSION samples are nearly identical. Based on a comparison of σ- ε -u behavior, it is seen that RECOMPRESSION samples exhibit a more sensitive and brittle behavior. The fact that RECOMPRESSION samples have larger normalized

pore pressures, and also a higher NESE at q_f explains why $q_f/\bar{\sigma}_{vc}$ from SHANSEP and RECOMPRESSION samples turned out to be practically identical.

2. $q_f/\bar{\sigma}_{vm}$ from the UUC (θ=0) test is also essentially the same as that from $\overline{CK_oUC}$ tests on SHANSEP and RECOMPRESSION samples at OCR of four, though the UUC test sample has the largest ε_f and the smallest $E_u/\bar{\sigma}_{vm}$. The agreement in q_f values indicates that disturbance due to structural damage and removal of in situ stress is relatively small, though the modulus is significantly reduced. (Note, however, that disturbance due to separation of varves, if there is any, shown from TC tests will be small since shearing would tend to close the separations or cracks). However, as will be shown in Section 6.2.2, the disturbance of this block sample is mainly due to separation of varves. This has a pronounced effect with horizontal and inclined samples, but little effect on vertical samples.

It is emphasized that the good agreement among UUC and the SHANSEP and RECOMPRESSION $\overline{CK_oUC}$ test $q_f/\bar{\sigma}_{vm}$ values is based on data obtained with very good quality block samples from East Windsor. With 5 in. diameter tube samples (such as those from Amherst and Northampton), where the effect of disturbance is greater, especially from samples at great depths, $q_f(V)/\bar{\sigma}_{vc}$ from UUC tests are generally much lower than those from SHANSEP samples, Fig. 6.6. These data also show that, because of

variable sample disturbance, UUC q_f values from tube samples are very scattered.

$q_f(V)/\bar{\sigma}_{vc}$ from a RECOMPRESSION $\overline{CK_oUC}$ test on 5 in. diameter Amherst tube sample is compared with the values from SHANSEP and RECOMPRESSION block samples in Fig. 6.7. These data show that $q_f(V)/\bar{\sigma}_{vc}$ of an Amherst sample with OCR of 1.6 is only slightly higher than that of the SHANSEP samples. It should be noted that $q_f(V)/\bar{\sigma}_{vc}$ from a $\overline{CIUC}$ (θ=0) test on a RECOMPRESSION sample (OCR=1.7) from East Windsor (Long, 1975) is approximately equal to that of the K_o consolidated RECOMPRESSION samples. These data indicate an insignificant effect on stress system during consolidation on $q_f(V)/\bar{\sigma}_{vc}$ obtained from the RECOMPRESSION method. (The same trend is also observed with SHANSEP samples; see Chapter 5.)

At an OCR of four, it is also interesting to compare data from $\overline{CIUC}$ tests on SHANSEP and RECOMPRESSION samples. (Note that at OCR=4, the "$\overline{CK_oU}$" RECOMPRESSION sample was isotropically consolidated.) $q_f/\bar{\sigma}_{vc}$ and NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ from these tests are roughly the same (see bottom part of Table 6.3), though A_f and NESE at q_f of the RECOMPRESSION sample are higher.

In conclusion, for vertical loading, the SHANSEP method of consolidation yields reasonable values of q_f and NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$, even though the magnitude of $\bar{\sigma}_{vm}$ is significantly larger (3.6 versus 2.4 kg/cm^2) than the in situ value. However, it yields a lower $E_u/\bar{\sigma}_{vc}$ and quite different behavior in terms of effective stress compared to the RECOMPRESSION method.

6.2.2 Behavior from Horizontal Loading

Test data in Figs. 6.4, 6.5 and 6.8 to 6.11 and the summary in Table 6.2 show the following results.

(1) At OCR's of two and four, the RECOMPRESSION $\overline{CK_oUE}$ test samples have a lower $q_f/\bar{\sigma}_{vc}$ and ε_f, and a higher A_f and $E_u/\bar{\sigma}_{vc}$ than SHANSEP samples at the same OCR (Figs. 6.8 and 6.9), though the NESE at q_f (also same envelope as at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$) of these samples are identical. Therefore, the decrease in $q_f/\bar{\sigma}_{vc}$ from SHANSEP to RECOMPRESSION results from larger normalized pore pressures in the RECOMPRESSION samples. The lower values of ε_f, and the higher values of $E_u/\bar{\sigma}_{vc}$, (Fig. 6.10), A_f and $\Delta u/\bar{\sigma}_{vc}$ from RECOMPRESSION samples show that RECOMPRESSION samples exhibit a more brittle and sensitive behavior and thus have a more sensitive soil structure. These results and results in Section 6.2.1, therefore, lead to the conclusion that the SHANSEP method of consolidation causes the soil to have a less sensitive and less brittle structure than the in situ soil since the high quality undisturbed RECOMPRESSION block sample should have a soil structure close to that of the in situ soil.

(2) At the in situ OCR (OCR=4), $q_f(H)/\bar{\sigma}_{vm}$ from the UUC (θ=90) test is about 80% of the RECOMPRESSION $\overline{CIUC}$ (θ=90) value and 62% of the SHANSEP $\overline{CK_oUE}$ value (Fig. 6.8). The lower UUC q_f and the large decrease in strength after failure is thought to be mainly caused by disturbance due to varve separation, even though the sample developed a failure plane across the varves. However, it is not known whether the separation occurred during sampling or mostly as the result of trimming. (Note

that the effect on vertical samples would be much smaller since shearing would tend to close the cracks.)

(3) Further evidence that disturbance (presumably due to varve separation) is most important with θ=90 sample is partly provided (since $\overline{CIUC}$ SHANSEP and RECOMPRESSION samples have different soil structure) by comparing $\overline{CIUC}$ (θ=90) test data on RECOMPRESSION and SHANSEP samples. The strength of the RECOMPRESSION sample is only about two-thirds of that from the SHANSEP sample at OCR's of 2 and 4 (Table 6.3). The moduli from RECOMPRESSION tests are also lower (Figs. 6.11 to 6.13). This is in contrast to the $\overline{CK_oUE}$ test where the RECOMPRESSION strengths were 21% less at OCR = 4 and 6% less at OCR = 2. (Note, however, that the $\overline{CK_oU}$ RECOMPRESSION samples had a much higher undrained modulus and larger pore pressure, i.e., they behave in more sensitive and brittle fashion). This suggests that the RECOMPRESSION method of consolidation only partially minimizes the effect of varve separation with horizontal samples.

The fact that the strength in a $\overline{CIUC}$ (θ=90) test is lower than that in a $\overline{CK_oUE}$ (θ=0) test, both performed on RECOMPRESSION samples at OCR = 4, despite the larger magnitude of σ_2 applied in the extension test also suggests that disturbance (presumably due to varve separation) is most important with θ=90 samples.

In summary, the two most important findings appear to be:

1. Disturbance due to varve separation is most important with horizontally trimmed samples and can be only partially overcome by the RECOMPRESSION method.
2. The SHANSEP method causes the soil to have less

sensitive and brittle behavior.

Finally, varve separation may have affected the RECOMPRESSION $\overline{CK_oUE}$ test at the in situ OCR of four, perhaps causing a lowering of the strength, since the difference in the RECOMPRESSION and SHANSEP strengths at the OCR is much greater than an OCR of 2.

6.2.3 Behavior from Inclined Loading

Test data from the $\overline{CIUC}$ (θ=45) and UUC (θ= 45) tests on overconsolidated samples (OCR=4) from East Windsor, presented in Fig. 6.14 and Table 6.3 (lower part), show the following results. $q_f/\bar{\sigma}_{vm}$ from the UUC (θ=45) test is only 64% of that from the SHANSEP $\overline{CIUC}$ (θ=45) test and 75% of that from the RECOMPRESSION $\overline{CIUC}$ (θ=45) test. The decrease in $q_f/\bar{\sigma}_{vc}$ from SHANSEP to RECOMPRESSION samples is due to larger normalized pore pressures while the NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ of these samples is probably identical (Fig. 6.15). The fact that UUC tests and the RECOMPRESSION $\overline{CIUC}$ (θ=45) test samples failed along a plane of weakness (located at the interface between the clay and silt portions), while the SHANSEP sample failed within the clay layer partly explains their lower $q_f/\bar{\sigma}_{vm}$. However, because of the use of a high isotropic consolidation stress, the SHANSEP strength is probably too high compared to the in situ soil.

Comparison of test data (Figs. 6.16 and 6.17) from SHANSEP $\overline{CK_oU}$DSS tests and from one RECOMPRESSION sample also indicates a trend similar to that from $\overline{CIUC}$ (θ=45) tests: $\tau_h/\bar{\sigma}_{vc}$ of the RECOMPRESSION ($\bar{\sigma}_{vc} = \bar{\sigma}_{vo}$) Amherst sample is lower than that of the SHANSEP sample (Fig. 6.16). The value of $G/\bar{\sigma}_{vc}$ at γ = 1%

from RECOMPRESSION test is also somewhat lower (Fig. 6.17).

6.3 DISCUSSION

Disturbance in Varved Clay

As stated in Section 6.1.1, disturbance in varved clay is due to the removal of the in situ stresses, soil structure damage, and varve separation. For block samples, unless σ_{1f} acts perpendicular to the varves, disturbance due to varve separation, which probably occurs mostly during trimming, is an important factor. As shown from UUC tests, it is most pronounced in inclined and horizontal samples, but perhaps small in vertical samples. Thus, disturbance in varved clay is directional dependent. The following experimental evidence illustrates the existence of disturbance due to varve separations in block samples.

(1) As shown in Fig. 6.18, for an OCR = 4, the increase in q_f from a UUC ($\theta = 45$) test to a RECOMPRESSION $\overline{CIUC}$ ($\theta = 45$) test is about the same as that from a UUC ($\theta = 90$) test to a RECOMPRESSION $\overline{CIUC}$ ($\theta = 90$) test, while $q_f(V)$ from a UUC ($\theta=0$) test is essentially identical to that from a RECOMPRESSION $\overline{CK_oUC}$ ($\theta=0$) test at OCR = 4.

(2) The q_f in a $\overline{CIUC}$ ($\theta = 90$) test is lower than that in a $\overline{CK_oUE}$ ($\theta=0$) test, both performed on RECOMPRESSION samples at OCR = 4, despite the larger magnitude of σ_2 applied in the extension test. (See Section 6.2.2.)

(3) The fact that a failure plane developed at the interface between varves in the UUC ($\theta=45$) test instead of within the

clay layer as observed from SHANSEP samples indicates the existence of a plane of weakness at this location (Section 6.2.3).

With 5 in. diameter tube samples, the effect of disturbance is greater (see test data from Amherst and Northampton, Section 6.2.1) than for block samples, probably because of increased soil structure damage. With tube samples, $q_f(V)$ from UUC ($\theta = 0$) tests are generally much lower than would be obtained from RECOMPRESSION $\overline{CK_oUC}$ tests. Thus, based on data from vertical loading, one expects that disturbance in tube samples will cause greater reductions in q_f in all directions, compared to block samples.

SHANSEP Versus RECOMPRESSION as a Method for Minimizing Sample Disturbance

The increase in q_f from UUC tests to RECOMPRESSION ($\bar{\sigma}_{vc}=\bar{\sigma}_{vo}$) $\overline{CIUC}$ tests on inclined and horizontal samples indicates that disturbance due to varve separations can be minimized by reconsolidating the samples. SHANSEP method of consolidation appears to be more effective than the RECOMPRESSION method in minimizing this type of disturbance, especially with inclined and horizontal samples, based on the following data from $\overline{CIUC}$ ($\theta=45$) tests on block samples. Despite reconsolidation to $\bar{\sigma}_{vo}$, the failure plane which developed in the RECOMPRESSION $\overline{CIUC}$ ($\theta=45$) test occurred at the interface between silt and clay portions, whereas it occurred within a clay layer for the SHANSEP sample. In addition, the difference in $q_f(H)/\bar{\sigma}_{vc}$ between the RECOMPRESSION and SHANSEP $\overline{CK_oUE}$ ($\theta=0$) tests increases with increasing OCR, i.e., 0.375/0.355 = 1.06 at OCR = 2, and 0.645/0.535 =1.21

at OCR = 4 (see Table 6.2). Since both SHANSEP samples had been consolidated beyond the in situ $\bar{\sigma}_{vm}$, this suggests, but does not prove, that SHANSEP may minimize disturbance due to varve separation better than the RECOMPRESSION method. However, it is not certain if the 20% difference in strength at OCR = 4 is mainly due to the influence of the varve separations.

Though the SHANSEP method of consolidation can effectively minimize disturbance due to separation of varves, it also has a drawback in that it causes significant changes in soil structure, especially when isotropic consolidation is used. The effect of these changes in the soil structure is discussed below.

With $\overline{CK_oU}$ tests, except for possible effects of varve separations (when σ_{1f} does not act perpendicular to the varves), it is thought that OC RECOMPRESSION data on block samples give the best indication of the in situ behavior of varved clay, since the structure in the RECOMPRESSION sample should be close to that of the in situ soil. For both vertical and horizontal loading, OC RECOMPRESSION samples compared to OC SHANSEP samples generally have significantly higher moduli, pore pressures and NESE at q_f (when it is not equal to that at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$, e.g., in the vertical loading). These general trends show that OC RECOMPRESSION samples act in a more brittle and sensitive manner. This in turn strongly suggests that the SHANSEP method of consolidation causes the test samples to have a less brittle and sensitive behavior than that of the in situ soil. Thus, though

data (mostly from NC soil) over a small MPPR range on Connecticut Valley varved clays show normalized behavior with respect to q_f, A_f, $\bar{\phi}$ and $\bar{c}/\bar{\sigma}_{vm}$ when consolidated above $\bar{\sigma}_{vm}$, based on OC East Windsor block sample test data one concludes that stress-strain-pore pressure parameters from OC ($2 \leq OCR \leq 4$) SHANSEP samples do not represent the true in situ soil behavior. However, SHANSEP samples apparently yield reasonable estimates of q_f and NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ for both vertical and horizontal loading.

With $\overline{CIU}$ tests, SHANSEP yields much larger q_f values than the RECOMPRESSION method, except for vertical loading on $\theta = 0$ samples, as shown in Fig. 6.18. This may result both from the influence of disturbance due to varve separations which is important with inclined and horizontal samples, and from the fact that high isotropic consolidation may produce a less parallel orientation of clay particles perpendicular to the in situ vertical direction.

6.4 SUMMARY AND CONCLUSIONS

Block sample test data presented in this chapter show that the UUC test is not suitable for measuring anisotropic strength and modulus. Despite using block samples, disturbance due to separation of varves causes the value of q_f from inclined and horizontal samples to be too low compared to more reliable data obtained from $\overline{CIUC}$ RECOMPRESSION tests. For typical tube samples, increased soil structure damage will probably cause a further reduction in q_f and undrained modulus in <u>all</u> directions.

OC $\overline{CK_oUC}$ block sample test data show that the SHANSEP method of consolidation causes a change in soil structure to produce samples that are less brittle and sensitive than the in situ clay. On the other hand, the RECOMPRESSION method better preserves the in situ structure. However, SHANSEP may yield more reasonable estimates of q_f in cases where disturbance due to separation of varves affects the results.

With $\overline{CK_oU}$ tube samples, which are likely to be much more disturbed than block samples, little data exist for comparison of the SHANSEP and RECOMPRESSION methods of consolidation. But in the view of the author, SHANSEP method is probably the better approach if the in situ soil has a low OCR, since RECOMPRESSION yields the maximum discrepancy between the in situ and laboratory compression curves in the vicinity of the maximum past pressure (e.g., see Fig. 6.1(a)), while use of the RECOMPRESSION method in tube sample is probably better if the clay is heavily overconsolidated and there is little disturbance due to separation of varves. In any case, SHANSEP has the advantage of minimizing adverse effects of varve separation whereas RECOMPRESSION has the advantage of better preserving the in situ structure of OC samples.

For both block and tube samples, $\overline{CIUC}$ tests on samples with different inclinations consolidated by either the SHANSEP or RECOMPRESSION methods are not recommended for obtaining anisotropic undrained parameters.

In general, the pros and cons of the SHANSEP and RECOMPRESSION methods of testing for use with good quality "undisturbed"

Connecticut varved clay samples are as follows:

(A) RECOMPRESSION Method

Advantages

(1) OC RECOMPRESSION samples (OCR $\geq$ 2) probably yield reasonable estimates of in situ $S_u/\bar{\sigma}_{vc}$, though we can not really be sure whether it is too high or too low and the error probably depends upon the sample quality and direction of loading.

(2) RECOMPRESSION samples probably better represent the in situ pre-failure σ-ε-u behavior of OC samples, especially for modes of failure involving rotation of principal stresses.

Disadvantages

(1) If soil parameters are not related to stress history, i.e., OCR, each new job will need $\overline{CK_oU}$ testing to obtain data versus depth.

(2) For NC and perhaps slightly OC deposits (OCR $\leq$ 2) the RECOMPRESSION method of consolidation ($\bar{\sigma}_{vc} = \bar{\sigma}_{vo}$) could overestimate the in situ q_f.

(B) SHANSEP Method

Advantages:

(1) For normally consolidated and slightly OC deposits, where good quality undisturbed samples are especially difficult to obtain, the SHANSEP method probably yields better estimates of the in situ q_f and σ-ε-u behavior.

(2) NSP from many types of SHANSEP strength tests have been shown to be the same for several locations within the Connecticut Valley (see Chapter 4).

(3) Thus, since soil parameters have been developed

as a function of OCR, the testing program can concentrate on consolidation tests for determining the stress history, plots of NSP versus OCR already having been established.

(4) NSP versus OCR data provide information to predict changes in in situ soil parameters due to changes in consolidation stress, such as occur during consolidation under embankments.

Disadvantages

(1) Because of changes in soil structure, the SHANSEP method of consolidation may not provide very reliable σ-ε-u data for OC samples.

(2) Because NSP are expressed as a function of OCR, the SHANSEP method is not suitable to use at a site where the stress history is very scattered. In such cases, tests such as the field vane are recommended.

In summary, the RECOMPRESSION method is recommended if the deposit is heavily overconsolidated (OCR> 4) and good quality samples can be obtained, though it would still be desirable to correlate the results with values of in situ OCR and with those from SHANSEP method. In such cases, when predictions of deformations and pore pressures are especially important, the RECOMPRESSION procedure will probably yield much better parameters. For cases where stability is of prime concern, and if the stress history can be established with reasonable certainty, then the SHANSEP method has a distinct advantage since extensive $S_u/\bar{\sigma}_{vo}$ versus OCR data exist for the Connecticut Valley varved

clay. Finally, for normally consolidated and slightly OC deposits, use of the SHANSEP method is recommended.

EXPERIMENTAL PROGRAM FOR INVESTIGATING THE EFFECT OF METHODS FOR MINIMIZING DISTURBANCE ON UNDRAINED SHEAR STRENGTH BEHAVIOR

Stress in kg/cm^2

Type of Undrained Shear	Type of Test	Soil Location (1)	θ°	MOC	$\bar{\sigma}_{vc}$	$\bar{\sigma}_{vm}$	OCR	$\dot{\varepsilon}$ %/hr
Vertical Undrained Shear	$\overline{CK_oUC}$	E	0	S	0.9, 1.8,	3.6	2,4	0.7
	$\overline{CK_oUC}$ (3)	E	0	S	0.6, 1.2,	2.4	2,4	0.35
	$\overline{CK_oUC}$	A	0	R	1.0,	1.6	1.6	1.0
	$\overline{CIUC}$	E	0	S	1.0,	4.0	4	0.7
	UU(V)	E	0	--	--	2.4	4	5.5
Horizontal Undrained Shear	$\overline{CK_oUE}$	E,A	0	S	0.9, 1.8,	3.6	2,4	0.7
	$\overline{CK_oUE}$ (3)	E	0	R	0.6, 1.2,	2.4	2,4	0.35
	$\overline{CIUC}$	E	90	S	1.0, 2.0,	4.0	2,4	0.7
				R	0.6, 1.2,	2.4	2,4	0.35
	UU(H)	E	90	--	--	2.4	4	5.5
Undrained Shear Within Clay Layer	$\overline{CIUC}$	E	45	S	1.0	4.0	4	0.7
	$\overline{CIUC}$	E	45	R	0.6	2.4	4	0.35
	$\overline{CK_oUDSS}$	A	0	R	1.1	1.7	1.5	5.0
	UU(45)	E	45	--	--	2.4	4	5.5

Notes: (1) E = East Windsor; A = Amherst.
(2) S = SHANSEP; R = RECOMPRESSION; MOC = Method of Consolidation
(3) $\overline{CIU}$ tests when K_o = 1.0 (i.e., at OCR = 4).

Table 6.1

COMPARISON OF TEST RESULTS FROM $\overline{CK_oU}$ TESTS ON SHANSEP AND RECOMPRESSION SAMPLES FROM CONNECTICUT VALLEY

Test No.	Type of Test	OCR	(2) MOC	Type of Sample (1)	Data at q_f					Data at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$					$E_u/\bar{\sigma}_{vc}$ at 1/2 $\frac{\Delta q}{\Delta q_f}$
					$\frac{q_f}{\bar{\sigma}_{vc}}$	A_f	ε_f %	$\frac{\bar{c}}{\bar{\sigma}_{vm}}$	$\bar{\phi}°$	$\frac{q}{\bar{\sigma}_{vc}}$	A_f	ε_f %	$\frac{\bar{c}}{\bar{\sigma}_{vm}}$	$\bar{\phi}°$	
AKC-2	$\overline{CK_oUC}$	2	S	5"ϕ	0.396	0.541	2.75	.088	14.0	0.364	0.936	12.61	0.017	30.0	205
ERKC-1		2	R	block	0.430	0.555	1.6	.017	30.0	0.430	0.555	1.60	0.017	30.0	265
EKC-5	$\overline{CK_oUC}$	4	S	block	0.680	0.215	6.2	.088	14.0	0.680	0.215	6.20	0.017	30.0	154
ERIC-1-1		4	R	block	0.640	0.500	2.2	.017	30.0	0.57	0.62	1.10	0.017	30.0	170
AKE-1	$\overline{CK_oUE}$	2	S	5"ϕ	0.375	0.695	12.5	.017	30.0	0.375	0.695	12.50	0.017	30.0	61
ERKE-1		2	R	block	0.355	0.935	3.2	.017	30.0	0.355	0.935	3.20	0.017	30.0	281
EKE-3	$\overline{CK_oUE}$	4	S	block	0.645	0.400	11.5	.017	30.0	0.645	0.400	11.50	0.017	30.0	59
ERIE-1-1		4	R	block	0.535	0.665	4.8	.017	30.0	0.535	0.665	4.80	0.017	30.0	223

Notes: (1) Block sample came from East Windsor
5"ϕ sample came from Amherst

(2) S = SHANSEP
R = RECOMPRESSION

Table 6.2

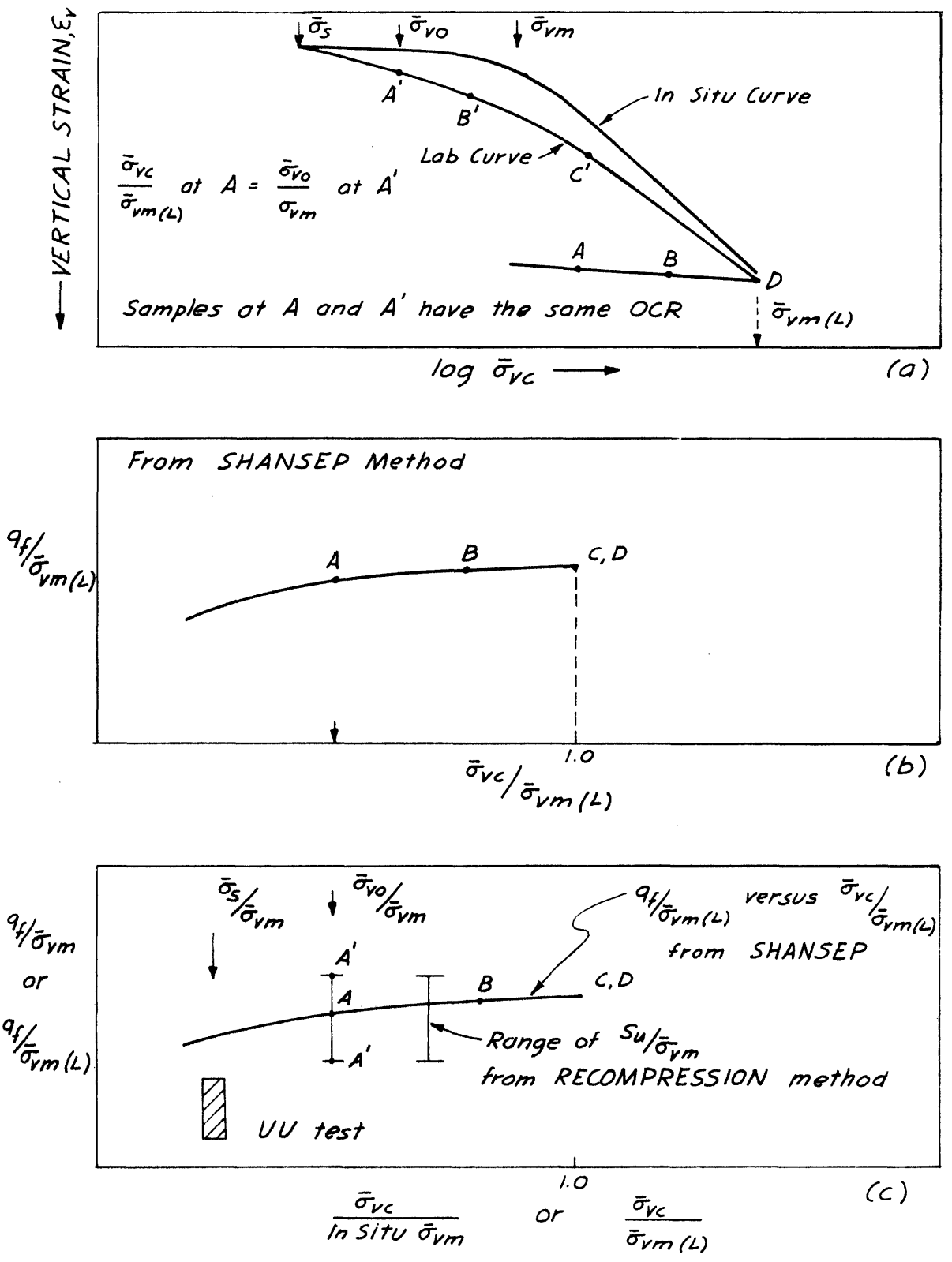
COMPARISON OF TEST RESULTS FROM $\overline{CIUC}$ TESTS ON SHANSEP AND RECOMPRESSION SAMPLES

All Samples from East Windsor

Test No.	Type of Test	OCR	θ°	(1) MOC	Data at q_f					Data at $(\sigma_1/\sigma_3)_{max}$					$E_u/\bar{\sigma}_{vc}$ at $\varepsilon = 1/2$ %
					$q_f/\bar{\sigma}_{vc}$	A_f	ε_f %	$\bar{c}/\bar{\sigma}_{vm}$	$\bar{\phi}^\circ$	$\frac{q}{\bar{\sigma}_{vc}}$	A_f	ε_f %	$\frac{\bar{c}}{\bar{\sigma}_{vm}}$	$\bar{\phi}^\circ$	
EIC-3-2	$\overline{CIUC}$	2	90	S	0.510	0.400	6.5	.017	30	.510	.400	6.5	.017	30	116
ERIC-3-1		2	90	R	0.345	0.870	2.9	.017	30	.340	1.03	8.8	.017	30	76
EIC-3-3	$\overline{CIUC}$	4	90	S	0.785	0.130	7.4	.017	30	.780	.135	9.0	.017	30	400
ERIC-3-2		4	90	R	0.480	0.520	8.8	.017	30	.450	.520	8.8	.017	30	304
EIC-2-3	$\overline{CIUC}$	4	45	S	0.575	0.200	4.8	.027	22	.550	.270	8.7	.027	22	280
ERIC-2-1		4	45	R	0.475	0.540	3.9	.027	22	.475	.540	3.9	.027	22	280
EIC-1-3	$\overline{CIUC}$	4	0	S	0.680	0.242	11.3	.017	30	.680	.254	13.6	.017	30	110
ERIC-1-1			0	R	0.640	0.500	2.2	.017	30	.570	.610	1.1	.017	30	154

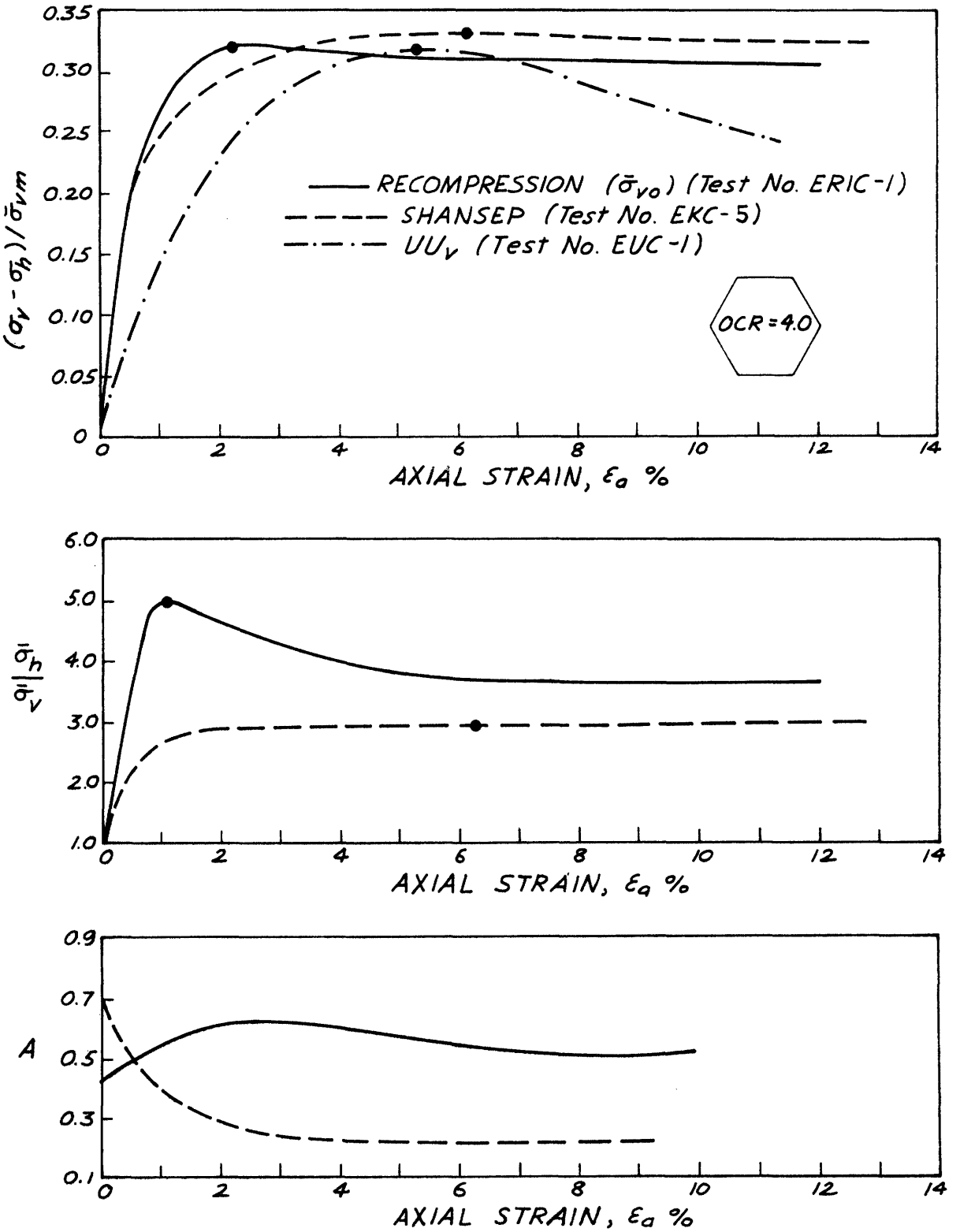
Note: (1) S = SHANSEP
R = RECOMPRESSION

Table 6.3



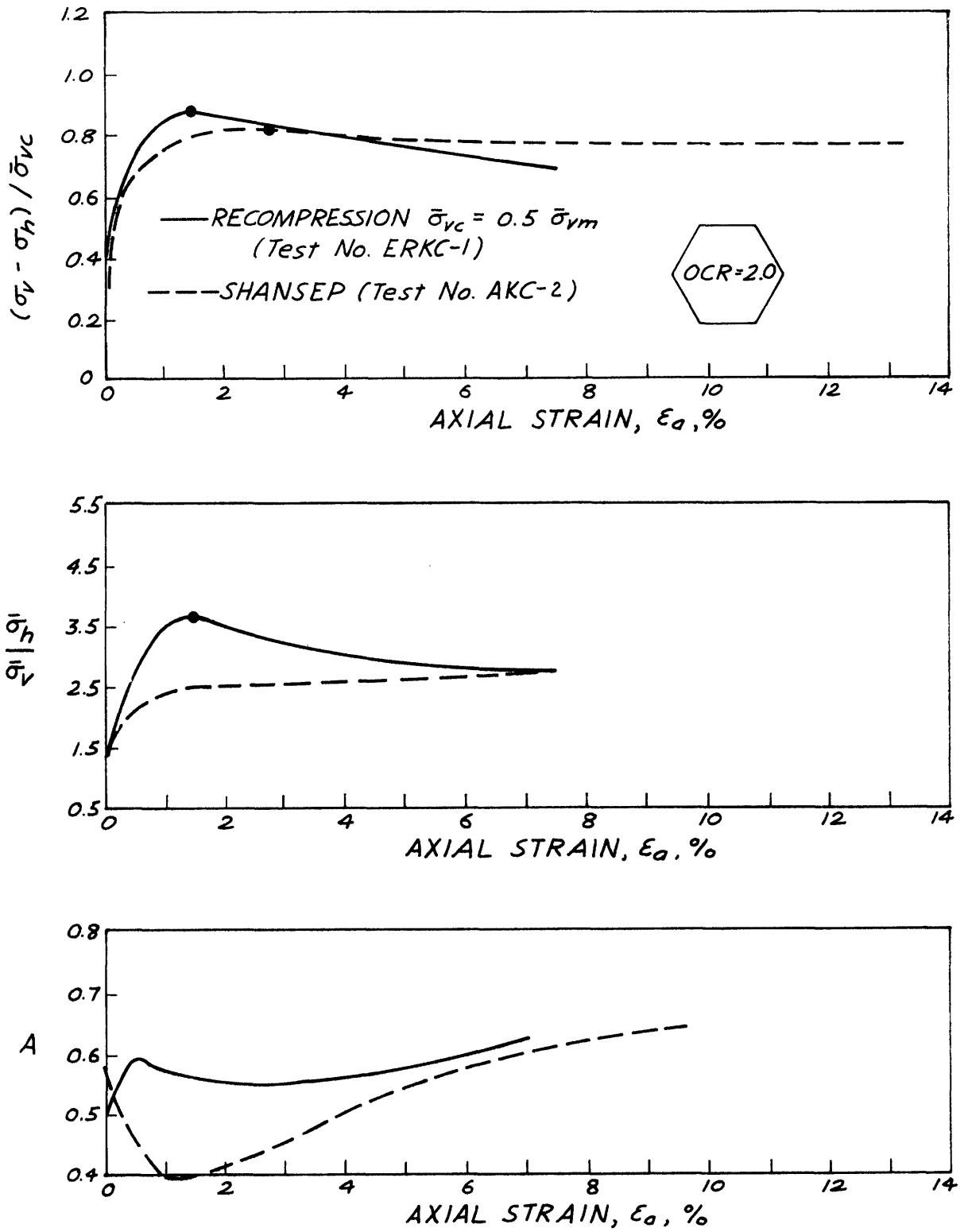
GENERAL EFFECTS OF DISTURBANCE ON UUC TEST, RECOMPRESSION, AND SHANSEP SAMPLES

FIGURE 6.1



EFFECT OF METHOD OF CONSOLIDATION ON VERTICAL UNDRAINED σ-ε-STRENGTH BEHAVIOR FROM TRIAXIAL COMPRESSION TESTS ON EAST WINDSOR VARVED CLAY OCR = 4

FIGURE 6.2



EFFECT OF METHOD OF CONSOLIDATION ON VERTICAL UNDRAINED σ-ϵ-STRENGTH BEHAVIOR FROM TRIAXIAL COMPRESSION TESTS ON EAST WINDSOR VARVED CLAY AT OCR=2 FIGURE 6.3

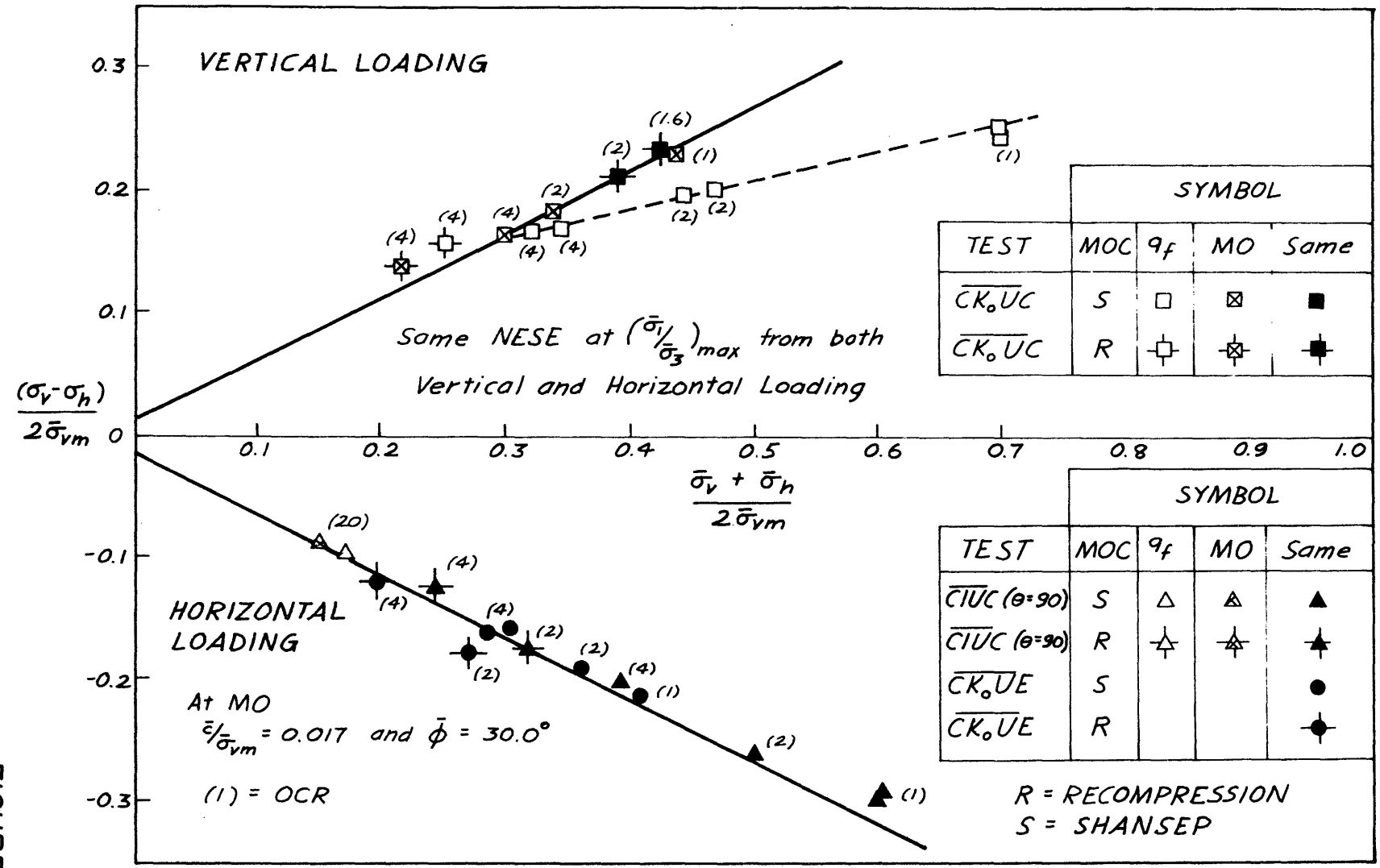


FIGURE 6.4 NORMALIZED EFFECTIVE STRESS ENVELOPES FROM $\overline{CU}$ TESTS ON SHANSEP AND RECOMPRESSION VARVED CLAY SAMPLES FROM CONNECTICUT VALLEY (DATA FROM BOTH AMHERST AND EAST WINDSOR FOR SHANSEP SAMPLES)

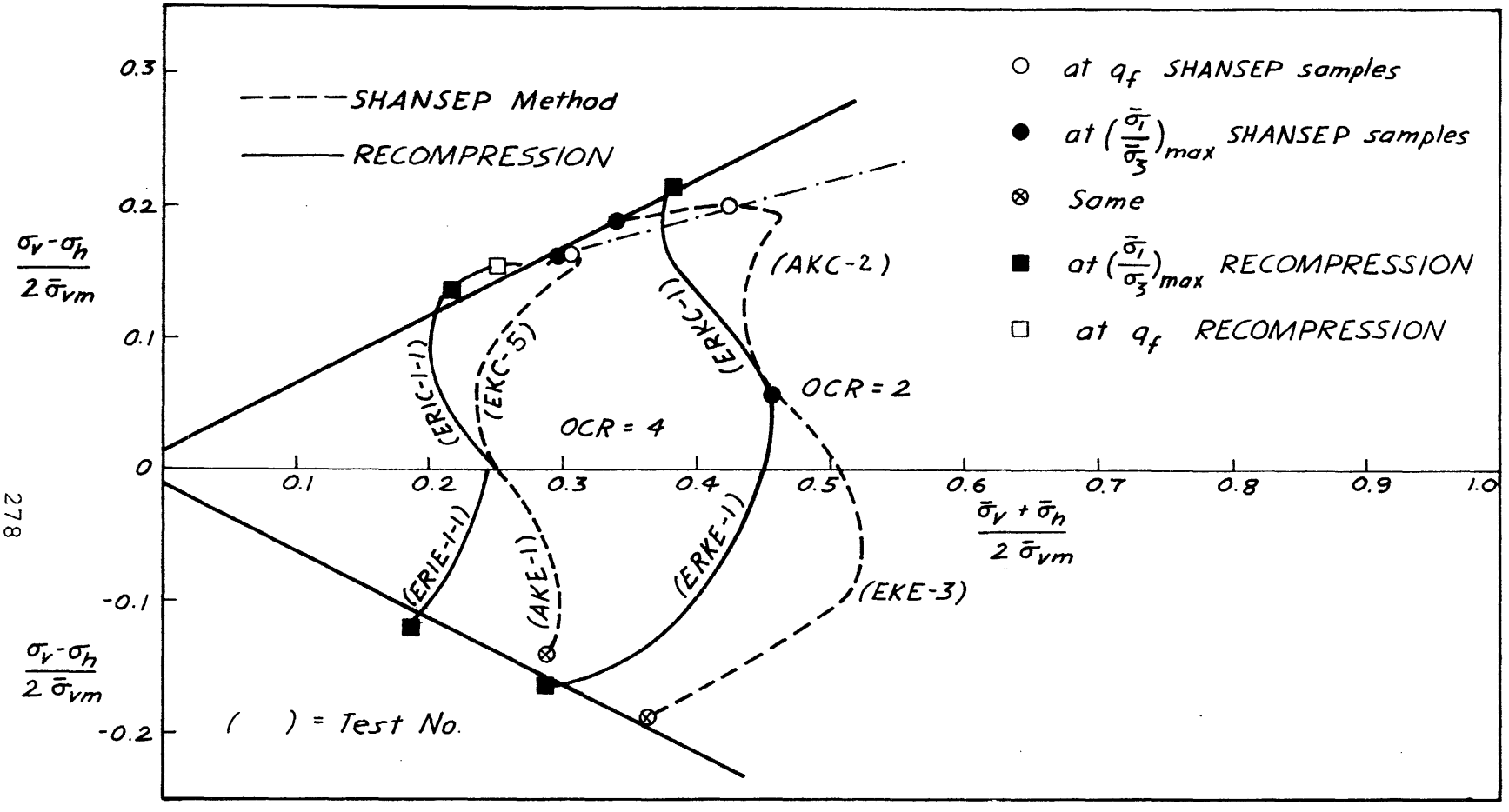
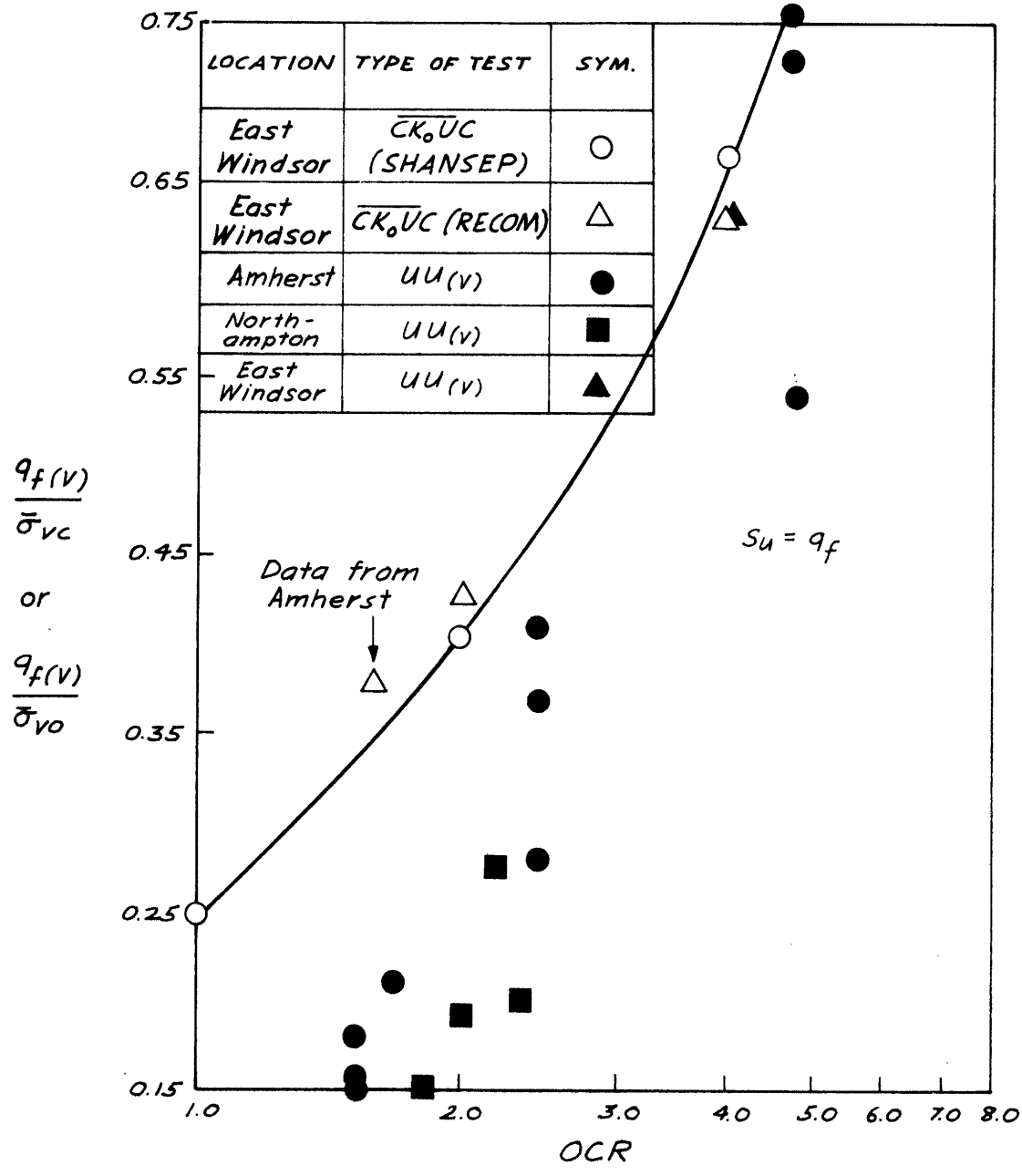


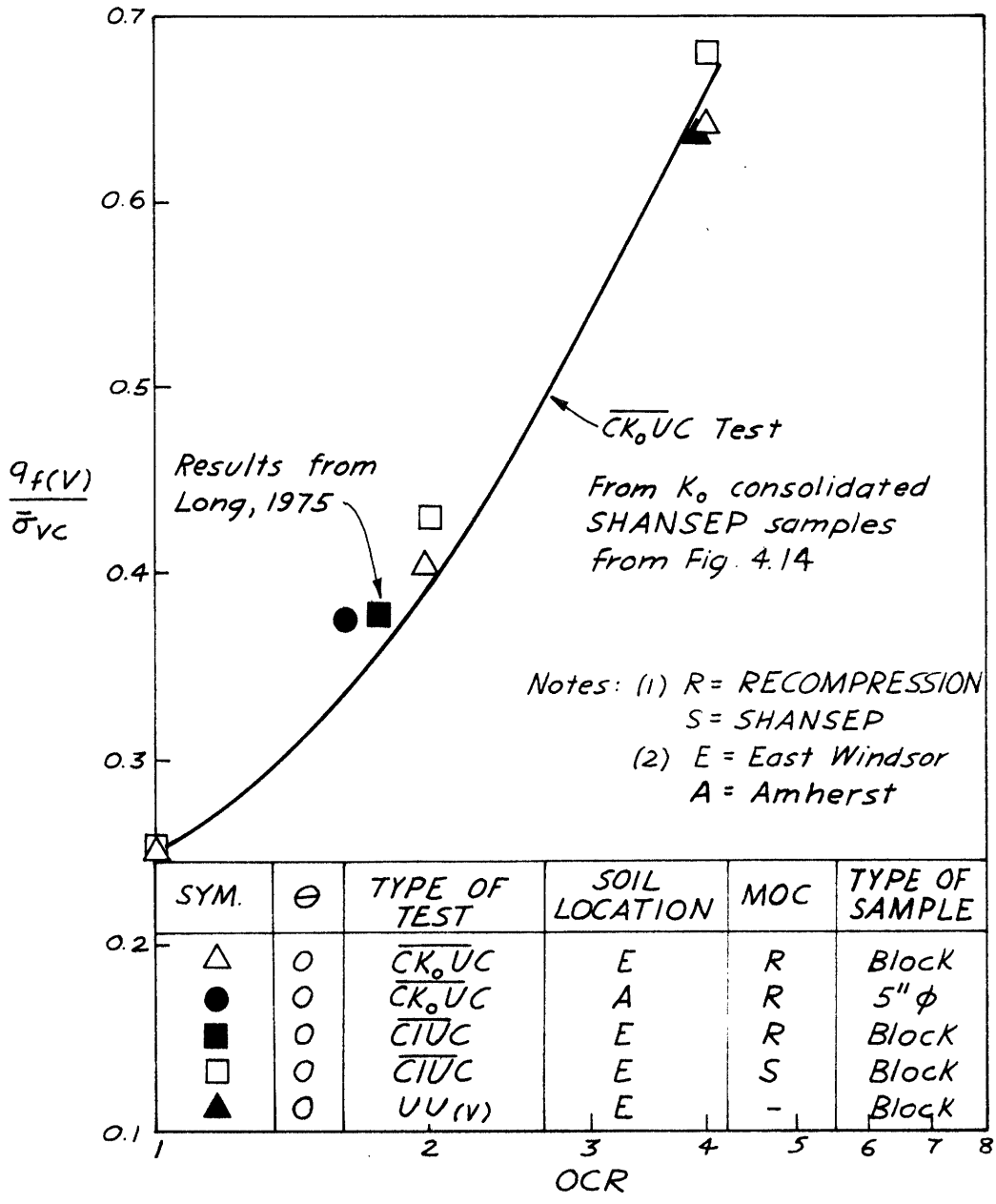
FIGURE 6.5

NORMALIZED STRESS PATHS FROM $\overline{CK_oU}$ TESTS ON SHANSEP AND RECOMPRESSION SAMPLES FROM CONNECTICUT VALLEY



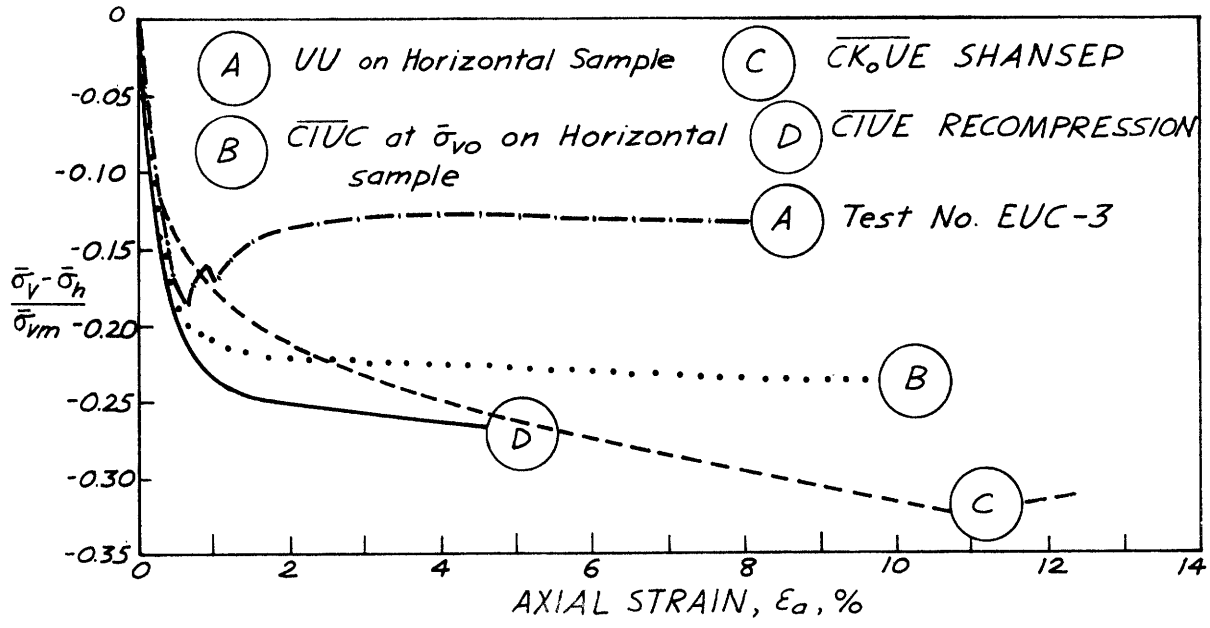
COMPARISON OF $\frac{q_f(V)}{\bar{\sigma}_{vc}}$ FROM $\overline{CK_oUC}$ TEST SHANSEP SAMPLES AND UU TEST SAMPLES FROM AMHERST AND NORTHAMPTON

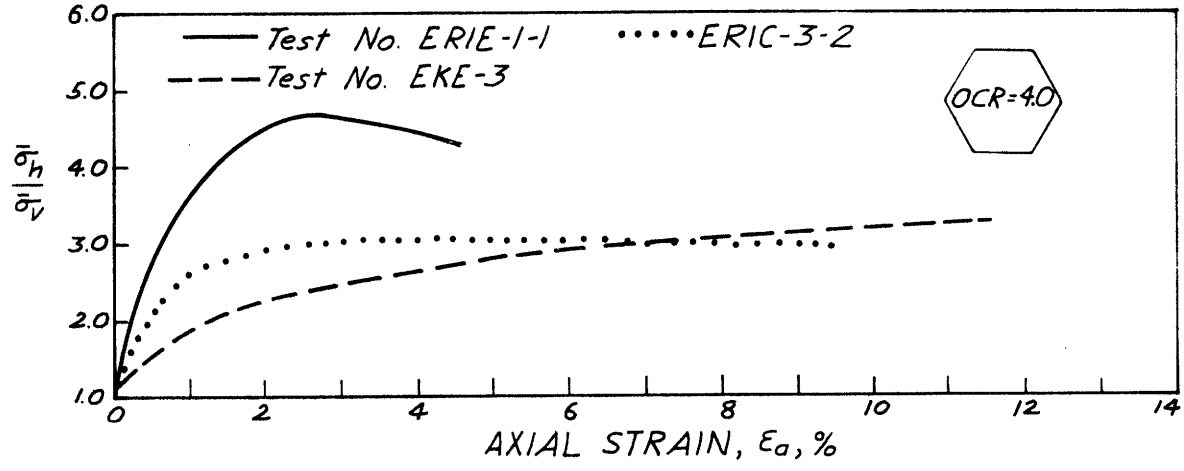
FIGURE 6.6

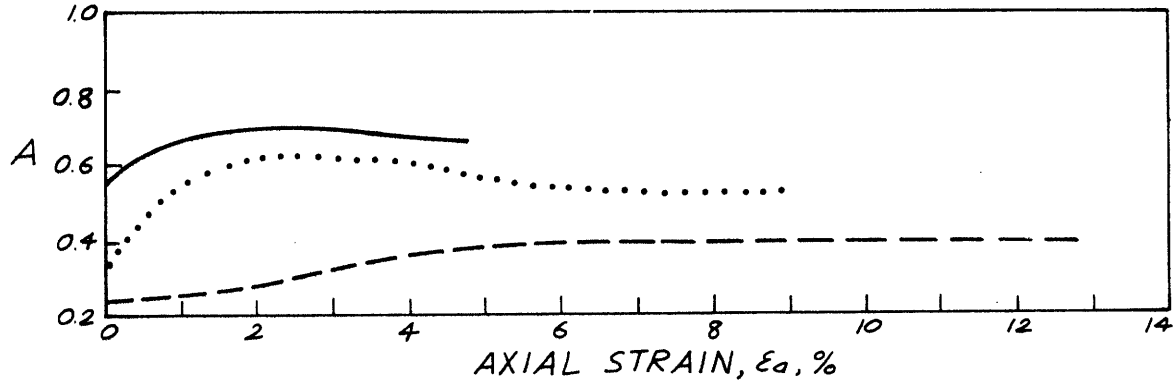


COMPARISON OF $\frac{q_f(V)}{\bar{\sigma}_{VC}}$ VERSUS OCR FROM $\overline{CK_oUC}$, $\overline{CIUC}$, AND UU(v) TESTS ON SHANSEP AND RECOMPRESSION SAMPLES OF AMHERST AND EAST WINDSOR VARVED CLAYS

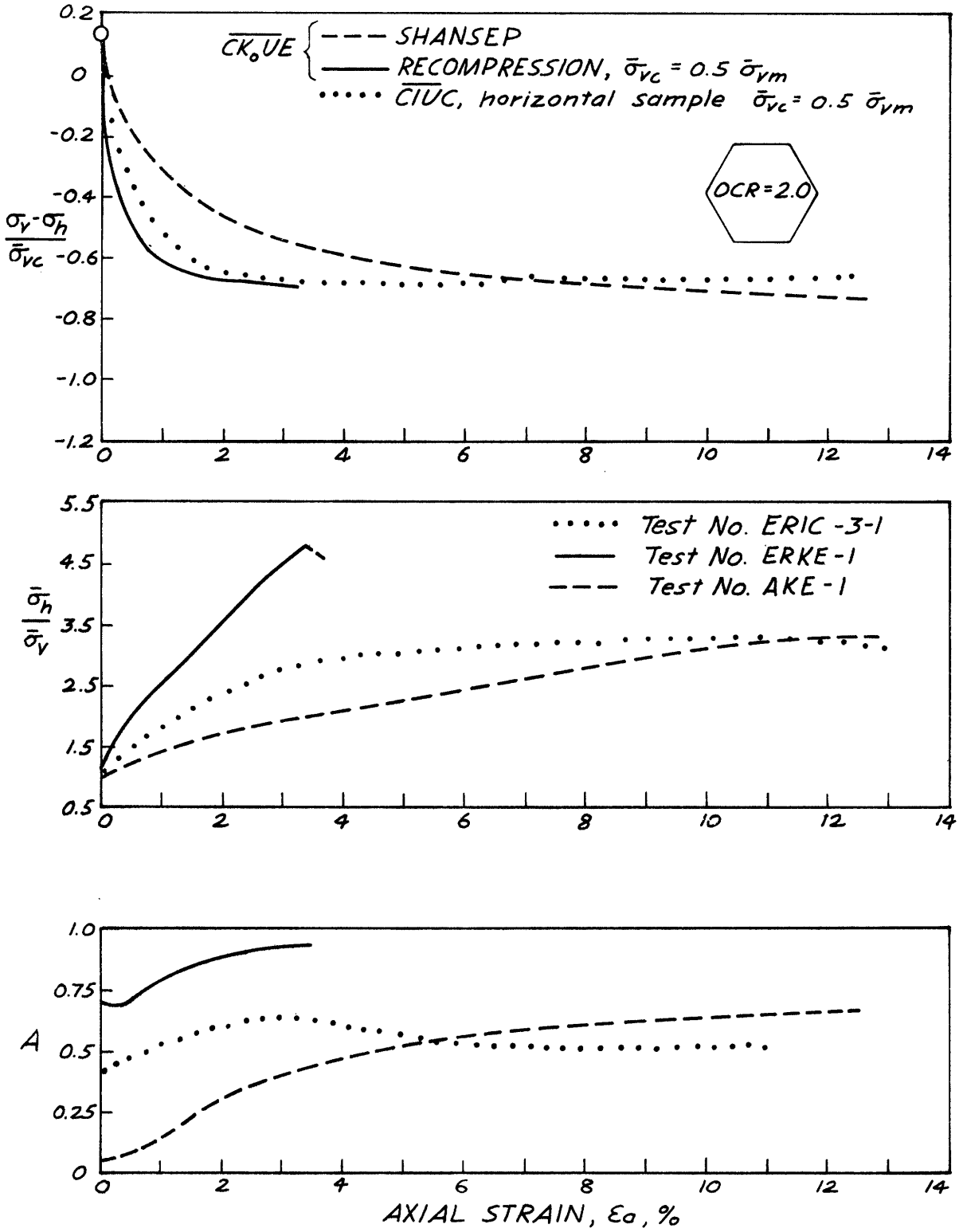
FIGURE 6.7





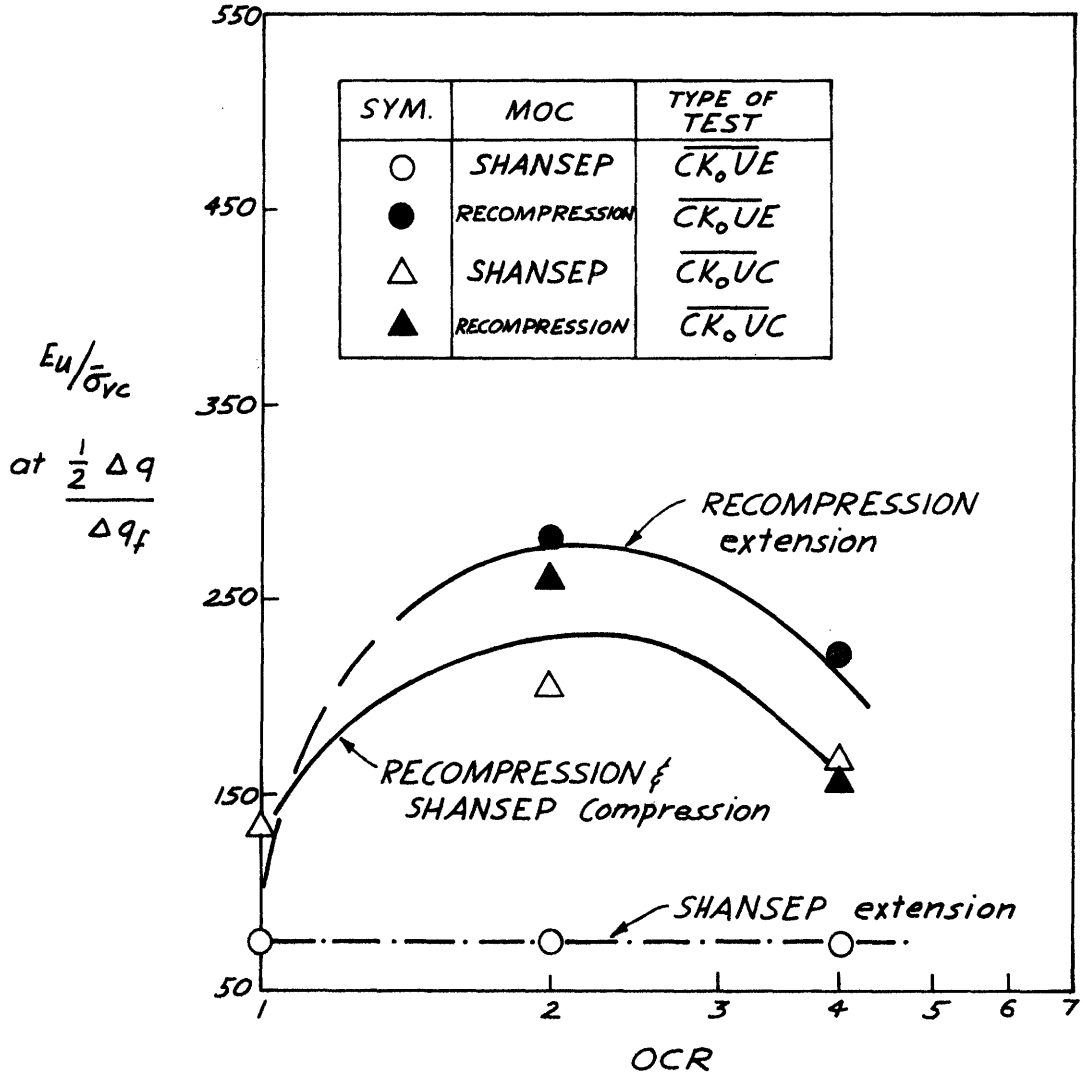


EFFECT OF METHOD OF CONSOLIDATION ON HORIZONTAL UNDRAINED σ-ϵ-STRENGTH BEHAVIOR FROM TRIAXIAL TESTS OF EAST WINDSOR VARVED CLAY AT OCR = 4 FIGURE 6.8



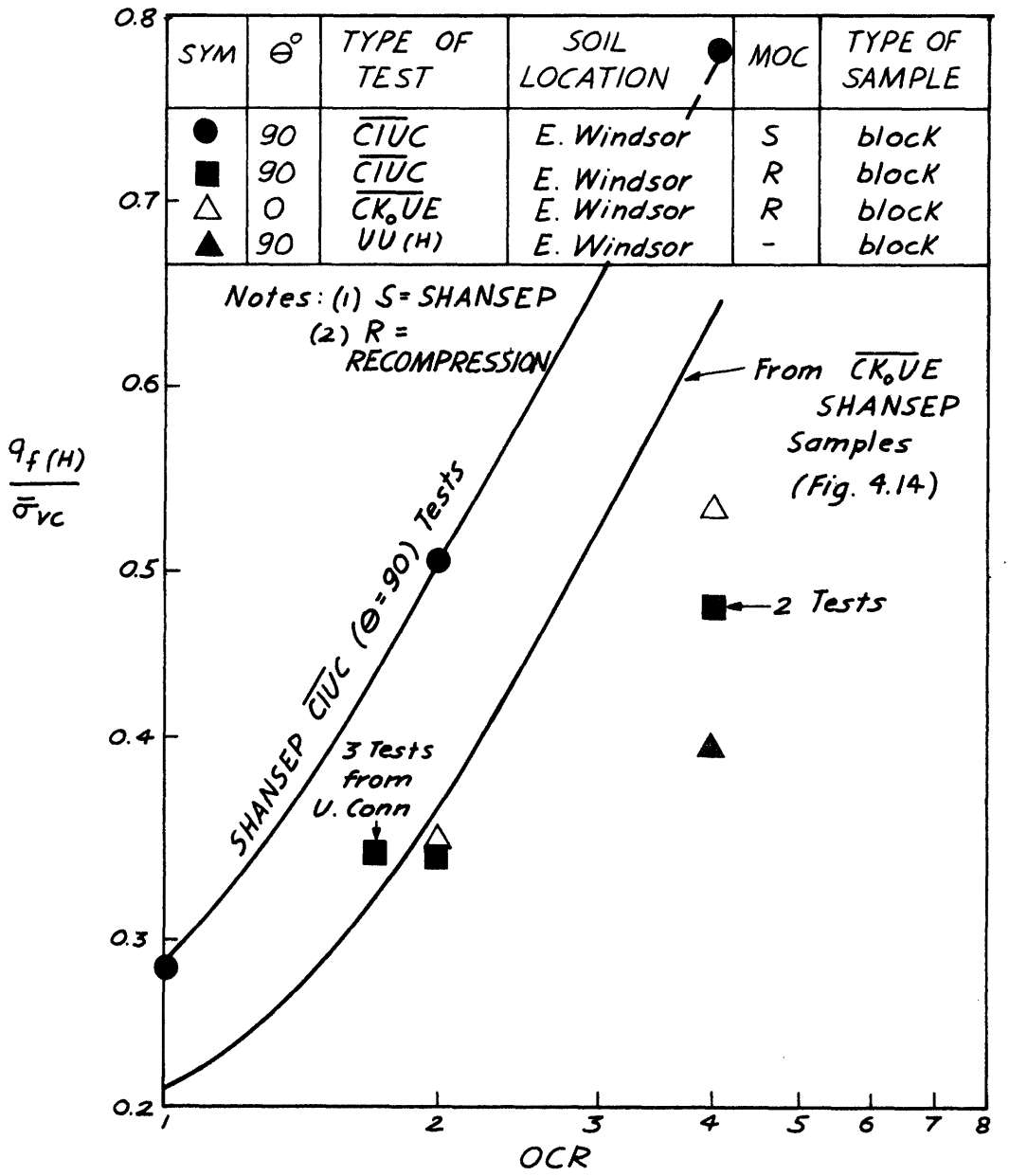
EFFECT OF METHOD OF CONSOLIDATION ON HORIZONTAL UNDRAINED σ-ϵ-STRENGTH BEHAVIOR FROM TRIAXIAL TESTS OF CONNECTICUT VARVED CLAYS AT OCR = 2.0

FIGURE 6.9



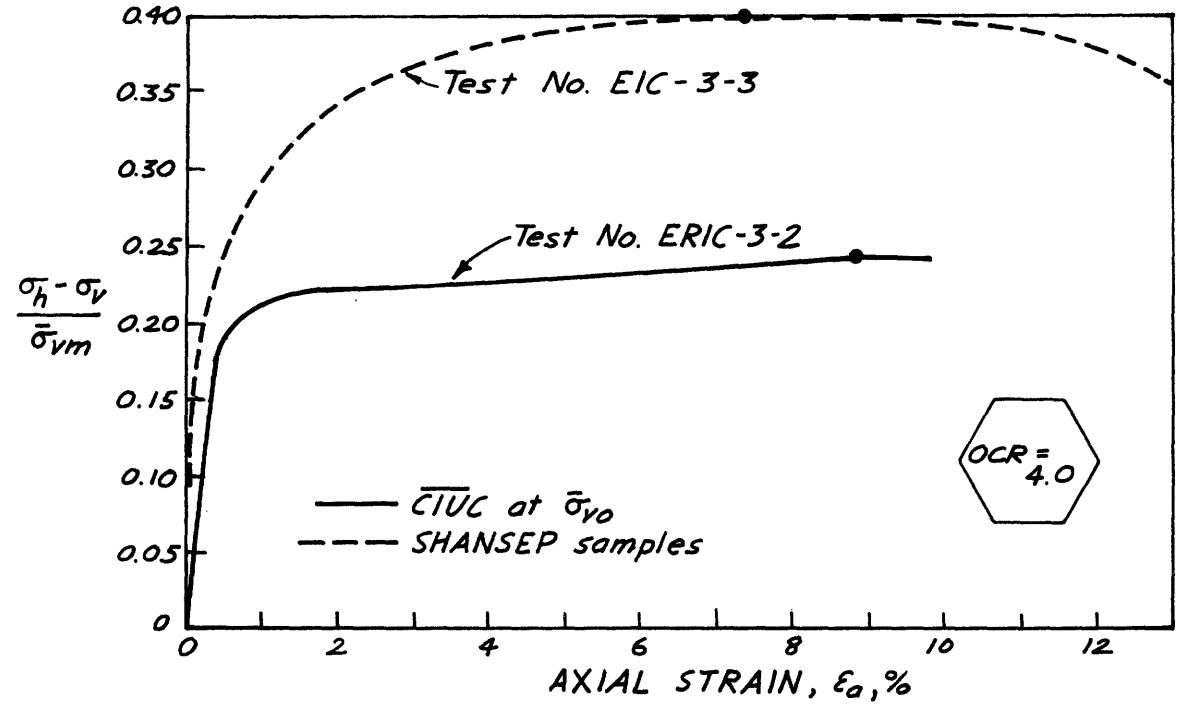
$\frac{E_u}{\bar{\sigma}_{vc}}$ AT $\frac{1}{2}\frac{\Delta q}{\Delta q_f}$ FROM $\overline{CK_oUC}$ AND $\overline{CK_oUE}$ TESTS ON SHANSEP AND RECOMPRESSION SAMPLES FROM CONNECTICUT VALLEY

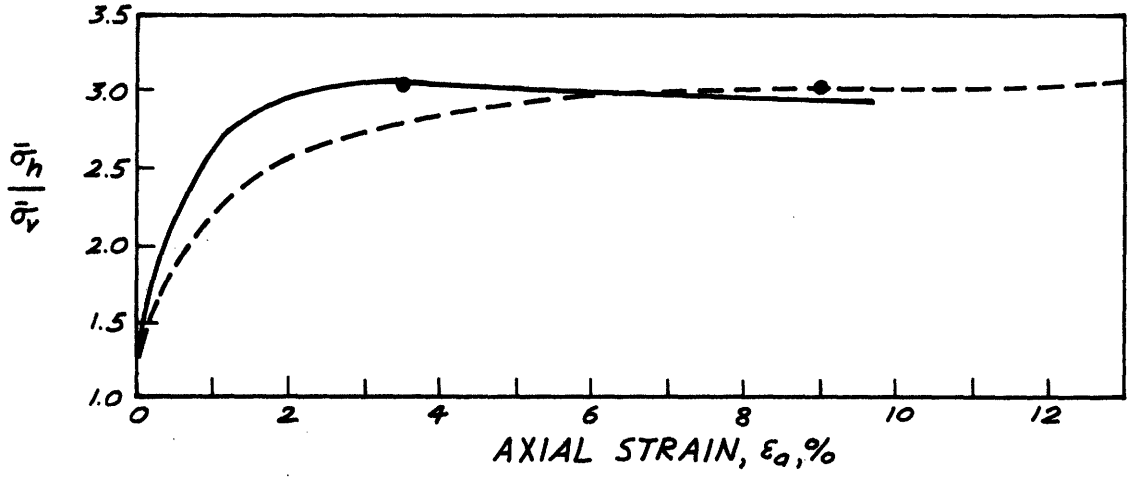
FIGURE 6.10

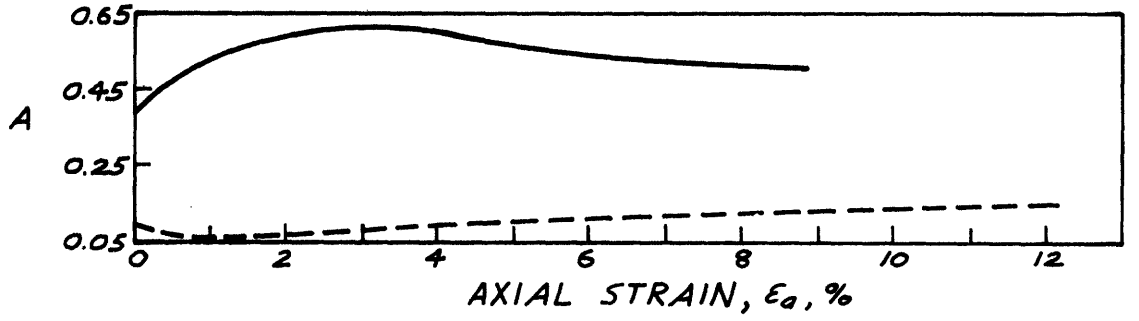


COMPARISON OF $\frac{q\ (H)}{\bar{\sigma}_{vc}}$ VERSUS OCR FROM $\overline{CK_oUE}$, $\overline{CIUC}$ ($\theta = 90$) AND UU ($\theta = 90$) TESTS ON SHANSEP AND RECOMPRESSION SAMPLES OF AMHERST AND EAST WINDSOR VARVED CLAYS

FIGURE 6.11

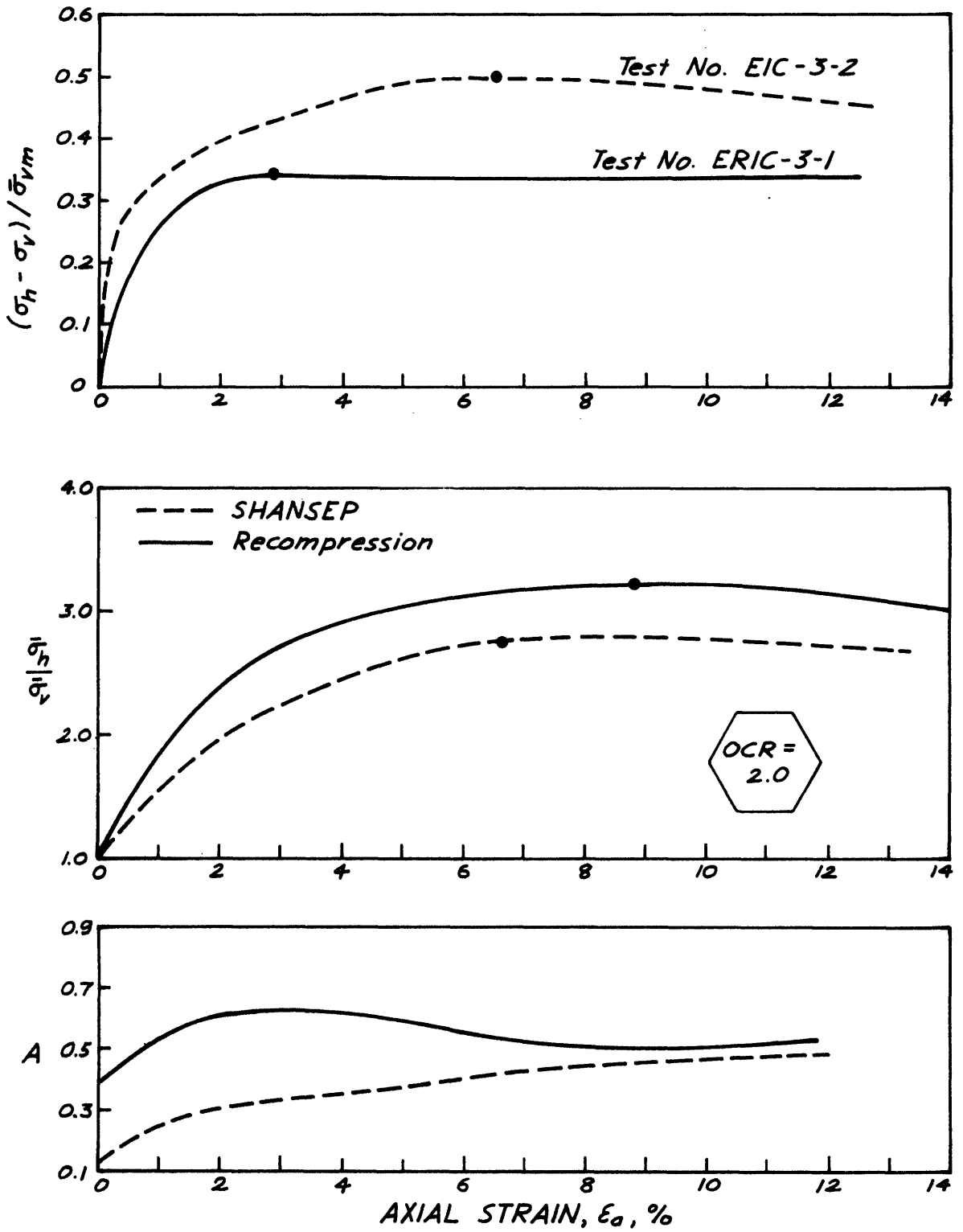






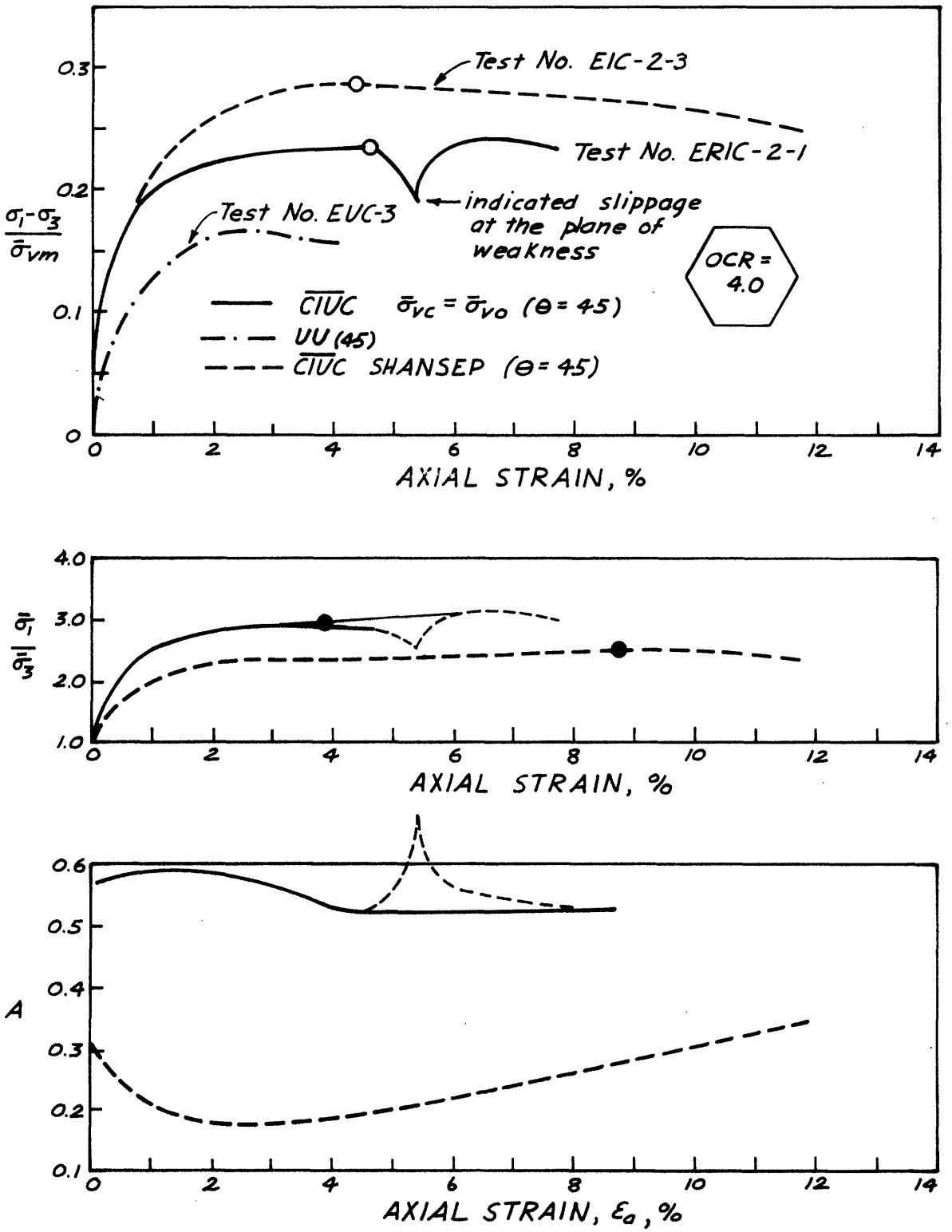
NORMALIZED STRESS-STRAIN CURVES FROM $\overline{CIUC}$ ($\theta = 90$) TESTS ON SHANSEP AND RECOMPRESSION SAMPLES((OCR=4) FROM EAST WINDSOR

FIGURE 6.12



NORMALIZED STRESS-STRAIN CURVES FROM $\overline{CIUC}$ ($\theta = 90$) TESTS ON SHANSEP AND RECOMPRESSION SAMPLES (OCR = 2) FROM EAST WINDSOR

FIGURE 6.13



NORMALIZED STRESS-STRAIN CURVES FROM $\overline{CIUC}$ (θ = 45) TEST ON SHANSEP AND RECOMPRESSION SAMPLES (OCR = 4) FROM EAST WINDSOR

FIGURE 6.14

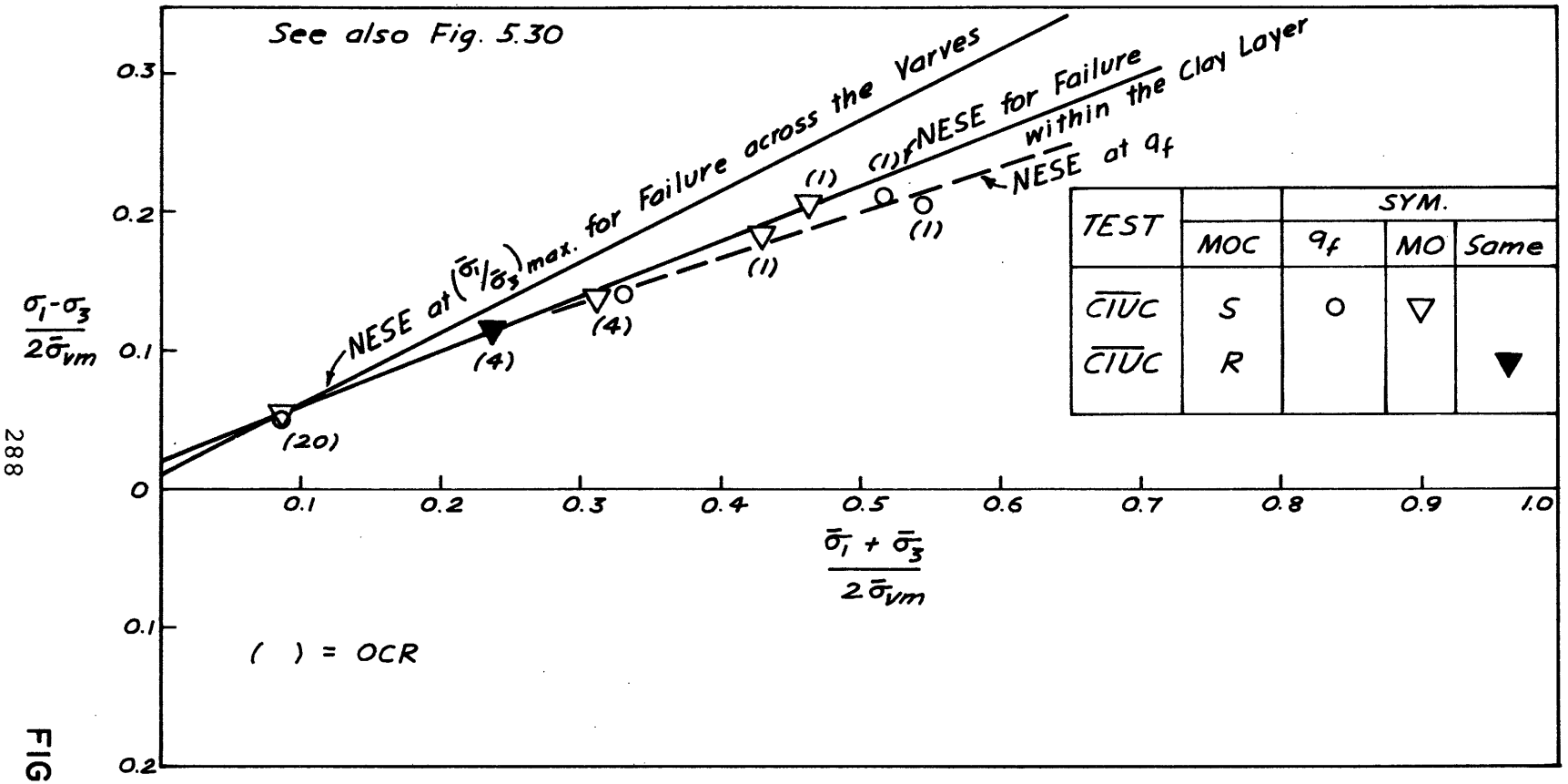
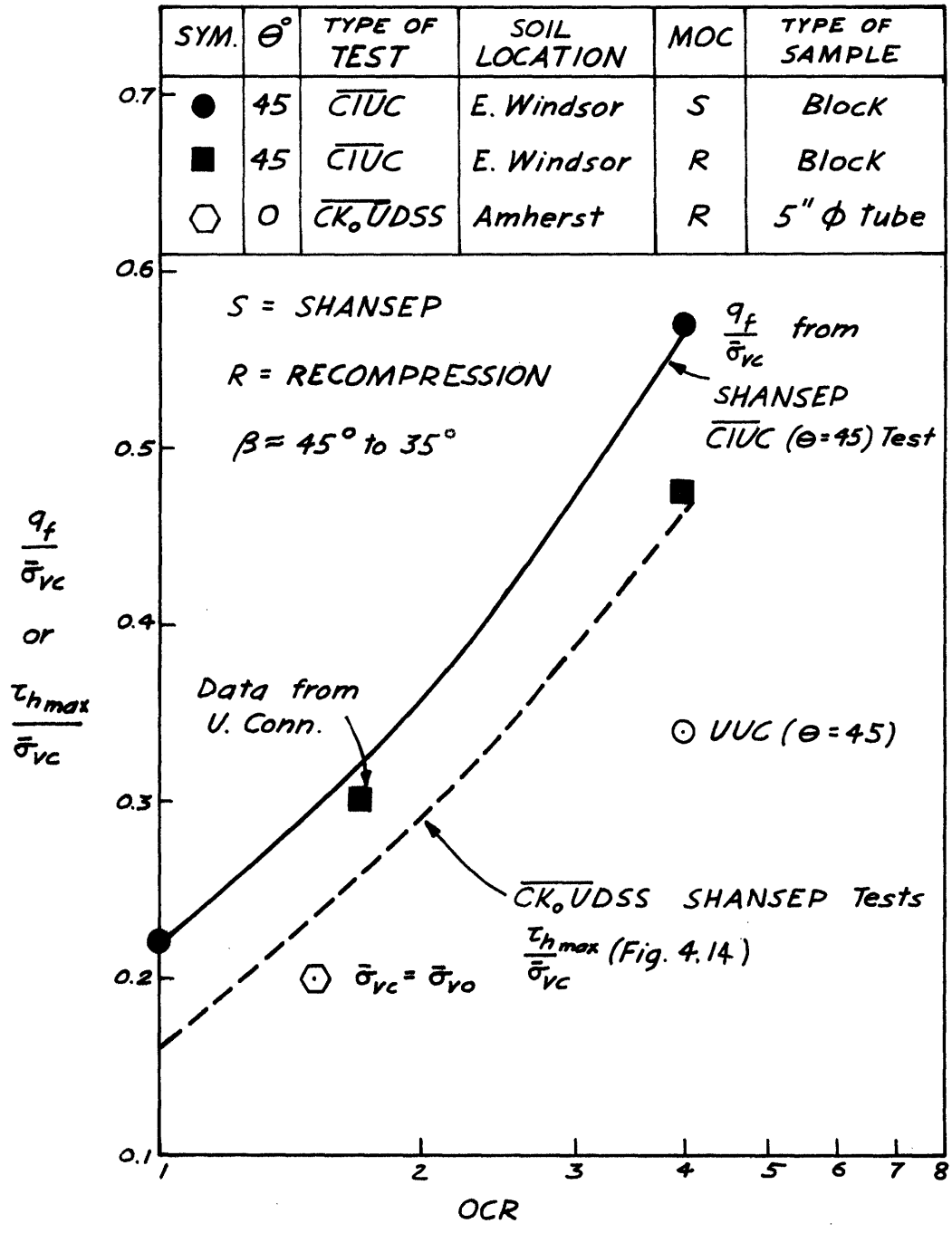


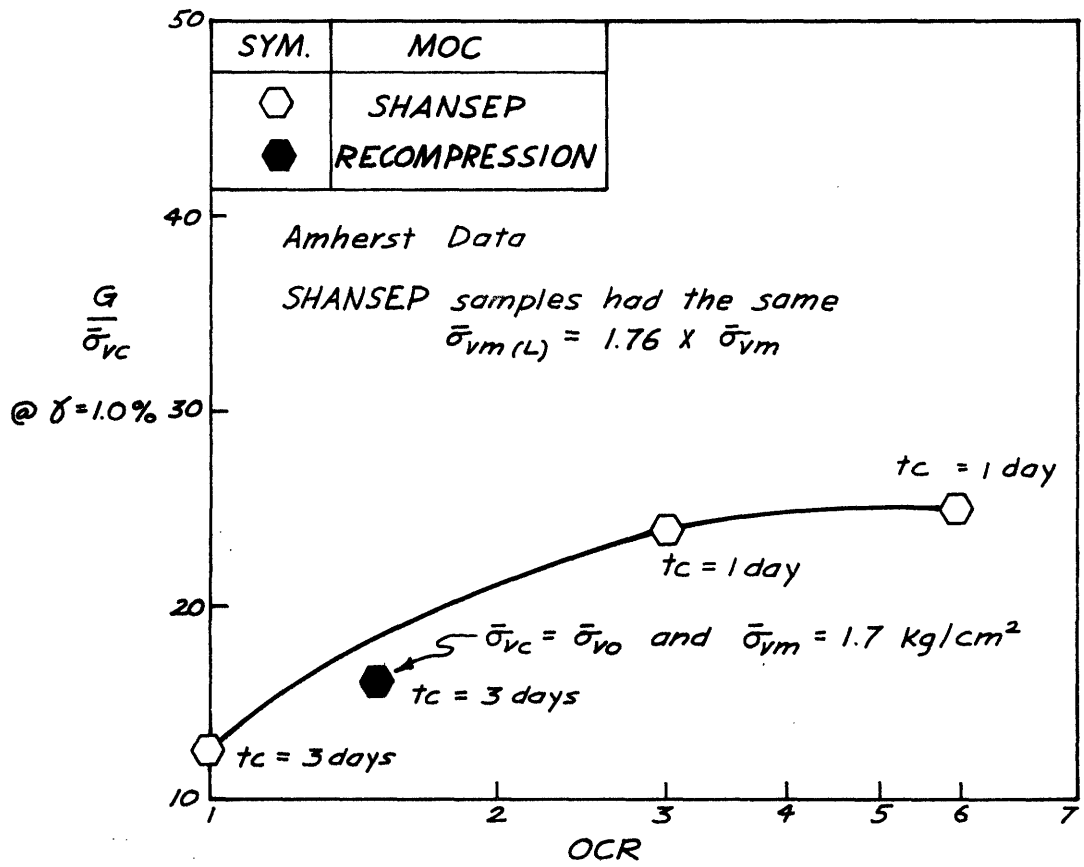
FIGURE 6.15

NESE FROM $\overline{CIUC}$ (θ=45)TESTS ON SHANSEP AND RECOMPRESSION SAMPLES



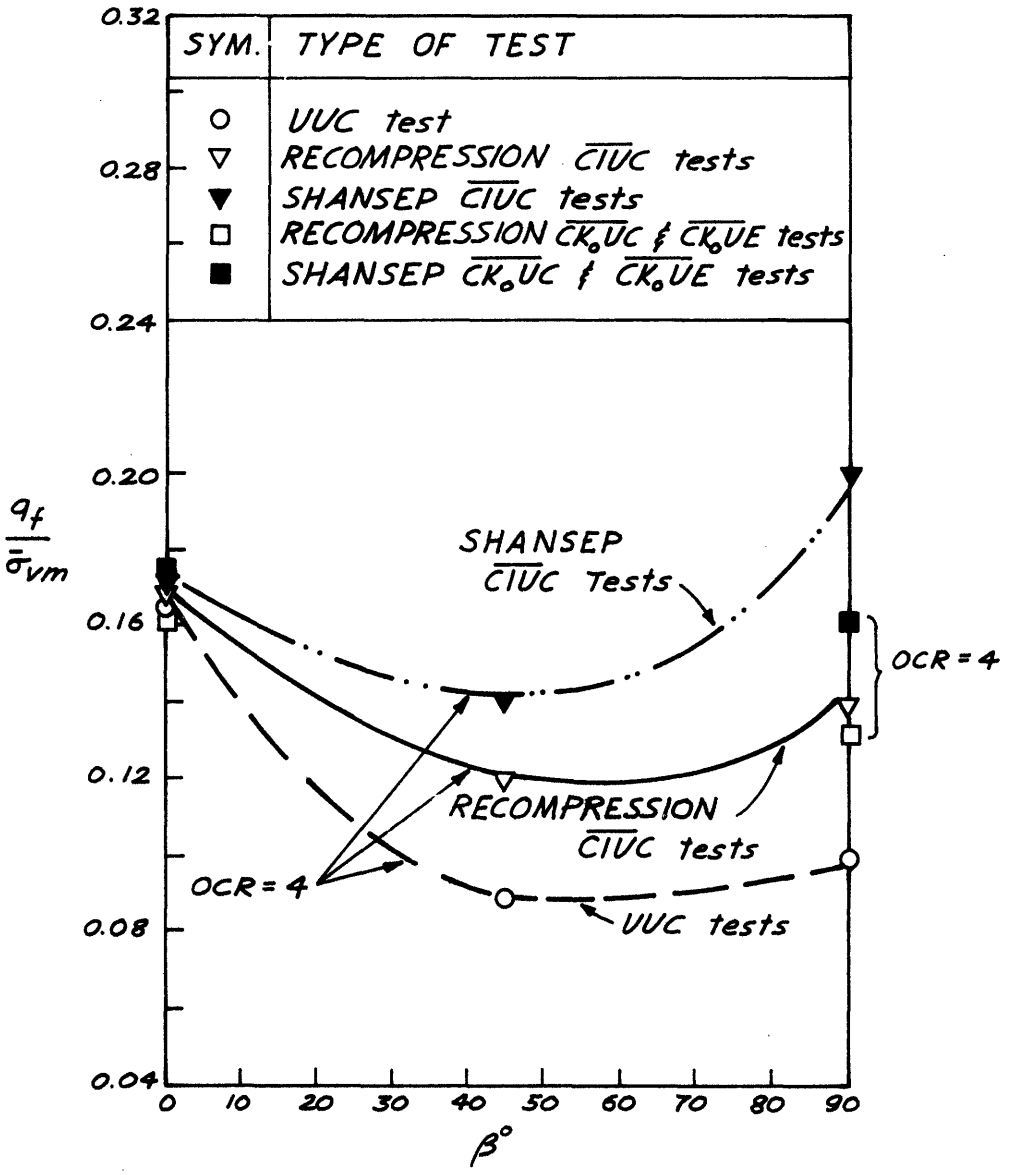
COMPARISON OF $\frac{q_f}{\bar{\sigma}_{vc}}$ VERSUS OCR FROM $\overline{CIUC}$ ($\theta = 45$), $\overline{CK_oUDSS}$, AND UU(45) TESTS ON SHANSEP AND RECOMPRESSION SAMPLES FROM AMHERST AND EAST WINDSOR

FIGURE 6.16



COMPARISON OF $\frac{G}{\bar{\sigma}_{vc}}$ AT $\gamma = 1.0\%$ FROM $\overline{CK_oUDSS}$ TESTS ON SHANSEP AND RECOMPRESSION SAMPLES

FIGURE 6.17



COMPARISON OF $\frac{q_f}{\bar{\sigma}_{vc}}$ FROM $\overline{CIUC}$ AND $\overline{CK_oU}$ TESTS ON SHANSEP AND RECOMPRESSION SAMPLES

FIGURE 6.18

7. INTERPRETATION OF ANISOTROPIC BEHAVIOR OF CONNECTICUT VALLEY VARVED CLAY

7.1 INTRODUCTION

As presented in Chapter 2, the term anisotropy means that the soil properties (e.g., q_f. A, $\bar{\phi}$, and $\bar{c}/\bar{\sigma}_{vm}$) vary with loading direction (defined by angle β; see Fig. 2.1) as they are measured in the laboratory using controlled tests with different directions of σ_{1f}. It is convenient to divide anisotropy as measured in the laboratory into three components, as follows:

1. Inherent Anisotropy--For varved clay, the inherent anisotropy includes:
   a. Structural Anisotropy: This component is a result of the mode of deposition and subsequent K_o consolidation that produces a preferred orientation of particles. K_o consolidation causes plately shaped particles to have a tendency to align themselves at right angles to the direction of the major principal stress.
   b. Material Anisotropy: This component of anisotropy results only from the layered nature of varved clay, the individual layers being considered as isotropic material.

   The two components of inherent anisotropy would

be expected to lead to changes in effective stress envelope, pore pressure parameter, and modulus with direction of loading.

2. Stress System Induced Anisotropy--This component of anisotropy results from the fact that different increments of shear stress are required to produce failure as the major principal stress varies between the vertical and horizontal directions when K_o is not equal to unity. In essence, stress system induced anisotropy results in changes in effective stress as σ_1 varies in direction during shear. This component occurs even if there is no change in properties due to inherent anisotropy.
3. Combined Anisotropy--This term (used in place of "in situ" anisotropy because the anisotropic behavior presented in this Chapter is measured via the SHANSEP method of consolidation) includes the total effects of inherent and stress system induced anisotropy.

The study presented in this chapter hopes to provide an understanding of the fundamentals controlling anisotropic undrained shear strength behavior. The study concentrates on the anisotropy in q_f interpreted in terms of effective stress, the anisotropy in NESE (both at q_f and $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$), and the anisotropy in pore pressure parameter A (considered at a particular strain). Some consideration is given to the anisotropy in undrained modulus.

7.2 METHOD OF INVESTIGATION

To provide this understanding, an attempt was made:

1. To separate the material anisotropy component from the structural and stress system induced anisotropy components so that the anisotropy in q_f, A, NESE and E_u primarily due to the layered nature of varved clay (i.e., the material anisotropy component) could be studied; and
2. To illustrate the general effects of structural anisotropy.

Combined anisotropy is studied in terms of effective stress from results of $\overline{CK_oUPSC}$, DSS and PSE test data, i.e., for plane strain test conditions. Although more extensive data exist for shear in TC and TE, interpretation of the results are complicated by effects due to changes in σ_2 which are difficult to separate out.

Details of the types of tests used for measuring material and combined anisotropy are presented below.

Measurement of Material Anisotropy

Conceptually, for plane strain loading, $\overline{CIU}$ PSC tests on samples with different inclinations that have been consolidated to extremely high stresses ($\bar{\sigma}_v >>> \bar{\sigma}_{vm}$) to minimize the in situ structural anisotropy should be used for this purpose. However, since this was not possible, SHANSEP $\overline{CIUC}$ tests on samples at different inclinations (θ=0, 45 and 90) at OCR's of 1, 4, and 20 were used to study material anisotropy (i.e., Option 2

in Table 2.1).

These $\overline{CIUC}$ tests have some drawbacks. The magnitude of σ_2 is controlled in tests of this type, but the direction of σ_2 relative to the varve orientation changes with sample inclination. This complicates interpretation of the results. Also, the values of $\bar{\sigma}_c$ used were only 4.0 kg/cm^2 (compared to $\bar{\sigma}_{vm}$ = 2.4 kg/cm^2 for the East Windsor varved clay which is used in the study), which is not high enough to eliminate the effects of structural anisotropy. Therefore, the word material is put in quotes, i.e., "material" anisotropy is used to emphasize the fact that the results are only an approximate measurement of the true material anisotropy.

Measurement of Combined Anisotropy

Since the anisotropic fabric is oriented in the same direction as the in situ soil and since shear starts from a K_o condition, SHANSEP $\overline{CK_oU}$PSC, $\overline{CK_oU}$DSS and $\overline{CK_oU}$PSE tests were performed for this purpose. Note, however, that results for OC $\overline{CK_oU}$PSC, PSE tests had to be estimated based on trends measured in $\overline{CK_oU}$C and $\overline{CK_oU}$E tests.

Effect of Structural Anisotropy Component

An attempt was made to approximately determine the structural anisotropy component. Conceptually, this component of anisotropy should be shown by comparing q_f values from isotropically consolidated PSC tests run on samples with different inclinations with those from $\overline{CK_oU}$ PSC, DSS and PSE tests

at OCR = 4 where K_o = 1.0 so that shearing of the $\overline{CIU}$ and $\overline{CK_oU}$ samples starts from the same conditions. Since this type of plane strain testing was not possible, SHANSEP $\overline{CIUC}$ test data were used for this purpose.

7.3 INTERPRETATION OF "MATERIAL" ANISOTROPIC BEHAVIOR

Test data from $\overline{CIUC}$ (θ=0, 45, 90) tests are plotted in Figs. 7.1 through 7.6 and summarized in Table 7.1. Interpretation of the anisotropic behavior arising primarily from the layered nature of varved clay for both NC and OC samples is as follows:

(1) The "material" anisotropy in $q_f/\bar{\sigma}_{vc}$ of varved clay results from anisotropy with respect to the normalized effective stress-strength parameters (Fig. 7.4), and to the pore pressure parameter A_f (Fig. 7.3 (c)), and as a result of this, to the excess pore pressure at failure (Fig. 7.4). It is seen that the variation in q_f with β is within 20% (Fig. 7.3(a)) and the ratio $q_f(\beta)/q_f(V)$ is independent of OCR (Fig. 7.3(b)). $q_f/\bar{\sigma}_{vc}$ from the horizontal samples are the largest and $q_f/\bar{\sigma}_{vc}$ from the inclined samples are the smallest (Fig. 7.3(a)). The inclined (45°) samples, which are sheared parallel to the varves, have the lowest NESE at q_f (Fig. 7.4) and for NC soil, also the largest A_f. Comparing vertical and horizontal samples ($4 \geq$ OCR ≥ 1), both being sheared across the varves, vertical loading always develops larger pore pressures and causes a lower NESE at q_f (Fig. 7.4). These both result in a lower

$q_f/\bar{\sigma}_{vc}$.

(2) The anisotropy in NESE at q_f when $\bar{p}_f/\bar{\sigma}_{vm} > 0.3$ (Fig. 7.4) or in $\bar{\phi}$ ($\bar{c} = 0$) of NC soil (Table 7.1) is due to "material" anisotropy and variation in the degree of mobilization of friction and perhaps cohesion during shear. Because of the incomplete mobilization of friction for vertical loading, NESE at q_f is lower than that at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$. For horizontal loading, since NESE at q_f is equal to that at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$, it is thought that the frictional component is fully mobilized at q_f (Bjerrum, 1973). As a result, NESE at q_f for horizontal loading is higher than for vertical loading when $\bar{p}_f/\bar{\sigma}_{vm} > 0.3$. Note that at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$, NESE for vertical and horizontal samples are identical. Because failure takes place within a clay layer, NESE at q_f for inclined loading is the lowest.

(3) The anisotropy in NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ is only the result of whether failure occurs across the varves or within a clay layer when shearing is parallel to the varves. The independency from inclinations of the failure plane (α = 60° in vertical sample and α = 30° in horizontal sample) of NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ for failure across the varves suggests that $\bar{\phi}$ and $\bar{c}/\bar{\sigma}_{vm}$ at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ of the clay and silt portions are isotropic. That is, structural anisotropy does not cause a change in the NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$. It then follows that the increase in NESE for failure across the varves (i.e., vertical and horizontal samples) compared to failure within a clay layer (θ=45° sample) is mainly the result of the increased strength of the silt layer. [CD direct shear test data (see Fig. 2.3 in Lacasse

et al., 1972) show a higher envelope for failure in a silt layer than in a clay layer, except at very high OCR values.] Furthermore, the NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$, for failure across the varves and for failure within a clay layer are considered to be material properties.

(4) The anisotropy in pore pressure A at $\varepsilon = 1.0\%$ is shown in Fig. 7.5. At the same strain ($\varepsilon = 1.0\%$), the horizontal samples at OCR's of one and four have smaller A and Δu than vertical samples, while results from inclined samples fall between these values. It is difficult to assess whether this behavior is mainly due to material or structural anisotropy, or a combination of both effects.

(5) In Fig. 7.6, the increase in $E_u(H)$ compared to that from vertical and inclined samples is probably due to a lower Δu and a larger horizontal drained modulus. An indication of a larger horizontal drained modulus compared to the vertical modulus is obtained from CRSC (constant rate of strain compression test) data on Hackensack Valley varved clay (Saxena et al., 1974). CR and RR of a vertical sample were larger than those from a horizontal sample at the same depth.

In summary, the "material" anisotropy causes:

1. Anisotropy in q_f due to anisotropy in both NESE at q_f and the pore pressure at failure ($q_f(H) > q_f(V) > q_f(45°)$). The $q_f(\beta)/q_f(V)$ ratio was found to be independent of OCR.
2. Anisotropy in NESE at q_f, though this behavior

is also due to variation of friction and cohesion during shear (horizontal NESE > vertical NESE > inclined NESE).

3. Anisotropy in NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$. This anisotropy, considered to be a true material property, depends upon whether failure occurs within the clay layer or across the varves, the latter leading to a significantly higher failure envelope.
4. Anisotropy in excess pore pressures, these generally being largest in vertical samples and lowest in horizontal samples.
5. Anisotropy in E_u at the same strain (ε = 1.0%) ($E_u(H) > E_u(V) \approx E_u(45°)$).

7.4 EFFECT OF STRUCTURAL ANISOTROPY COMPONENT

Figure 7.7 shows a plot of $q_f/\bar{\sigma}_{vm}$ versus β at OCR's equal to 1 and 4 from $\overline{CIUC}$ tests on samples with different inclinations and from $\overline{CK_oU}$PSC, $\overline{CK_oU}$DSS and $\overline{CK_oU}$PSE tests (results presented in Table 7.2). The shaded area for the OCR=4 data represents the combined effects of structural anisotropy and variations in σ_2. For OCR = 1.0, the shaded area also includes the effects of stress induced anisotropy and of shearing samples which have different initial shear stresses, since K_o is less than 1. In Fig. 7.8, an estimation of the structural anisotropy component is shown. Plotting $q_f(\beta)/q_f(V)$ versus β, where $q_f(V)$ is from $\overline{CIUC}$ ($\theta = 0$) and $\overline{CK_oU}$PSC tests,

is an approximate method for adjusting $\overline{CIUC}$ test q_f values to equivalent $\overline{CIUPSC}$ test conditions as discussed below.

As previously stated, the structural anisotropy component effect, at least conceptually, would be shown by data at OCR= 4 (i.e., when $K_o = 1$) from $\overline{CIUPSC}$ tests on samples with different inclinations and from $\overline{CK_oUPSC}$, $\overline{CK_oUDSS}$, and $\overline{CK_oUPSE}$ tests. However, it is reasonable to assume that $q_f(V)$ from "$\overline{CIUC}$ PSC (θ=0)" and $\overline{CK_oUPSC}$ tests would be essentially equal since K_o versus isotropic consolidation showed little effect on CU TC strengths (Section 5.3.1). In essence, Fig. 7.8 obtains the equivalent q_f from the plane strain condition by increasing q_f values for $\overline{CIUC}$ samples at inclinations of 0, 45, and 90 by a factor equal to

$$\frac{q_f(V) \text{ from } \overline{CK_oUPSC} \text{ test}}{q_f(V) \text{ from } \overline{CIUC} \ (\theta=0) \text{ test}} .$$

[Therefore, for example, the estimated q_f from a $\overline{CIU}$ PSC (θ=45) test is equal to q_f from $\overline{CIUC}$ (θ=45°) test x $\frac{q_f(V) \text{ from } \overline{CK_oUPSC} \text{ test}}{q_f(V) \text{ from } \overline{CIUC} \ (\theta=0) \text{ test}}$ and $\frac{q_f \text{ from } \overline{CIUPSC} \ (\theta=45) \text{ test}}{q_f(V) \text{ from } \overline{CIUPSC} \ (\theta=0) \text{ test}} =$ $\frac{q_f \text{ from } \overline{CIUC} \ (\theta=45) \text{ test}}{q_f(V) \text{ from } \overline{CIUC} \ (\theta=0) \text{ test}}$.] Also, there is no stress system induced anisotropy when $K_o = 1$. Thus, Fig. 7.8 represents an estimate of the effect of structural anisotropy on undrained strength.

As shown in Fig. 7.8, it is seen that structural anisotropy significantly decreases q_f for inclined and horizontal loading and therefore increases the anisotropy in q_f over

that due to material anisotropy. The increased anisotropy in q_f is probably due to changes in anisotropy in NESE at q_f and the pore pressure response, but these effects are difficult to estimate.

7.5 INTERPRETATION OF COMBINED ANISOTROPY

The combined anisotropy in q_f, A_f and NESE from $\overline{CK_oU}$PSC, $\overline{CK_oU}$DSS, and $\overline{CK_oU}$PSE data ($4 \geq OCR \geq 1$) are presented in Figs. 7.9 and 7.10 and Table 7.2. The method used to estimate the OC q_f values was presented in Section 5.3.3 and Fig. 5.24. A_f was computed* from K_o values (see Fig. 5.2) and estimated locations of NESE at q_f (presented in Section 5.3.3). In Fig. 7.10, the locations of NESE at q_f and at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ for NC and OC $\overline{CK_oU}$PSC and $\overline{CK_oU}$PSE samples are estimated as follows:

1. As shown in Fig. 5.22, the NESE at q_f when $\bar{p}_f/\bar{\sigma}_{vc} > 0.3$ for NC and OC $\overline{CK_oU}$PSC samples was estimated to be slightly above the NESE at q_f for NC and OC $\overline{CK_oU}$C samples based on NC $\overline{CK_oU}$PSC test data. At $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$, the NESE values for failure across the varves (a material property) is believed to hold for NC and OC $\overline{CK_oU}$PSC samples.
2. The NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ for failure across the varves is believed to be the same as the failure envelopes at q_f and at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ for NC and OC

*The values are sensitive to the assumed location of the NESE, especially at OCR=4.

$\overline{CK_oUPSE}$ samples, i.e., for these samples, the envelopes at q_f and $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ are identical.

With help from the study presented in Sections 7.3 and 7.4, interpretations of the combined anisotropy in q_f, A, and NESE of NC and OC samples ($4 \geq OCR \geq 1$) are as follows:

1. The combined ansiotropy in $q_f/\bar{\sigma}_{vc}$ (Fig. 7.9) results from:
   a. Anisotropy with respect to NESE at q_f (or the anisotropy in $\bar{\phi}$ ($\bar{c}$=0) for NC samples, Table 7.2);
   b. Anisotropy with respect to the pore pressure parameter A_f (Fig. 7.9), and
   c. Differences in the magnitudes of Δq_f due to the rotation of principal planes (i.e., the stress system induced anisotropy component) when K_o is less than 1.

   The anisotropy in A_f and the changes in Δq_f in turn lead to further anisotropy in the pore pressure at failure. q_f(H) is slightly smaller than q_f(V) even though the NESE at q_f is much higher for horizontal loading. The decrease results from significantly lower effective stress at failure. This occurs even though A_f for horizontal loading is lower, as illustrated below for data on NC soil.

Type of Test	$\bar{p}_f/\bar{\sigma}_{vc}$	A_f	$\bar{\phi}^o$
$\overline{CK_oUPSC}$	0.70	1.09	23.5
$\overline{CK_oUPSE}$	0.55	0.85	27

The significantly lower $q_f/\bar{\sigma}_{vc}$ for inclined loading in DSS tests results both from a lower NESE at q_f (Fig. 7.10) and probably from a lower value of $\bar{p}_f/\bar{\sigma}_{vc}$. For NC soil, the DSS $\bar{p}_f,\bar{\sigma}_{vc}$ is 0.49 (Fig. 5.33) compared to 0.55 for PSE.

2. As previously discussed, the anisotropy in A is due to both the material and structural components (i.e., due to inherent anisotropy). A at $\varepsilon = 0.3\%$ and $\Delta q/\Delta q_f = 1/2$ for NC $\overline{CK_oU}$PSE are smaller than for $\overline{CK_oU}$PSC as shown below.

Type of Test	A at $\varepsilon = 0.3\%$	A at $\Delta q/\Delta q_f = 1/2$
$\overline{CK_oU}$PSC	1.02	0.80
$\overline{CK_oU}$PSE	0.45	0.52

3. The anisotropy in NESE at q_f, when NESE at q_f is not equal to that at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$, or the anisotropy in Mohr-Coulomb $\bar{\phi}$ at q_f of NC soil are the result of inherent anisotropy and of variation in the degree of mobilization of friction and cohesion. For failure across the varves, since the mobilization of friction and cohesion are strain dependent, the much larger ε_f of a NC $\overline{CK_oU}$PSE test compared to that of a NC $\overline{CK_oU}$PSC test (ε_f = 4.0% versus 0.4%) partly explains why $\bar{\phi}$ at q_f from a horizontally loaded sample is larger. [Note that NC $\bar{\phi}$ at q_f and at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ from $\overline{CK_oU}$PSE tests are identical (Table 7.2).] Because failure occurs within a clay layer instead

of across the varves (i.e., the material anisotropy component), the NESE at q_f from $\overline{CK_oU}$DSS tests is the lowest. The fact that a larger difference between NESE at q_f from vertical and horizontal loading is obtained from $\overline{CK_oU}$ plane strain compression and extension tests than from $\overline{CIU}$C tests on vertical and horizontal samples (comparing Fig. 7.4 to Fig. 7.10) might suggest that the structural anisotropy could cause the anisotropy in NESE.

4. The anisotropy is NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ is only due to the material anisotropy component, i.e., failure across the varves versus failure within a clay layer. Based on several types of tests reported in Chapter 5, NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ for vertically and horizontally loaded samples are identical. Thus, the lower $\bar{\phi}$ at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ from NC $\overline{CK_oU}$PSE compared to $\overline{CK_oU}$PSC (Table 7.2) is due to larger $\bar{p}/\bar{\sigma}_{vc}$ of the $\overline{CK_oU}$PSE Sample.

For the sake of completeness, the combined anisotropy in undrained shear modulus is shown in Figs. 7.11 and 7.12 by comparing test data from $\overline{CK_oU}$DSS(C), $\overline{CK_oU}$DSS, and $\overline{CK_oU}$DSS(E) tests on NC soil. At low stress levels, inclined loading results in a lower modulus than for either vertical or horizontal loading. But at the higher shear stress level, little modulus anisotropy is measured in this type of testing. This

contradicts the results shown in Fig. 5.27 from PSC and PSE tests, but these data may be influenced by apparatus friction.

7.6 SUMMARY AND CONCLUSIONS

The factors that control anisotropic strength behavior of varved clay are the layered nature of the soil (the material anisotropy component), the prior K_o consolidation (the structural anistropy component) and the stress system induced anisotropy, especially important when K_o is low. Interpretation of the combined anisotropy in q_f, A, Δu, $\bar{\phi}$ and $\bar{c}/\bar{\sigma}_{vm}$ (or Mohr-Coulomb $\bar{\phi}$ and $\bar{c}$) of NC and OC samples ($1 \leq$ OCR ≤ 4) as shown fro the SHANSEP method of consolidation are as follows.

1. The anisotropy in q_f results from anisotropy in effective stress at failure and in NESE at q_f. Due to material anisotropy component, q_f(H) of NC and OC samples is the largest. With the additional effects of structural and stress system induced anisotropy components, the anisotropy in q_f increases from that due to the material anisotropy component and q_f(V) from $\overline{CK_oU}$PSC tests is larger than q_f(H) from $\overline{CK_oU}$PSE tests. The lower q_f(H) compared to q_f(V) from $\overline{CK_oU}$ plane strain tests results from the lower $\bar{p}_f$ of the $\overline{CK_oU}$PSE sample, though the NESE at q_f is higher. Mostly because of having the lowest NESE at q_f, q_f for inclined loading is the smallest.

2. The anisotropy in NESE at q_f, when NESE at q_f is not equal to that at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$, is due to inherent anisotropy component and variation in the degree of mobilization of friction and cohesion. For vertical loading, because of the incomplete mobilization of the friction and cohesion components of strength, NESE at q_f is lower than that at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ with NC soil. For shearing across the varves, because of differences in the degree of mobilization of friction and cohesion, NESE at q_f for vertically loaded samples is lower than that for horizontally loaded samples. This lowering is also due to the structural anisotropy component. Because failure takes place within a clay layer, NESE at q_f for inclined loading is the lowest.
3. The anisotropy in NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ is only dependent upon whether failure occurs within a clay layer or across the varves. Thus, the anisotropy in NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ is only due to the material anisotropy component. Despite having the same NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$, $\bar{\phi}$ ($\bar{c}$=0) at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ for NC $\overline{CK_oU}$PSC tests is larger than that from $\overline{CK_oU}$PSE tests, because $\bar{p}/\bar{\sigma}_{vc}$ at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ is smaller.
4. The anisotropy in pore pressure is due to anisotropy in A resulting from the inherent anisotropy and from differences in the magnitudes of Δq.

Due to the material anisotropy component, at the same strain, A for vertical loading is generally the largest and A for horizontal loading is the lowest. However, at the same strain, despite having a lower A, a $\overline{CK_oU}$PSE test can have higher pore pressure and a lower $\bar{p}/\bar{\sigma}_{vc}$ than a $\overline{CK_oU}$PSC test because of the much larger Δq.

SUMMARY OF TEST DATA FROM $\overline{CIUC}$ TESTS FOR THE INVESTIGATION OF "MATERIAL" ANISOTROPY

Avg. Value if more than one test

			$(\sigma_1 - \sigma_3)_{max}$					$(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$						
θ°	β°	OCR	$q_f/\bar{\sigma}_{vc}$	A_f	ε %	$\bar{\phi}^\circ$	$\frac{\bar{c}}{\bar{\sigma}_{vm}}$	$q/\bar{\sigma}_{vc}$	A_f	ε %	$\bar{\phi}^\circ$	$\frac{\bar{c}}{\bar{\sigma}_{vm}}$	$E_u/\bar{\sigma}_{vc}$ (2) at ε=1/2%	Shearing Directions (1)
0	0		0.245±0.005	1.41±.02	4.7±0.3	27±1.5	-	0.255	1.51	16.8	33.0	-	66	AV
45	45	1	0.210±0.005	1.61±.01	5.56±.2	23.5±1	-	.200±.01	1.93±.2	10.1±.2	26±1	-	44	PA
90	90		0.285±0.005	1.19±.00	9.5 ±.5	28±.5	-	.285±.005	1.19±.01	9.5±.2	28±.5	-	70	AV
0	0		0.685	0.240	11.2	22.5	.060	0.680	0.255	13.6	30.0	.017	120	AV
45	45	4	0.575	0.200	4.87	17.0	.045	0.550	0.270	8.7	22.0	.027	112	PV
90	90		0.785	0.130	7.4	30.0	.017	0.780	0.135	9.0	30.0	.017	180	AV
0	0		1.600	-0.050	6.3	30.0	.017	1.045	-0.155	6.3	30.0	.017	220	AV
45	45	20	1.330	-0.083	4.8	22.0	.027	1.330	-0.085	4.8	22.0	.027	220	PV
90	90		1.957	-0.134	7.1	30.0	.017	1.760	-0.070	3.4	30.0	.017	410	AV

Notes: (1) AV = Across the Varves
PV = Parallel to the Varves

(2) $E_u = \frac{\text{Axial Stress}}{\text{Axial Strain}}$

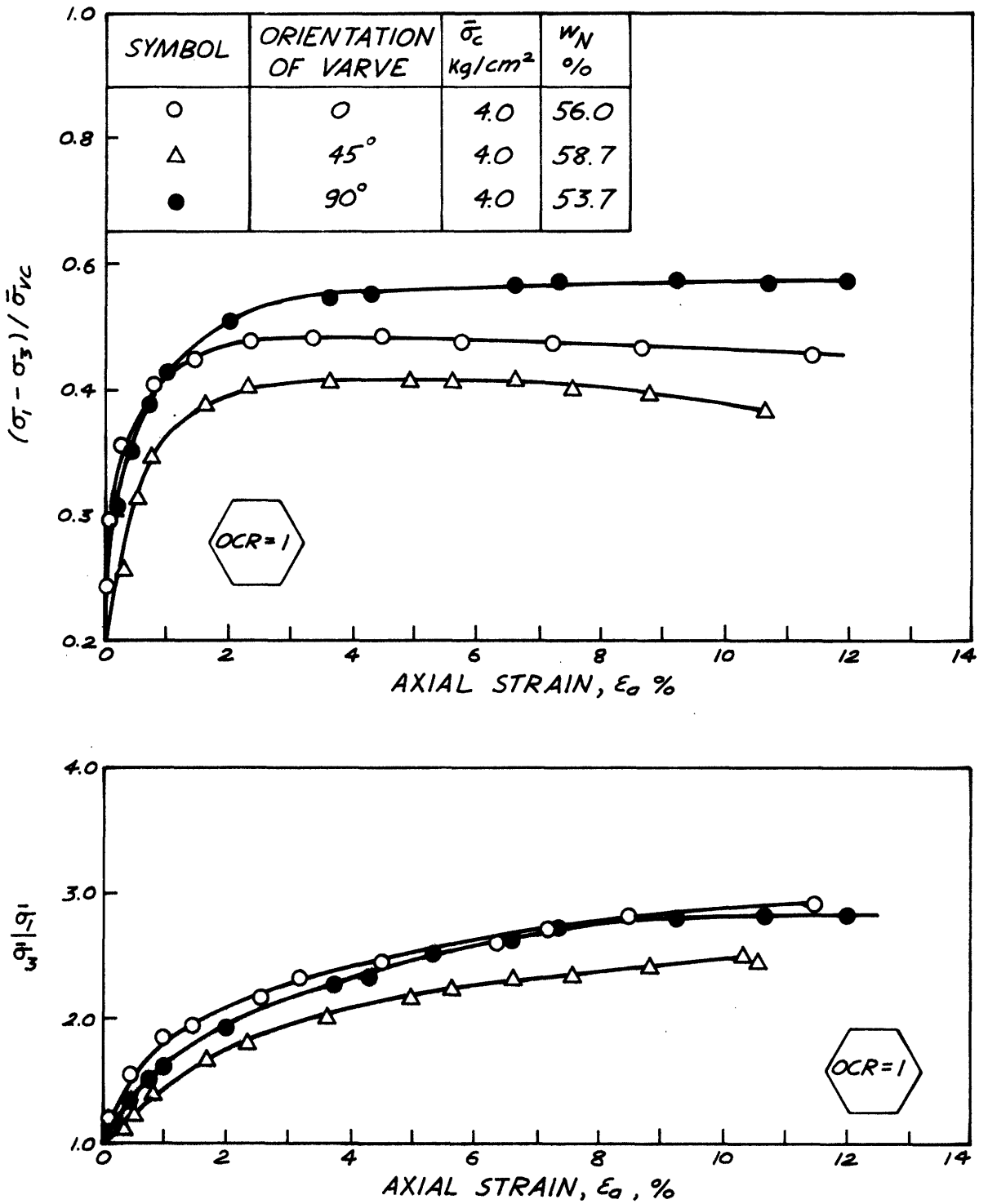
Table 7.1

SUMMARY OF MEASURED AND ESTIMATED DATA FROM $\overline{CK_oU}PSC$, $\overline{CK_oU}DSS$, AND $\overline{CK_oU}PSE$ TESTS FOR THE STUDY OF COMBINED ANISOTROPY

		$(\sigma_1 - \sigma_3)_{max}$				$(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$		Remarks
$\beta°$	OCR	$\frac{q_f}{\bar{\sigma}_{vc}}$	A_f	$\bar{\phi}°$	$\bar{c}/\bar{\sigma}_{vm}$	$\bar{\phi}°$	$\bar{c}/\bar{\sigma}_{vm}$	
0		0.28	1.09	23.5	--	33	--	
31[3]	1	0.19	1.48	23	--	26	--	
90		0.25	0.85	27	--	27	--	
0		0.47[1]	0.20[2]	16[1]	0.080[1]	30	0.017	Estimated Values for Plane Strain Tests
29[3]	2	0.33	0.82	17	0.045	22	0.027	
90		0.44[1]	0.58[2]	30	0.017[1]	30	0.017	
0		0.76[1]	0.08[2]	16[1]	0.080[1]	30	0.017	Estimated Values for Plane Strain Tests
37.5[3]	4	0.48	0.43	17	0.045	22	0.027	
90		0.72[1]	0.28[2]	30[1]	0.017	30	0.017	

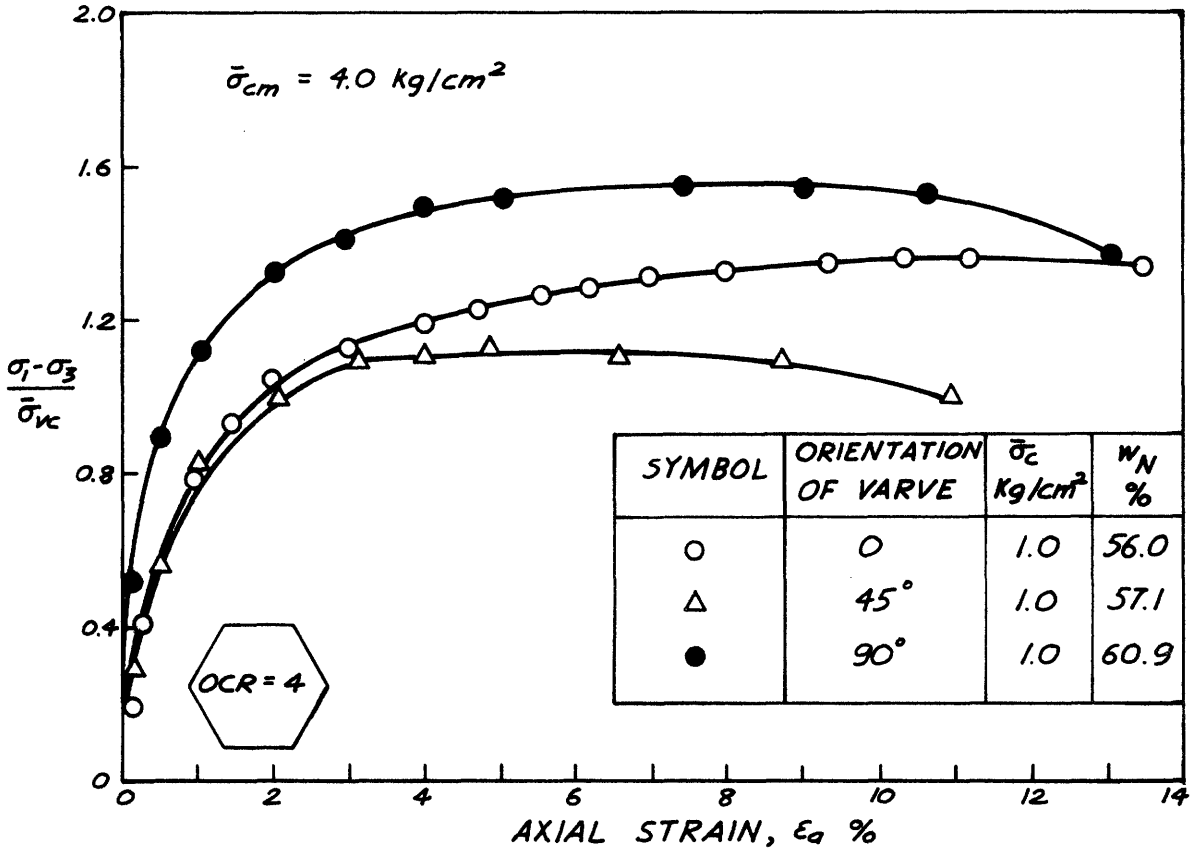
Notes: (1) Estimated values in Section 5.3.3.
(2) Estimated from knowing K_o from (Fig. 5.2), estimated NESE at q_f, and q_f values.
(3) From Section 5.4.3.

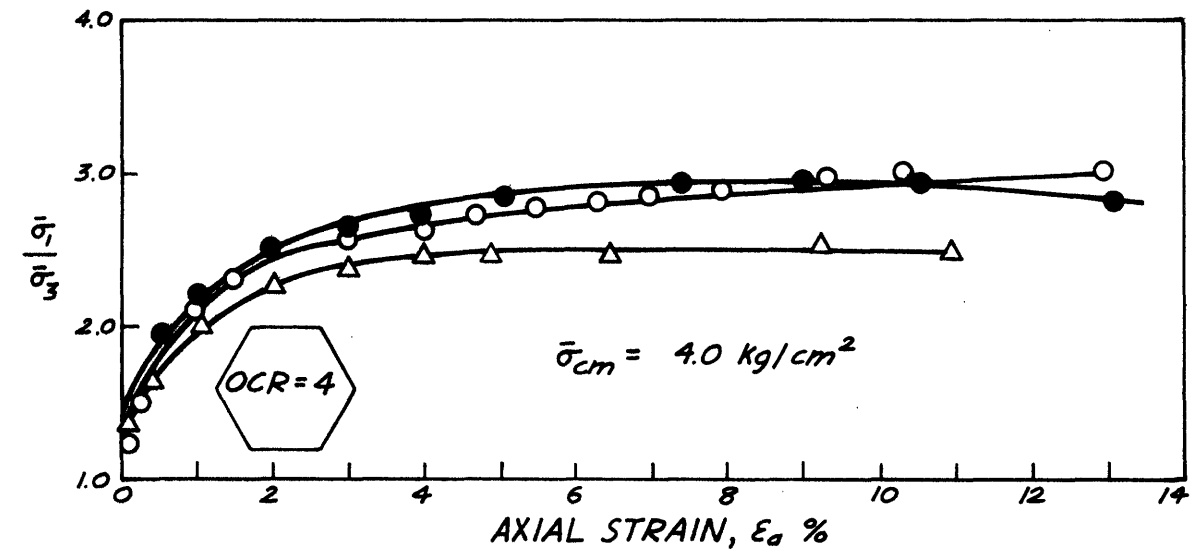
Table 7.2



EFFECT OF SAMPLE ORIENTATION ON STRESS VS STRAIN FROM $\overline{CIUC}$ TESTS ON SAMPLES OF NORMALLY CONSOLIDATED EAST WINDSOR VARVED CLAY

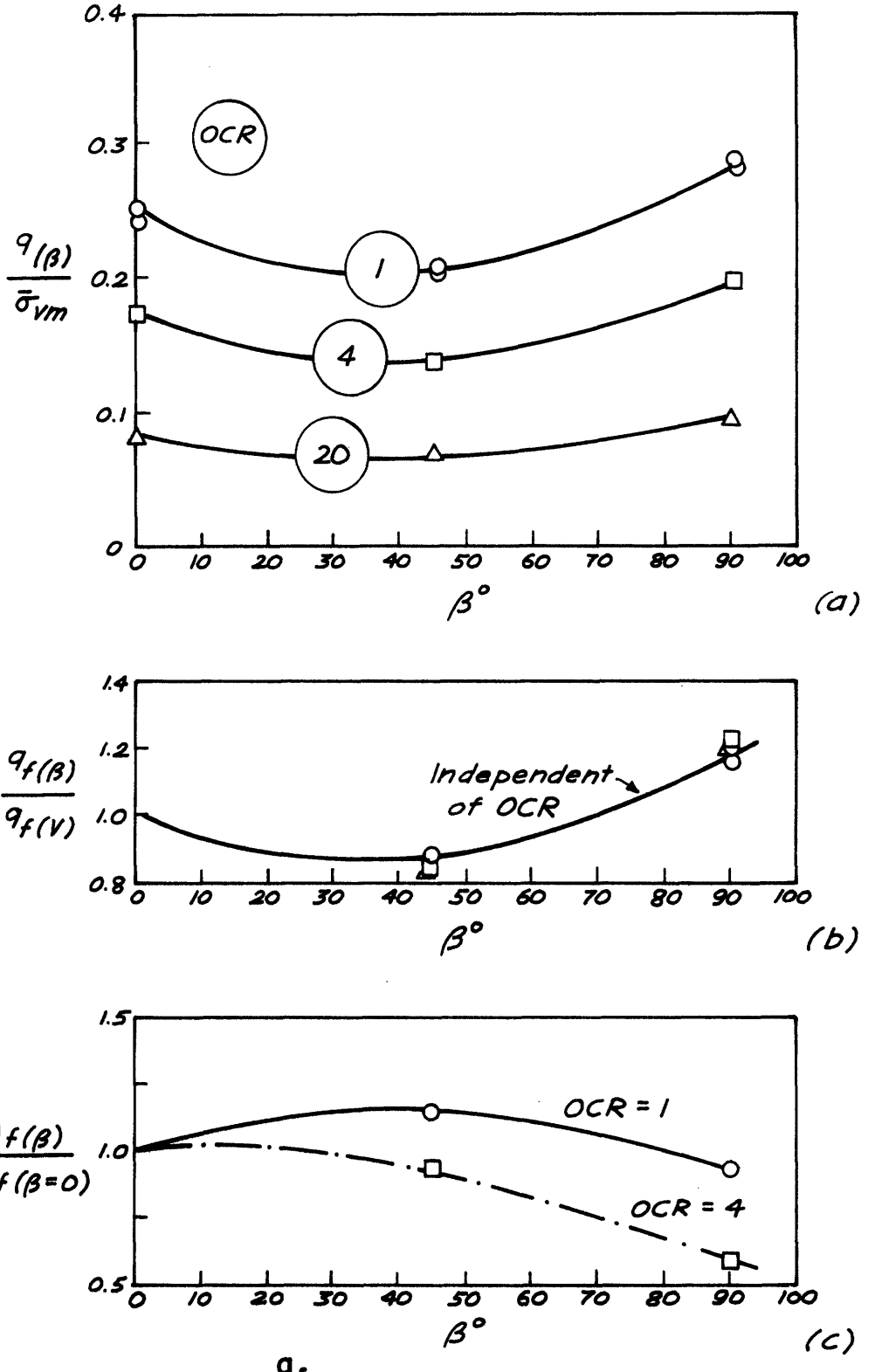
FIGURE 7.1





EFFECT OF SAMPLE ORIENTATION ON STRESS VS STRAIN FROM $\overline{CIUC}$ TESTS ON SAMPLES OF OVERCONSOLIDATED (OCR = 4.0) EAST WINDSOR VARVED CLAY

FIGURE 7.2



ANISOTROPY IN $\frac{q_f}{\bar{\sigma}_{vc}}$ AND A_f OF CONNECTICUT VALLEY VARVED CLAY RESULTING FROM "MATERIAL" ANISOTROPY COMPONENT

FIGURE 7.3

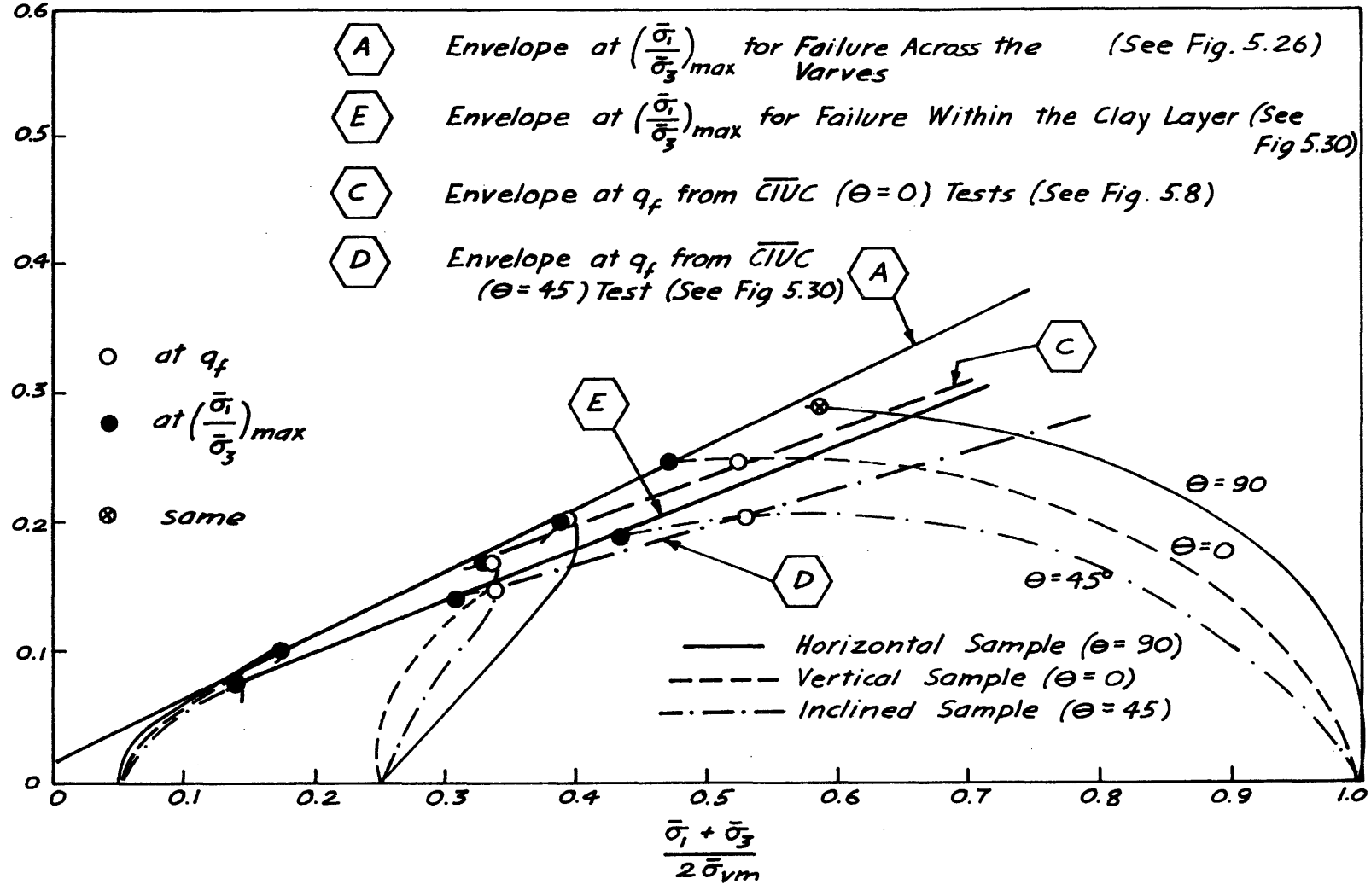
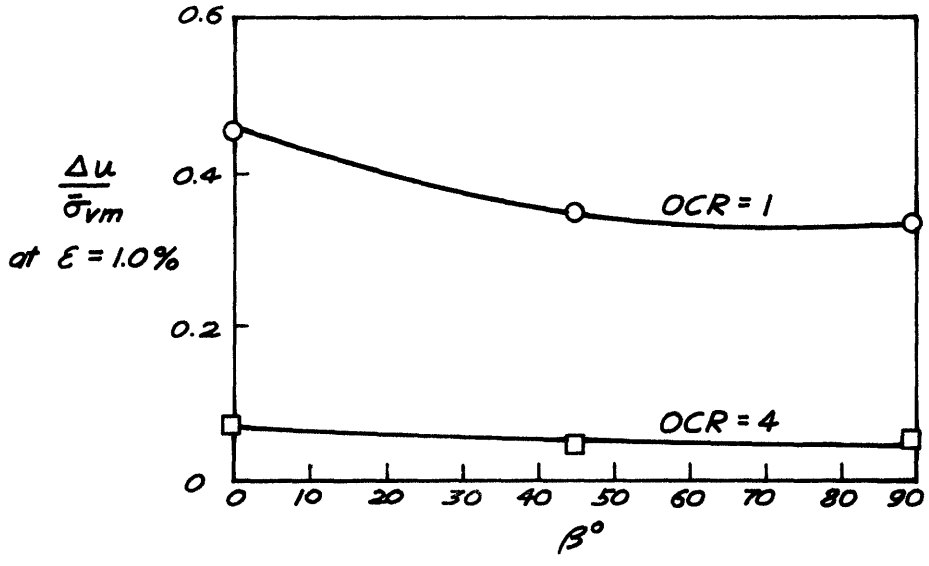
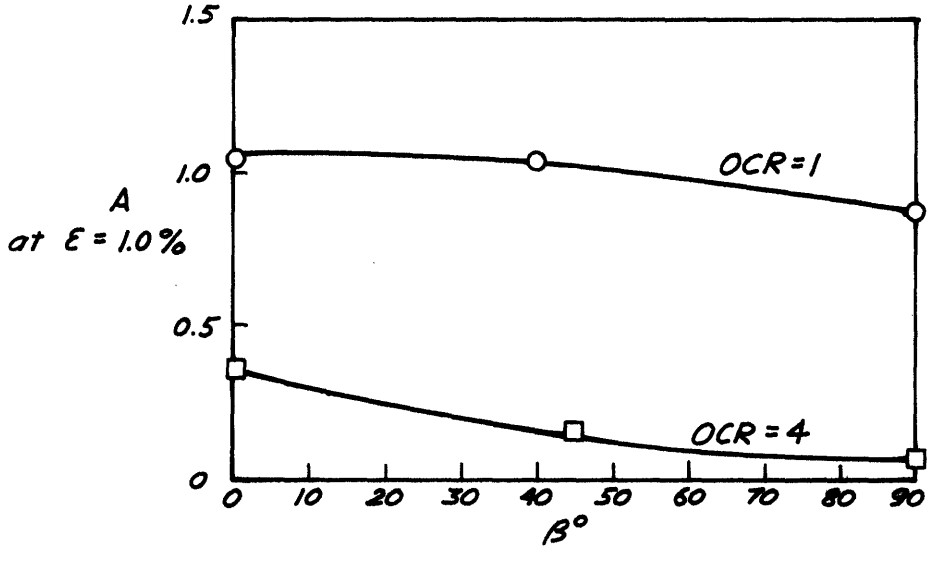


FIGURE 7.4

NORMALIZED EFFECTIVE STRESS PATHS AND ENVELOPES FROM CONN. VALLEY VARVED CLAY AT DIFFERENT VARVED INCLINATIONS



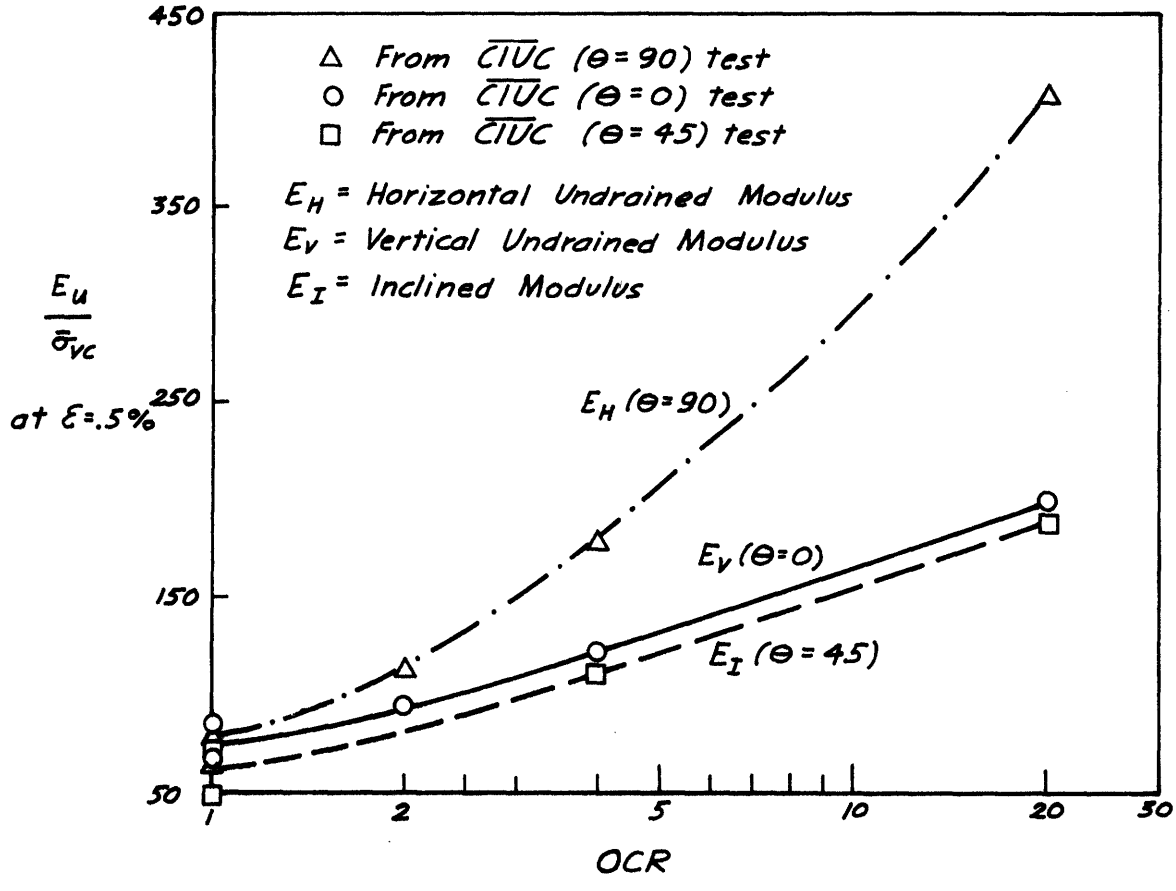
(a)



(b)

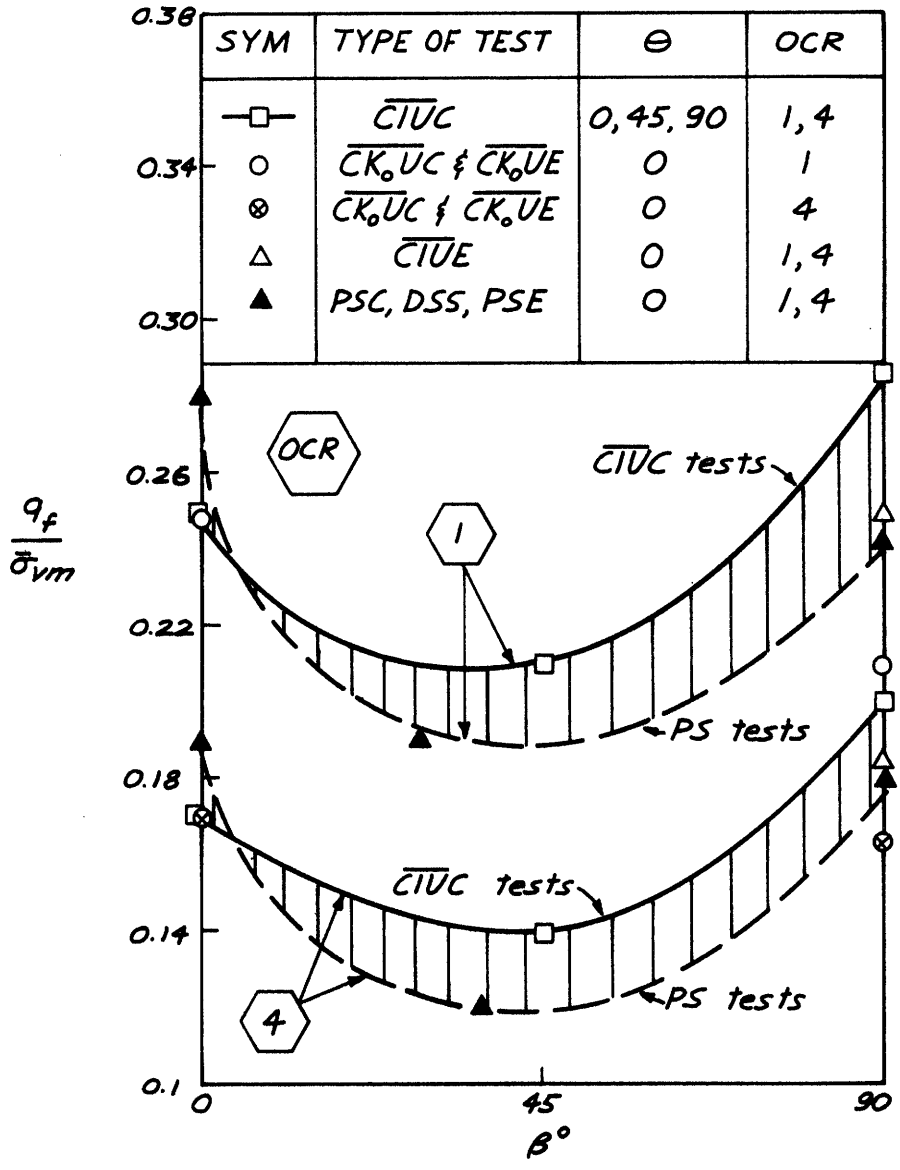
"MATERIAL" ANISOTROPY IN PORE PRESSURE AND PORE PRESSURE PARAMETER A AT $\varepsilon = 1.0\%$

FIGURE 7.5



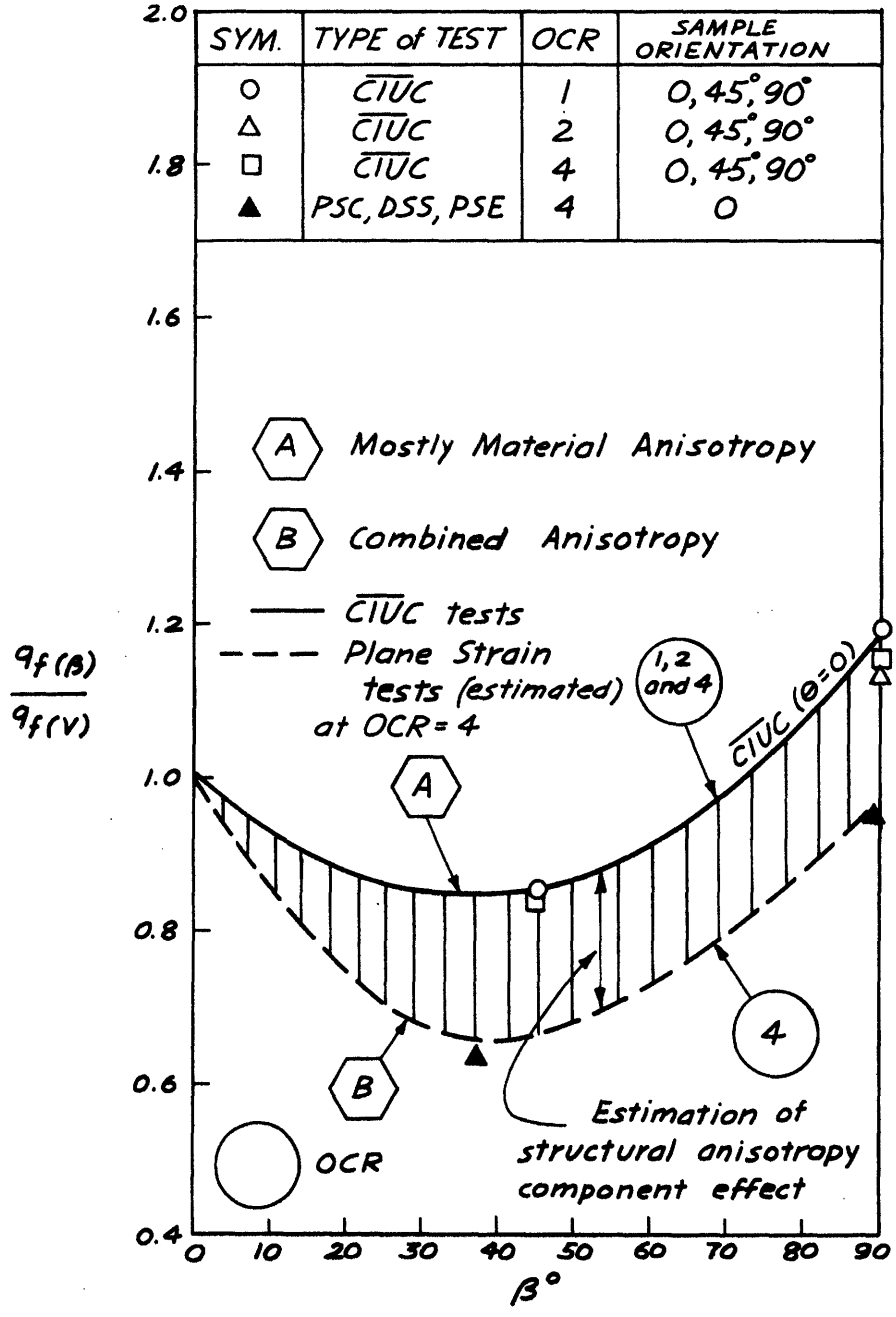
ANISOTROPY IN UNDRAINED MODULUS FROM $\overline{CIUC}$ TESTS

FIGURE 7.6



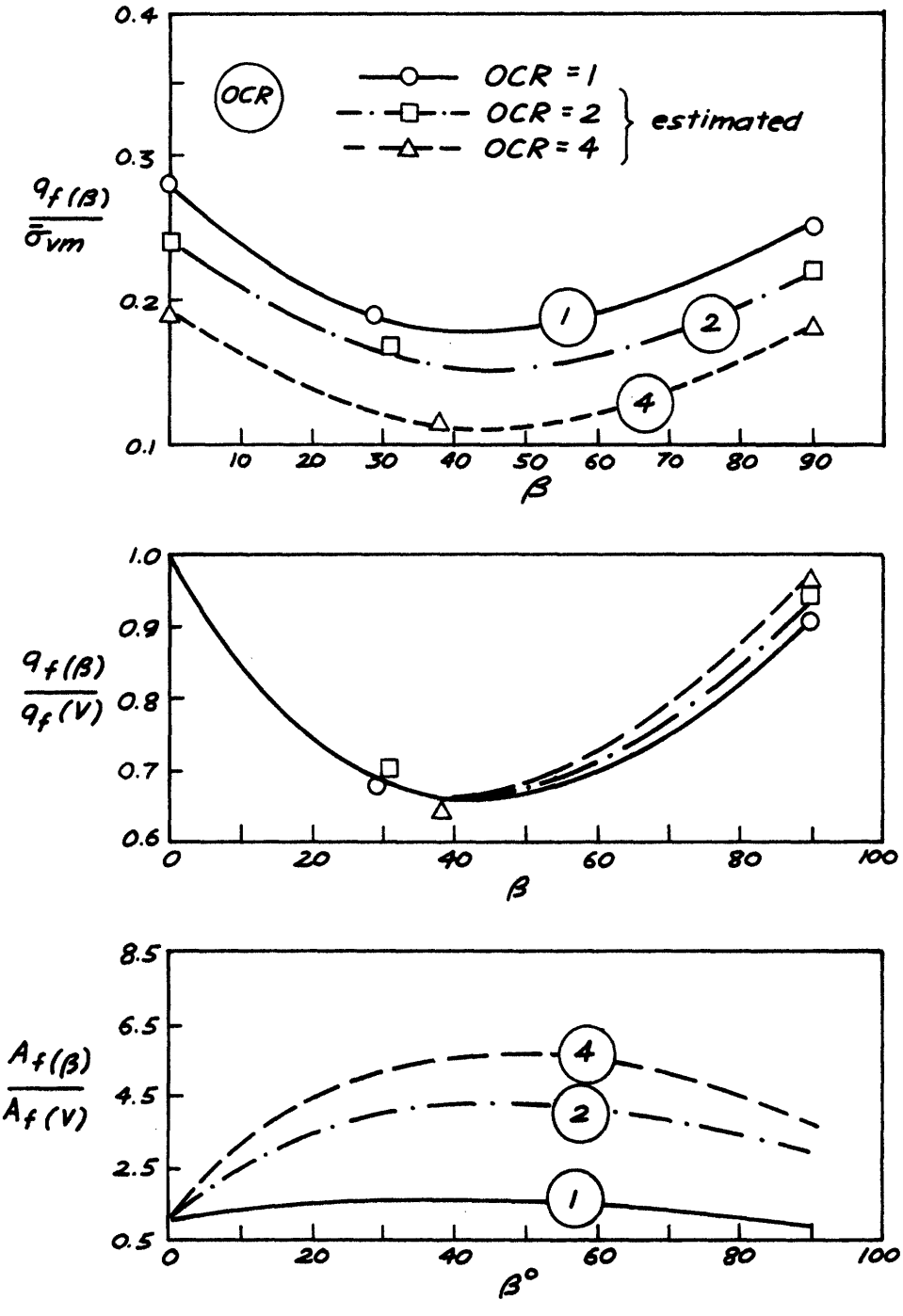
COMPARISON OF ANISOTROPY IN q_f MEASURED FROM PLANE STRAIN AND $\overline{CIUC}$ TESTS

FIGURE 7.7



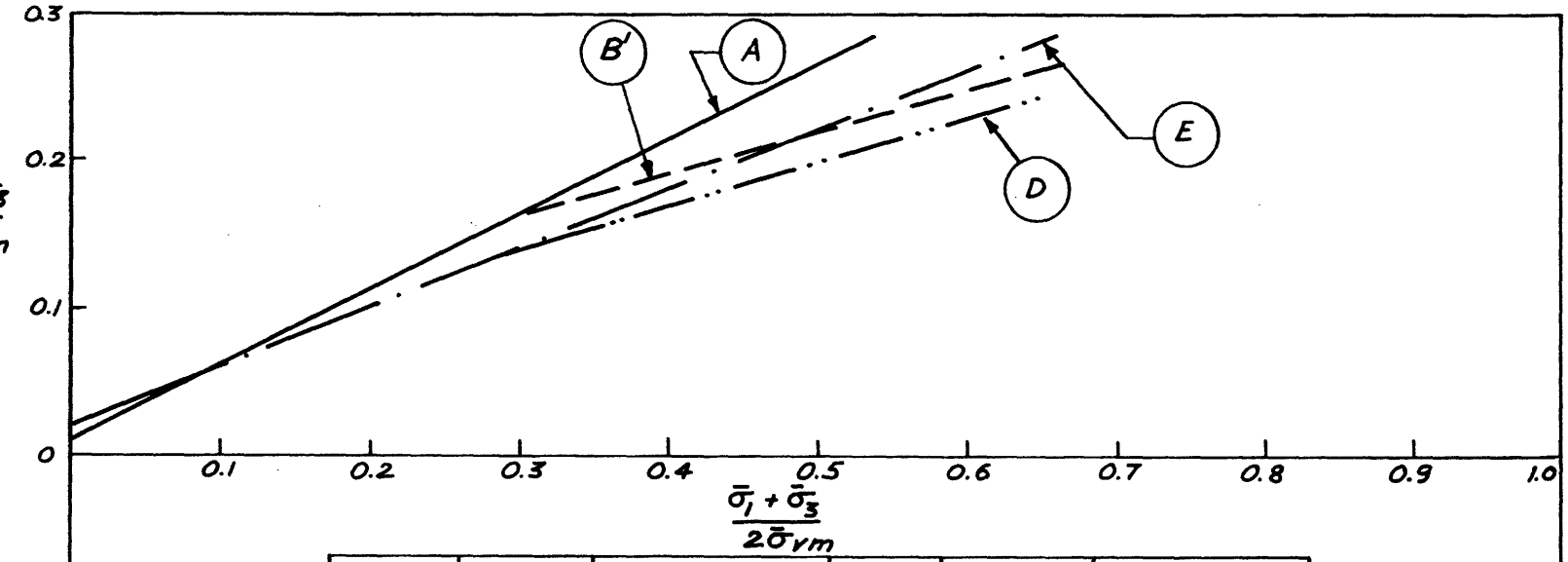
EFFECT OF STRUCTURAL ANISOTROPY COMPONENT ON ANISOTROPY IN q_f

FIGURE 7.8



COMBINED ANISOTROPY IN q_f OF CONNECTICUT VALLEY VARVED CLAY MEASURED FROM $\overline{CK_oU}$ PLANE STRAIN TESTS

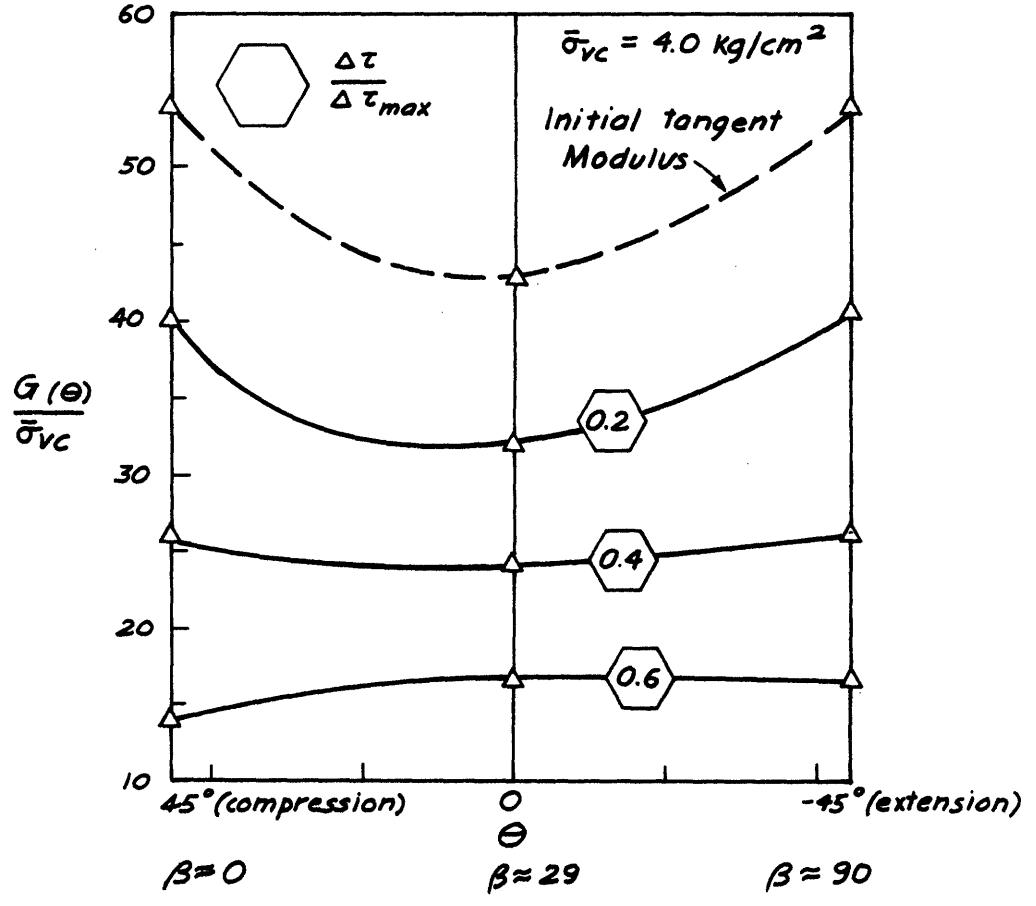
FIGURE 7.9



NESE	β°	CONDITION	$\bar{\phi}^\circ$	$\bar{c}/\bar{\sigma}_{vm}$	REMARKS
(A)	90	q_f and $(\frac{\bar{\sigma}_1}{\bar{\sigma}_3})_{max}$	30	0.017	
(A)	0	$(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$	30	0.017	
(B')	0	q_f	16	0.080	$\bar{P}_f/\bar{\sigma}_{vm} > 0.3$
(D)	30-45	q_f	17	0.045	$\bar{P}_f/\bar{\sigma}_{vm} > 0.3$
(E)	30-45	$(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$	22	0.027	

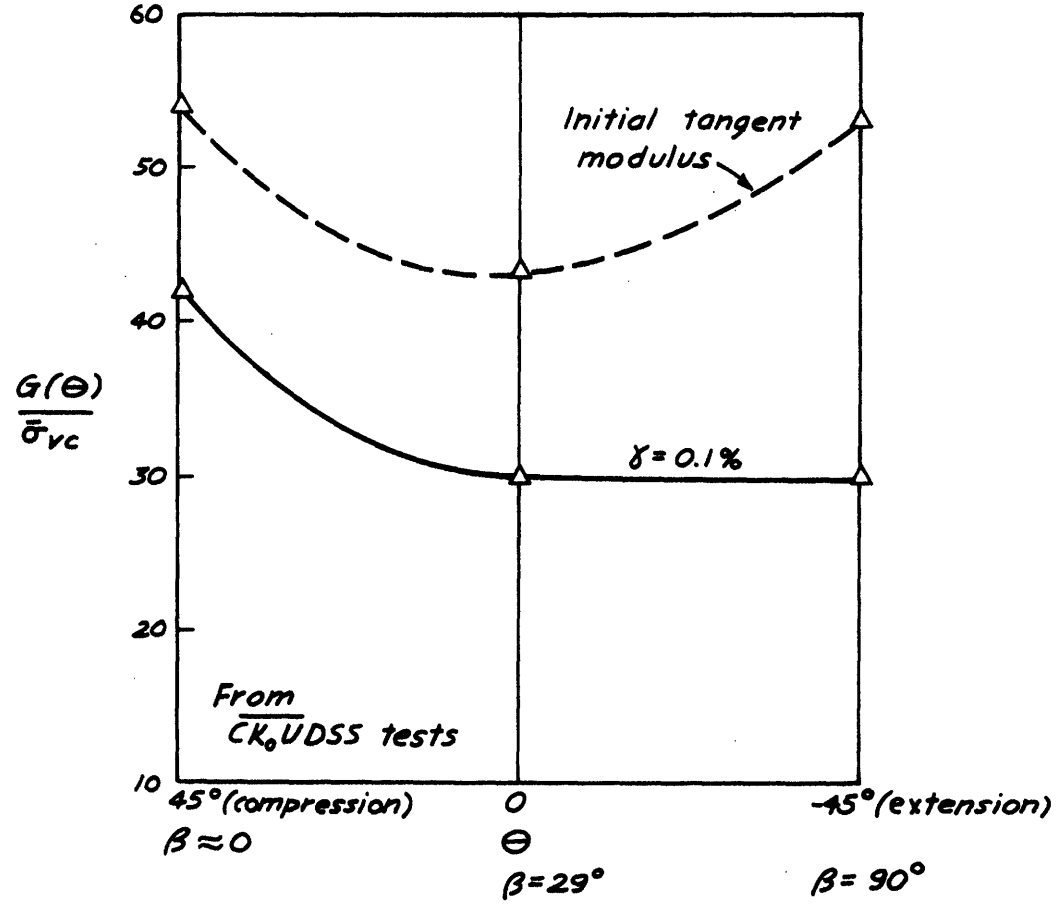
FIGURE 7.10

ESTIMATED COMBINED ANISOTROPY IN NESE FROM PLANE STRAIN LOADING



ANISOTROPY WITH RESPECT TO $\frac{G}{\bar{\sigma}_{vc}}$ OF NORMALLY CONSOLIDATED EAST WINDSOR VARVED CLAY AT VARIOUS APPLIED SHEAR STRESS LEVELS

FIGURE 7.11



ANISOTROPY WITH RESPECT TO $\frac{G}{\bar{\sigma}_{vc}}$ OF NORMALLY CONSOLIDATED EAST WINDSOR VARVED CLAY AT $\gamma = 0.1\%$

FIGURE 7.12

8. EVALUATION OF UNDRAINED STRESS-STRAIN-STRENGTH PROPERTIES OF CVVC FOR USE IN DESIGN

8.1 OBJECTIVES AND SCOPE

The purposes of this chapter are: (1) to present best estimates of the normalized stress-strain-strength parameters for use in the design of structures on Connecticut Valley varved clays, with the background showing how these parameters were developed; and (2) to recommend suitable testing procedures for developing anisotropic $S_u/\bar{\sigma}_{vc}$ parameters for other varved clay deposits.

The normalized stress-strain strength parameters are for the following analyses that might be included as part of the design process of structures (specifically an embankment) constructed on Connecticut Valley varved clays.

1. Total stress stability analyses to determine the factor of safety for undrained and partially drained conditions. Table 8.1 summarizes recommended uses for total stress analyses.
2. Analyses to predict stress, pore pressure, and deformation due to undrained loading.

Recommended parameters such as A_f and E_u are presented to use with available chart solutions and with MIT's finite element program FEECON. (See Table 8.2 for required input soil parameters.) The parameters for estimation of initial settlement based on D'Appolonia et al. (1971) method presented in Table 8.3. Soil parameters for effective stress stability

analyses for drained and undrained conditions, and those for predicting the amount and rate of long term settlements are not presented here. However, such information can be obtained from Ladd (1975).

The material presented in this chapter will concentrate on the background for the development of the soil parameters which is based upon the study presented in Chapters 4 through 7 (including the use of normalized FV data) rather than presenting the details of methods of analyses. Since the background for the development of soil parameters of Connecticut Valley varved clay is fully discussed, this approach can also be extended to develop normalized anisotropic S_u parameters for other varved clay deposits, presented in Section 8.6.2.

8.2 DEVELOPMENT OF UNDRAINED STRENGTH PARAMETERS OF CVVC FOR TOTAL STRESS STABILITY ANALYSES USING SHANSEP

Two requirements for obtaining reliable estimates of factor of safety from total stress analyses are:

1. The use of a suitable method of analysis; and
2. The selection of suitable undrained shear strength values which are compatible with the method of analysis and reflect in situ soil behavior.

Since the undrained shear strength behavior of varved clays is highly anisotropic, the method of analyses should be capable of taking this anisotropy into consideration. The Morgenstern and Price (1965) method of analysis (M-P) or similar "generalized"

methods, is recommended for all cases involving the input of anisotropic undrained strength data.

Strength data obtained from the SHANSEP method of consolidation will be used for developing the design S_u parameters for the reasons given below.

1. It is thought that the SHANSEP method should yield better S_u values for NC clays than the RECOMPRESSION method, and also give reasonable estimates of S_u for OC deposits.
2. NSP for different varved clays located in the Connecticut Valley as measured in several types of tests using the SHANSEP method of consolidation are practically identical.

This section presents: (1) a recommended method for obtaining suitable anisotropic undrained shear strength parameters measured with the SHANSEP method for use with M-P analyses; and (2) recommended values of SHANSEP anisotropic undrained shear strength parameters to use in the design of structures on Connecticut Valley varved clays.

8.2.1 Criteria for Developing Design Undrained Strength Parameters

Values of undrained shear strength parameters should be theoretically compatible with the method of analysis (e.g., Morganstern-Price (M-P) analyses as recommended in this thesis) and also reflect in situ behavior. The various factors that need to be considered in developing appropriate design values of S_u for total stress analyses are as follows:

1. The need to consider the anisotropy in q_f--As shown in Chapter 7, the values of peak strength and stress-strain characteristics are directional dependent. Though tnese properties can be measured for various loading directions, a practical method (e.g., that proposed by Ladd (1975) in Fig. 8.1) for considering anisotropy and other factors is needed.

2. The suitable definition of undrained shear strength for M-P analyses--M-P analyses accurately analyze failure surfaces composed of straight lines. Therefore, the undrained shear strength for such analyses is defined as the shear strength on the failure plane. In order to reflect in situ behavior and since effective rather than total stresses control this behavior, it is appropriate to adopt the inclination of failure planes from the effective stress Mohr-Coulomb failure criteria (i.e., the inclination of failure plane with the major principal plane is $(45° + \bar{\phi}/2)$.* Consequently, for failure at the peak strength (q_f), the shear stress on the failure plane, τ_{ff}, for a given mode of failure (PSC mode for vertical loading, DSS mode for inclined loading and PSE mode for horizontal loading), is equal to $q_f \cos\bar{\phi}$ ($\bar{\phi}$ at the

*Observations in the laboratory indicate that the inclination of failure planes, if visible, with the direction of the major principal plane is more than 45° (predicted by a total stress Mohr-Coulomb failure criteria) and is close to $45° + \bar{\phi}/2$ ($\bar{\phi}$ at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$).

q_f condition.

3. The strain rate effect--The stress-strain characteristics and hence q_f are dependent upon the strain rate, a decrease in the rate of strain decreasing q_f. The magnitude of this effect is uncertain. However, based on information from Ladd et al. (1977), it is thought that the slower in situ strain rate compared to that used in these $\overline{CK_oU}$ laboratory testing programs should cause relatively little reduction in q_f.

4. A correction for strain compatibility--Because of anisotropy in stress-strain-strength properties of soil, it is unlikely that the peak strength (q_f) for varying mode of failure can be simultaneously mobilized in strain softening material since these peaks occur at different strains. In fact, the formation of a distinct failure (slip) surface throughout the foundation soil is thought to be primarily a function of the strains which occur. Ladd (1975) proposed a method (Strain Compatibility Method) to account for strain compatibility assuming that the shear strains along a potential failure surface are equal. Note, however, that this assumption is not based on theory or observation.

8.2.2 Anisotropic S_u Parameters Developed by Ladd (1975)

This section presents Ladd's (1975) basic approach for obtaining anisotropic S_u design values. This was done before all test data (especially for horizontally loaded samples) presented in this thesis were available. The basic approach is as follows:

1. For simplicity, the three types of failure surfaces and corresponding strengths shown in Fig. 8.1 are recommended for considering S_u anisotropy for use with the M-P method of analysis.
2. The definition of undrained shear strength is that previously defined in Section 8.2.1. Thus, $\tau_{ff} = q_f \cos\bar{\phi}$ for PSC and PSE modes and τ_{ff} in the DSS mode is assumed to be equal to $\tau_{h_{max}}$.
3. The strain rate effect is not considered.
4. The strain compatibility effect is considered by using the strain compatibility method described below.

The recommended anisotropic S_u values from three modes of failure are considered to be equal to the laboratory measured shear stresses which yield the maximum average shearing resistance that could be mobilized at the same shear strain for the three modes of failure. The shear stresses and shear strains are determined as follows.

(a) The value of shear stress on the potential failure plane at a particular strain for either PSC or PSE modes of failure is equal to $q \cos\bar{\phi}$.

The values of $\cos\bar{\phi}$ selected were:

Mode of Failure	OCR	$\cos\bar{\phi}$
PSC	1	0.92
	≥ 2	0.90
PSE	≥ 1	0.90

(b) The shear stress on the potential failure plane at a particular shear strain for the DSS mode is equal to τ_h.

(c) To obtain shear strains, the measured vertical strains for PSC and PSE samples is multiplied by 2 (i.e., $\gamma = 2\ \varepsilon$ vertical). In DSS tests, γ is equal to the horizontal displacement divided by the height of the sample.

Table 8.4 shows how SHANSEP anisotropic S_u values at OCR-1.0 were determined. It is seen that the average maximum mobilized resistance, which occurs at a shear strain of 6%, is 8% less than the average of the peak shear strengths. At this strain, S_u for PSC is significantly less than the peak vertical shear strength (0.258), due to the strain softening. The DSS S_u is at τ_{ff} (= $\tau_{h_{max}}$ =0.16). The 6% shear strain is not large enough to fully mobilize the peak horizontal shear strength (0.225). In essence, for considering strain compatibility, τ_{ff} values for PSC and PSE were reduced by 16% and 6% respectively, i.e., the strain compatibility factor (SCF) have values of 0.86 and 0.94 respectively. For the DSS modes, the SCF is equal to 1.0.

For OC soil, since the laboratory plane strain stress-

strain-strength data were not available, OC q_f values for PSC and PSE were estimated from very limited TC and TE results (using a method similar to that presented in Fig. 5.24). SR values for OC PSC samples used by Ladd (1975) are essentially the same as those presented in Fig. 5.24. However, $q_f(H)$ for OC PSE samples was estimated by assuming that K_s ($K_s = q_f(H)/q_f(V)$) was equal to 0.89 independent of OCR. This means that the same* relationship of SR versus OCR was used for vertical and horizontal loading. To consider strain compatibility, the following values of SCF were used based on stress-strain behavior observed with other clays.

	SCF			Avg. SCF
OCR	PSC	DSS	PSE	
2	0.925	1.0	0.975	0.967
4	1.0	1.0	1.0	1.0

5. Finally, Ladd (1975) arbitrarily reduced the design strength by 5%, i.e., $S_u = q_f \cos\bar{\phi} \times SCR \times 0.95$.

8.2.3 Design Parameters Developed by the Author

The basic approach used by the author for developing anisotropic S_u design parameters generally follows that proposed by Ladd (1975), with the exceptions that:

1. The method for estimating OC SCR for each mode of failure is different. The author used results from triaxial tests (details given below).
2. A strain rate effect is considered. A strain

*As shown in Fig. 5.24, SR versus OCR relationships for PSC and PSE are not the same.

rate factor of 0.95 is arbitrarily selected for Connecticut Valley varved clays. (Note that based on Ladd et al. (1977), $\Delta q_f/\Delta \log t_f \leq 10 \pm 5\%$ at $\dot{\varepsilon} < 0.5\%/hr.$) Thus, $s_u = q_f \cos\bar{\phi}^*$ x SCF x 0.95.

3. The OC q_f values for PSC and PSE were estimated from Fig. 5.24. This gives the same q_f(V) but different values of q_f(H) compared to Ladd (1975).

SCF values for NC soil are those from Table 8.4. The basis for estimating SCF for OC (OCR=2 and 4) soil is as follows.

Comparing Tables 8.4 and 8.5, it is seen that the average SCF at OCR = 1.0 for plane strain and triaxial tests are essentially identical, i.e., by using the stress-strain-strength data from $\overline{CK_oU}C$, $\overline{CK_oU}DSS$, and $\overline{CK_oU}E$ tests. (For $\overline{CK_oU}C$ and $\overline{CK_oU}E$ tests, γ is equal to 1.5 ε vertical and the shear stress on the potential failure plane at a particular strain is equal to q $\cos\bar{\phi}$.) Note, however, that SCF values from PSC and PSE are different respectively from those from TC and TE. Furthermore, for both triaxial and plane strain loadings, SCF for the DSS mode is always equal to unity. On the basis of the NC data, the estimated OC average SCF values for plane strain loadings at OCR's of 2 and 4 are assumed to be equal to those from the triaxial loadings, shown in Tables 8.6(a) and 8.6(b). The OC SCF for the DSS mode was assumed to be equal to unity.

Details of how SCF for OC (OCR=2 and 4) PSC and PSE modes, and hence S_u values (Table 8.7), were evaluated are given below.

*$\cos\bar{\phi}$ values are identical to those proposed by Ladd (1975).

1. At OCR=4, since the average SCF is practically equal to 1, and since it is thought that the $\overline{CK_oU}$PSC sample should not exhibit significant strain softening, SCF for the PSC and PSE modes were selected to be 1.0.
2. At OCR = 2, SCF values for PSC and PSE modes are determined from the average SCF value from triaxial loading (avg. SCF = 0.96) and from the estimated $S_u(H)/S_u(V)$ ratio. As shown in Table 8.7, at OCR's of 1 and 4, this ratio is equal to 1.05. Thus, at an OCR of two, it is reasonable to adopt $S_u(H)/S_u(V) = 1.05$ for the determinations of $S_u(V)$ and $S_u(H)$.

Table 8.8 compares S_u values recommended by the author and by Ladd (1975). It is seen that the only difference is in $S_u(H)$ at OCR's of 2 and 4 (the author's $S_u(H)$ being 9% larger at OCR = 4 and 6% larger at OCR = 2).

8.3 COMPARISON OF THE USE OF SHANSEP ANISOTROPIC S_u VERSUS CORRECTED FV STRENGTH DATA

8.3.1 Introduction

The objectives of this section are to present:

1. The background of how the correction factor μ is estimated; and
2. Checking of the compatibility of using recommended SHANSEP strength data with M-P analyses for undrained total stress stability analyses.

This is done by comparing factors of safety from the simplified Bishop circular arc analyses with corrected field vane strengths and from M-P analyses with Ladd's (1975) SHANSEP anisotropic S_u parameters. (Note that since the author's recommended anisotropic S_u values are only slightly different from those recommended by Ladd (1975), the conclusions drawn from using Ladd's (1975) recommendation should also hold with the author's anisotropic S_u parameters.)

8.3.2 Recommended Correction Factor (μ) For Connecticut Valley Varved Clay

According to Bjerrum (1972), the correction factor μ empirically relates the field vane strength data to the value of the average in situ undrained shear strength on an assumed circular arc failure surface obtained from total stress analyses with $\phi = 0$ and c = field vane strength. Because of its empirical nature, reliable values of μ can only be determined from case studies.

The basis for recommending two values (μ = 0.85 for OC soil and μ = 1.0 for NC soil) for CVVC is as follows.

(1) The recommended μ = 0.85 for OC clay is based on case studies at Amherst and Northampton. The average OCR's of the foundation clay within the critical zone at these two locations were approximately 3.5 at Amherst (ranging from 1.5 to 8) and about 2 at Northampton. Thus, the correction factor of 0.85 was developed for moderately to heavily OC clay.

(2) The recommended μ = 1.0 for NC clay is based upon the trend established from a comparison of the average SHANSEP anisotropic design shear strength parameters with corrected and

uncorrected field vane strengths (Table 8.8). At OCR's of 2 and 4, (i.e., within the OCR range of the case studies), the average SHANSEP design $S_u/\bar{\sigma}_{vc}$ from both Ladd (1975) and the author is close to the corrected FV strength using $\mu = 0.85$. However, at OCR = 1.0, the average SHANSEP $S_u/\bar{\sigma}_{vc}$ at 0.19 is equal to measured field vane strength ($S_u/\bar{\sigma}_{vc} = 0.19$). Therefore, the value of μ for NC soil is taken to be equal to 1.0.

Since the use of corrected field vane strengths with Simplified Bishop circular arc analyses is an empirical approach and since the value of μ is dependent on several factors (e.g., strain rate, anisotropy, strain compatibility, etc.), its use has limitations which are discussed below.

(1) The value of μ is dependent upon the geometry of the failure surface and hence the geometry of the embankment. Because of the anisotropy in S_u, the average shear strength along a long, shallow wedge-type surface will be different from that along a deep circular surface. Thus, an embankment with wide berms would require a different correction factor μ from one without berms.

(2) The value of μ is dependent upon the stress history of the site. The anisotropy, the strain rate and the strain compatibility effects are dependent upon OCR. Plane strain data from Boston Blue clay (see Chapter 2) show that $q_f(H)/q_f(V)$ increases with increasing OCR, though a much smaller increase was observed from estimated data for the Connecticut Valley varved clays (see Chapter 7).

Furthermore, the use of circular arc analyses with corrected

field vane strengths for total stress stability analyses for the partially drained case is not reliable.

8.3.3 Compatibility of Using SHANSEP Anisotropic S_u with M-P Analyses

To check the compatibility of using SHANSEP anisotropic S_u with M-P analyses, it is necessary to see if the M-P analyses with SHANSEP strengths yield about the same factor of safety as that determined from the Simplified Bishop analyses with corrected field vane strengths. Note that the correction factor was determined from case studies.

Figure 8.3 compares values of the factor of safety and the locations of the critical surface of an embankment, used in a design problem presented in Ladd and Foott (1977), from M-P analyses with Ladd's (1975) SHANSEP anisotropic strength parameters and from Simplified Bishop circular arc analyses with corrected field vane strengths. The correction factor was equal to 0.85. Since the stress history, soil profile, and the geometry of the embankment used in the design problem are somewhat similar to those from the case studies, the fact that the factor of safety from Ladd's (1975) SHANSEP S_u is slightly lower than that from the corrected field vane strengths, and that these values of factor of safety are equal to or very close to 1, suggests that Ladd's (1975) SHANSEP S_u values used with M-P analyses are suitable to determine the factor of safety from total stress stability analyses for Connecticut Valley varved clays.

Since the difference between Ladd's (1975) and the author's

S_u values is only the value of S_u(H) for OC soil (the author's OC S_u(H) is about 10% larger), the factor of safety using the author's recommended S_u values with M-P analyses should also be suitable.

Comparing S_u values from triaxial loading, Tables 8.5, 8.6(a), and 8.6(b) to those from Ladd (1975) (Table 8.8), the use of S_u values from triaxial testing with M-P analyses will be conservative.

8.4 EVALUATION OF UNDRAINED SHEAR STRENGTH FOR TOTAL STRESS ANALYSES IN PRACTICE

The objective of this section is to recommend practical methods for evaluating the undrained shear strength of Connecticut Valley varved clays for total stress analyses used in undrained and partially drained cases (Table 8.1). This information is partly extracted from Ladd (1975).

8.4.1 Initial Undrained Shear Strength for Total Stress Analyses (Undrained Case)

The factor of safety for the undrained case determined by total stress analyses can be obtained from two approaches:

1. Corrected Geonor field vane strengths with Simplified Bishop circular arc failure analyses.
2. SHANSEP anisotropic shear strength parameters determined from Section 8.2 (Fig. 8.4) with M-P analyses.

Since the correction factor μ for Connecticut Valley varved clays

(presented in Section 8.3) is believed to be reasonably reliable and since the critical failure surface from Simplified Bishop analyses can easily be determined using computer programs, such as LEASE (Bailey, W.A. and Christian, J.T., 1969), the first approach is generally recommended for determining the initial factor of safety. However, as previously discussed for embankments with long berms, since the values of μ can be affected by the geometry of the slip surface, the use of the μ values recommended in Section 8.3 may be unsafe.

Since field vane strength data generally show some scatter, sufficient Geonor field vane tests should be performed to determine the change in $S_u(FV)$ with depth, and to ascertain whether or not there is a significant lateral variation. Upon knowing the stress history of the site, the plot of $S_u(FV)/\bar{\sigma}_{vc}$ versus OCR (Fig. 8.4) can also be checked. Section 8.6 presents recommended types of laboratory tests and also Geonor field vane tests for checking pertinent soil parameters.

8.4.2 Undrained Shear Strength for Total Stress Analysis in a Partially Drained Case

Total stress M-P analyses with SHANSEP anisotropic undrained shear strengths are recommended for determining the factor of safety for the partially drained case (Table 8.1) for three reasons.

1. SHANSEP undrained strength data can reasonably be used for estimating the gain in S_u with consolidation for stability analyses, even though consolidation of the foundation clay under the embankment

will not conform to K_o consolidation. The error in vertical undrained shear strength should be minor (Section 5.2.3); S_u(DSS) should increase when consolidated with a horizontal shear stress (Ladd and Edger, 1972); and the horizontal undrained shear strength should be unaffected since this portion of the failure plane will usually lie beyond the influence of consolidation from the embankment.

2. A method of analysis whereby the difference in S_u with different failure surface inclinations can be taken into consideration is needed.
3. M-P total stress analysis is relatively simple compared with effective stress analyses. Furthermore, because of the difficulty in predicting the pore pressure at failure, effective stress analyses are not recommended.

For the evaluation of S_u for M-P analyses of the partially drained case, the following steps are recommended.

1. Determine the initial stress history, i.e., $\bar{\sigma}_{vo}$ and $\bar{\sigma}_{vm}$ prior to construction.
2. Compute the increase in total vertical stress $\Delta\sigma_v$ versus depth at representative locations using elastic theory.
3. Determine the degree of consolidation, U, versus depth corresponding to the time at which the factor of safety is required.

4. Compute the increase in the vertical stress, $\Delta\bar{\sigma}_{vc}$, versus depth at representative offsets from $\Delta\bar{\sigma}_{vc} = U\Delta\sigma_v$.
5. Compute $\bar{\sigma}_{vc} = \bar{\sigma}_{vo} + \Delta\bar{\sigma}_{vc}$ versus depth (i.e., assume that $\bar{\sigma}_v$ does not change during undrained loading). Though this is not absolutely correct, the error is unimportant compared to other uncertainties in the analyses.
6. Divide the foundation clay into zones having the same $\bar{\sigma}_{vm}$ and $\bar{\sigma}_{vc}$ versus depth.
7. Compute values of the anisotropic S_u for the appropriate modes of failure (i.e., PSC, DSS, PSE) within each zone using the $S_u/\bar{\sigma}_{vc}$ versus OCR = $\bar{\sigma}_{vm}/\bar{\sigma}_{vc}$ curves plotted in Fig. 8.4.

Ladd (1975) presents an example problem using the above procedure for computing the factor of safety for the partially drained case.

The evaluation of S_u from SHANSEP anisotropic shear strength parameters (Fig. 8.4) requires an accurate evaluation of the stress history, i.e., $\bar{\sigma}_{vo}$ and $\bar{\sigma}_{vm}$. The value of the initial in situ vertical effective stress ($\bar{\sigma}_{vo}$) is determined from measurement of total unit weights and the pore pressure conditions. The latter requires a knowledge of the elevation of the water table and a check to determine whether or not the pore pressures are hydrostatic. The maximum past pressure, $\bar{\sigma}_{vm}$, is commonly evaluated from the results of consolidometer tests. Ladd (1975) discussed appropriate methods for estimating $\bar{\sigma}_{vm}$ from compression curves.

8.5 RECOMMENDED PARAMETERS FOR UNDRAINED ANALYSES OF PORE PRESSURE AND DEFORMATION

The objective of this section is to present soil parameters to be used in making predictions of pore pressure and undrained deformations. Either one of the following two approaches, distinctly different in their level of sophistication and amount of information yielded, is recommended for such predictions. The first approach (Table 8.3) employs hand calculations and the theory of elasticity corrected for effects of local yielding (D'Appolonia, Poulos, and Ladd, 1971). The required soil parameters for this approach are: (1) E_u, initial shear stress ratio (f), and the undrained factor of safety for the predictions of the initial settlement, ρ_i; and (2) Skempton's (1954) pore pressure parameter, A, for pore pressure predictions. The second approach, still being developed, employs the finite element program FEECON for estimating the undrained deformations, stresses, and pore pressures throughout the foundation clay. This approach requires hyperbolic undrained stress-strain-strength parameters (including anisotropic effects), and the pore pressure parameter, $a(\beta)$. Table 8.2 lists the required soil parameters for the finite element program, FEECON.

The presentation will include background for the development of these soil parameters and a brief outline of the procedures.

8.5.1 Parameters for Predictions Using Elastic Theory

(A) Prediction of Initial Settlement

The relationship for prediction of initial settlement (D'Appolonia, Poulos, and Ladd, 1971) at the center of the

loaded area is given below.

$$\rho_i = \frac{\Delta\sigma_v \; B \; I_p}{E_{u(field)} S_R} \qquad \text{Eq. 8.1}$$

where

$\Delta\sigma_v$ = applied vertical stress;

I_p = influence factor;

B = width of the loaded area;

$E_{u(field)}$ = in situ undrained modulus;

S_R = correction factor for local yielding

$$= \frac{\text{initial settlement } (\rho_i)}{\text{elastic settlement } (\rho_e)}$$

The values of S_R can be obtained from Fig. 8.7. S_R is a function of: (1) the initial shear stress ratio (f) (Fig. 8.6); (2) the applied shear stress ratio, $\Delta q/\Delta q_f$, = 1/FS; and (3) the ratio of the depth of the foundation (H) to the width of the loaded area (B). The coefficient, I_p, can be obtained from charts in Poulos and Davis (1974).

For the evaluation of $E_{u(field)}$, a correction factor needs to be applied to the measured E_u values given in Fig. 8.5 for the following three reasons:

(1) The laboratory SHANSEP E_u is generally lower than that from the RECOMPRESSION method, though the difference is small for vertical loading (Chapter 6). The in situ E_u may also be somewhat different from that obtained from the RECOMPRESSION method.

(2) Eq. 8.1 requires a single value of E_u, while the in situ E_u varies with loading direction. Though E_u/S_u values from SHANSEP $\overline{CK_oUC}$ tests are only somewhat larger than those from

SHANSEP $\overline{CK_oU}$DSS tests (Fig. 8.5), the larger value of S_u from $\overline{CK_oU}$C tests compared to that from $\overline{CK_oU}$DSS tests ($q_f(V) > \tau_{h_{max}}$) causes a significant increase in E_u. E_u from $\overline{CK_oU}$E tests on overconsolidated RECOMPRESSION samples are also higher than those from $\overline{CK_oU}$C tests (Chapter 6), though the NC value measured using SHANSEP is lower.

(3) Although a correction is made for local yielding (Eq. 8.1), an error in computing ρ_i still exists because of fitting a straight line to curved σ-ε data used in the linear elasticity theory. The values of secant modulus decrease with decreasing factor of safety.

The correction factors presented below are from Ladd (1975) based on a case study of am embankment at Northampton, Mass. (Connell et al., 1973).

Factor of Safety	$\frac{E_{u(field)}}{E_u(DSS)}$
≥ 2.5	2 - 3
1.25 - 2.5	1.5 - 2.0
≤ 1.25	1.0 - 1.5

The factor of safety is determined from the method recommended in Table 8.1. The following are the five steps used in the evaluation of $E_{u(field)}$.

1. Determine the average OCR, average corrected field vane strengths, and the undrained factor of safety. The maximum depth for determining the average OCR and average field vane strength is recommended to be between 1.5 to 2.0 B.
2. Equate the applied stress ratio to the inverse of

the factor of safety.

3. Determine E_u/S_u from Fig. 8.5 at the appropriate OCR and $\tau_h/S_u = 1/FS$.
4. Compute an average $E_{u(DSS)}$ using the above E_u/S_u value and the average field vane strength.
5. Select the upper and lower estimates of $E_{u(field)}$ from the correction factors given above and compute $E_{u(field)}$ from: $E_{u(field)}$ = correction factor x $E_{u(DSS)}$.

(B) Prediction of Excess Pore Pressure

Pore pressure predictions by hand-calculations are obtained using Skempton's A parameter and Eq. 8.2 (for B = 1.0) given below.

$$\Delta u = \Delta\sigma_3 + A(\beta)(\Delta\sigma_1 - \Delta\sigma_3) \text{ for } \Delta q/\Delta q_f \leq 1.0 \qquad \text{Eq. 8.2}$$

The magnitudes of $\Delta\sigma_1$ and $\Delta\sigma_3$ are computed from the theory of elasticity. (See Poulos and Davis (1974) for comprehensive collection of graphs, tables and formulas.) Figure 8.8 presents the recommended A parameter for vertical, inclined and horizontal loading at $\Delta q/\Delta q_f$ = 0.5 and 1.0 versus OCR.

In Fig. 8.8, the recommended A values for NC soil are obtained from SHANSEP $\overline{CK_oU}$PSC and $\overline{CK_oU}$PSE tests since it is believed that the SHANSEP method of consolidation should yield the better value of A than the RECOMPRESSION method. However, for OC samples, since the RECOMPRESSION method yields more realistic values of A than the SHANSEP method (see Chapter 6), OC A_f for plane strain loading are estimated from the OC

RECOMPRESSION data. For both vertical and horizontal loading, OC A_f values for plane strain loading were estimated from the value of NESE at q_f for RECOMPRESSION $\overline{CK_oUC}$ and $\overline{CK_oUE}$ tests, which are also equal to NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ (the material property), from K_o values shown in Fig. 5.2, and from the estimated SHANSEP OC q_f values in Fig. 5.24. At $\Delta q/\Delta q_f = 1/2$, based on RECOMPRESSION $\overline{CK_oUC}$ and $\overline{CK_oUE}$ sample behavior for both vertical and horizontal loading, OC A values are equal to those from NC soil.

8.5.2 Parameters for Finite Element Program FEECON

The finite element program FEECON was developed at MIT for plane strain analyses of stresses and deformations caused by construction of embankments on soft clay foundation. Briefly, FEECON is capable of:

1. Considering initial in situ shear stress;
2. Simulating the accumulation of material (e.g., construction of an embankment, excavation of material, and incremental loading);
3. Using hyperbolic and bilinear stress-strain models; and
4. Considering anisotropy in q_f using elliptical relationships (Davis and Christian, 1971) for local yielding.

The anisotropic stress-strain behavior and pore pressure response can be taken into account by dividing the finite element grid into three zones: vertical, inclined, and horizontal. In each zone, the soil is assumed to be a piece-wise linear isotropic material. Therefore, three sets of soil parameters for stress-

strain and pore pressure are needed.

Table 8.2 lists the required soil parameters for the finite element program FEECON. They are the initial undrained modulus (G_i), the factor relating the actual undrained shear strength (q_f) to the strength computed at infinite strain (R_f), a/b ratio, K_s and the pore pressure parameter a.

The parameters R_f and G_i for the hyperbolic stress-strain model are obtained from the relationship given below:

For triaxial and plane strain tests:

$$\Delta q/\bar{\sigma}_{vc} = \frac{\gamma}{1/(G_i/\bar{\sigma}_{vc}) + \frac{R_f \gamma}{(q_f/\bar{\sigma}_{vc})}} \qquad \text{Eq. 8.4}$$

$$\Delta q = 0.5(\Delta\sigma_1 - \Delta\sigma_3)$$

$$\Delta q_f = 0.5(\Delta\sigma_{1f} - \Delta\sigma_{3f})$$

$\gamma = 1.5\,\varepsilon_{(axial)}$ for triaxial loading

and

$\gamma = 2\,\varepsilon_{(vertical)}$ for plane strain loading.

For direct simple shear tests:

$$\tau_h/\bar{\sigma}_{vc} = \frac{\gamma}{1/(G_i/\bar{\sigma}_{vc}) + \frac{R_f \gamma}{\left(\frac{\tau_{h_{max}}}{\bar{\sigma}_{vc}}\right)}} \qquad \text{Eq. 8.5}$$

Recommended values of $G_i/\bar{\sigma}_{vc}$ and R_f are shown in Fig. 8.9 along with the test data used to develop these recommendations.

The recommended values of K_o and $q_f(V)$ are shown in Figs. 5.2 and 8.10, respectively. The values of q_f for vertical loading and horizontal loading were taken from Fig. 5.24. Values of K_s and b/a ratio versus OCR for plane strain and triaxial

loading are presented in Fig. 8.11. q_f from $\overline{CK_oU}$DSS and the measured and estimated q_f values from plane strain vertical and horizontal loading were used to determine

$$b/a = \frac{q_f(45)}{\sqrt{q_f(V)\,q_f(H)}} \quad .$$

The recommended pore pressure parameter a versus OCR at applied shear stress ratios equal to 0.5 and 1.0 (Fig. 8.12) were determined from Skempton's A parameters (Fig. 8.8) with the equation given below (assuming $\Delta\sigma_2 = 0.5\ (\Delta\sigma_1 + \Delta\sigma_3)$).

$$a = \frac{(3A - 1.5)}{\sqrt{1.5}} \qquad \text{Eq. 8.6}$$

8.6 RECOMMENDED PROCEDURES FOR CHECKING CVVC NSP AND FOR DEVELOPING NORMALIZED SHEAR STRENGTH PARAMETERS FOR OTHER VARVED CLAYS

The objective of this section is to recommend suitable testing procedures for checking the design soil parameters of Connecticut Valley varved clays and for developing anisotropic S_u parameters used in the design of structures on other varved clay deposits. Section 8.6.1 will present the recommended testing methods for checking CVVC soil parameters. If the NSP presented in Sections 8.2 through 8.5 cannot be used, suitable laboratory testing procedures for developing other design NSP, specifically the anisotropic S_u values, are presented in Section 8.6.2.

8.6.1 Recommended Method for Checking Soil Parameters

The soil parameters presented in Section 8.2 through 8.5 were developed to use in the design of structures on Connecticut Valley varved clays. They are based on test data from Northampton, Amherst, and East Windsor varved clays. Therefore, besides determining the stress history of the site, and the Atterberg limits of the clay, silt and bulk portions for comparison with values given in Table 3.7, the testing program should include the following addition tests:

1. FV tests for indicating spacial variations in strength. The FV data are used for three purposes:
   (a) to obtain S_u(FV) for TSA;
   (b) for use in conjunction with results from oedometer tests to check whether the measured values of normalized FV strength $[S_u(FV)/\bar{\sigma}_{vo}]$ at a given OCR agree with that in Fig. 8.4; and
   (c) if they agree, to use the relationship in Fig. 8.4 to help for obtaining a better defined stress history profile.
2. SHANSEP $\overline{CIUC}$ (θ=0) tests or $\overline{CK_oUC}$ tests and constant volume direct shear or $\overline{CK_oUDSS}$ tests at various OCR's because of the need to check the shear strength in two out of three directions.

8.6.2 Recommended Laboratory Testing Procedure for Developing Normalized Soil Parameters for Other Varved Clay Deposits

(A) Non-Cemented Soil

Based on the study presented in Chapters 4, 5, and 6,

and on the background used for developing the design normalized soil parameters presented in this chapter, the SHANSEP $\overline{CK_oUC}$ $\overline{CK_oUDSS}$, and $\overline{CK_oUE}$ tests are recommended for developing anisotropic S_u. It is believed that for tube samples, the SHANSEP method of consolidation should yield reasonable estimates of the NC and OC in situ q_f values, especially when the soil has a low OCR, though it causes the sample to behave in a less brittle and sensitive manner compared to the in situ soil.

The use of SHANSEP $\overline{CK_oUC}$ and $\overline{CK_oUE}$ tests for the evaluation of S_u for plane strain loading will yield conservative values. As a result, an adjustment to convert triaxial S_u to plane strain S_u values is recommended. This can be done by assuming that $S_u(V)/q_f$ from $\overline{CK_oUC}$ test and $S_u(H)/q_f$ from $\overline{CK_oUE}$ test ratios from the Connecticut Valley varved clay also hold for other deposits. That is, for a given mode of failure, the design:

$$S_u = \frac{q_f \text{ (measured from other deposits)} \times \text{design } S_u \text{ values for Conn. Valley}}{q_f \text{ (measured from Conn. Valley deposit)}}$$

q_f values from $\overline{CK_oUC}$ and $\overline{CK_oUE}$ tests for Connecticut Valley varved clays are presented in Fig. 4.14 and S_u values for PSC and PSE failure modes for Connecticut Valley varved clays are presented in Fig. 8.4. τ_{hmax} from the $\overline{CK_oUDSS}$ test can be used directly as S_u value for the DSS mode.

Though conventionally used in practice, UUC tests or RE-COMPRESSION ($\dot{\bar{\sigma}}_{vc} = \bar{\sigma}_{vo}$) $\overline{CIUC}$ tests on samples with different inclinations are not recommended because of disturbance due to separation of varves and the fact that the principal planes are not rotated during shear (i.e., the stress system anisotropy

component is not included in the measurement). SHANSEP $\overline{CIUC}$ tests on smaples with different inclinations are also not recommended partly because they neglect the importance of stres system induced components of anisotropy. $q_f(H)$ and $q_f(45)$ from these tests are too large compared to those from $\overline{CK_oU}$PSE and $\overline{CK_oU}$DSS tests, though $q_f(V)$ from $\overline{CIUC}$ (θ=0) tests will probably be only slightly lower than that from $\overline{CK_oU}$PSC tests.

(B) Naturally Cemented Soil

The RECOMPRESSION method of consolidation ($\bar{\sigma}_{vc} = \bar{\sigma}_{vo}$) is recommended with the types of tests similar to those for the non-cemented soil.

TYPES OF STABILITY ANALYSES AND RECOMMENDATIONS FOR EMBANKMENTS ON CONNECTICUT VALLEY VARVED CLAYS (AFTER LADD, 1975)

Class of Stability Problem	Type of Stability Analysis	Strength Parameters and Pore Pressure	Remarks and Recommendations
I Undrained	Total Stress Analysis	Initial in situ S_u: See Section 8.4 (pore pressure (u) not considered).	Commonly accepted method[1] Recommended with $S_u = \mu \times S_u$(field vane)
	Effective[2] Stress Analysis	$\bar{c}$ and $\bar{\phi}$ from $\overline{CK_oU}$ tests. Required prediction of pore pressure (u_u)	Not recommended since far more complex and probably unreliable.
II Drained	Total Stress Analysis	S_u for consolidated clay (u not considered).	Seldom used and complex. Only justified if low FS with ESA.
	Effective[2] Stress Analysis	$\bar{c}$ and $\bar{\phi}$ from CD tests (Section 8.5) $u = u_d$ (equilibrium pore pressure).	Commonly accepted method. Recommended approach.
III Partially Drained	Total Stress Analysis	S_u = for partially consolidated clay. See Section 8.4.2 (u not considered).	Recommended for design purposes using SHANSEP.
	Effective[2] Stress Analysis	$\bar{c}$ and $\bar{\phi}$ from $\overline{CK_oU}$ tests. $u = u_m$ (measured pore pressures) or $u \cong u_u$ (required stress analysis with A_f).	Only recommended for use during construction using measured pore pressure

Notes: (1) Check with SHANSEP S_u for low factor of safety.
(2) See Ladd (1975) for recommendation.

Table 8.1

INPUT PARAMETERS FOR CONNECTICUT VALLEY VARVED CLAYS FOR UNDRAINED DEFORMATION AND PORE PRESSURE ANALYSES USING FEECON

Item	Parameters	Recommended Values
Initial Stresses	γ_b or $\bar{\sigma}_{vo}$	--
	K_o	K_o vs. OCR: Fig. 5.2
Anisotropic Undrained Shear Strength	$q_f(V)$ for PSC shear	$q_f(V)$ vs. OCR: Fig. 8.10
	$K_s = q_f(H)/q_f(V)$	K_s vs. OCR: Fig. 8.11 (b)
	a/b = axes of ellipse	a/b = 1/b/a vs. OCR: Fig. 8.11
Hyperbolic Stress-Strain and Bulk Modulus	G_i and R_f for vertical, inclined and horizontal loadings.	$G_i/\bar{\sigma}_{vc}$ and R_f vs. OCR: Fig. 8.9
	G_y = yielded modulus	$G_y = 0.005\ G_i$
	K = bulk modulus	$K = 25\ G_i$
Undrained Pore Pressure	$U = \Delta\sigma_{oct} + a\ \Delta\tau_{oct}$ for vertical and horizontal loading.	a vs. OCR: Fig. 8.12

Note: Using $K = 25\ G_i$ and $G_y = 0.005\ G_i$ corresponds to $\nu_i = 0.48$ and $\nu_y = 0.4990$.

Table 8.2

SOIL PARAMETERS FOR THE ESTIMATION OF INITIAL SETTLEMENT (D'APPOLONIA ET AL., 1971) AND PORE PRESSURE

Item	Soil Parameter
Initial Settlement: $\rho_i = \frac{\Delta\sigma_v \, B \, I_p}{E_{(field)} S_R}$ at ℄ of loaded area	$E_{u(field)}$ (Fig. 8.5) f (initial shear stress ratio) versus OCR: (Fig. 8.6) S_R: (Fig. 8.7)
Pore Pressure: (B = 1.0) $u = \Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)$	Pore Pressure Parameter A at $\Delta q/\Delta q_f$ = 0.5 and 1.0 (Fig. 8.8)

Table 8.3

DETERMINATION OF S_u AT OCR = 1.0 FROM PLANE STRAIN LOADING USING STRAIN COMPATIBILITY METHOD (AFTER LADD, 1975)

Avg. Normalized Peak Shear Strength $= 1/3\ (q_f(V)\cos\bar{\phi} + \tau_{h_{max}} + q_f(H)\cos\bar{\phi})$

$= 1/3\ (0.28 \times 0.92 + 0.16 + 0.25 \times 0.90)$

$= 1/3\,(0.258 + 0.16 + 0.225)$

$= 0.214$

ε (%)	γ (%)	PSC $q(V)\cos\bar{\phi}$	DSS τ_h	PSE $q(H)\cos\bar{\phi}$	Avg.	% of Avg. Peak Shear Strength
1.0	2	0.248	0.150	0.153	0.184	86
2.0	4	0.235	0.160	0.194	0.196	91
3.0	6	0.221	0.160	0.212	0.198	92 (max)
4.0	8	0.202	0.160	0.225	0.196	91

From above results.

SCF			Avg. SCF
PSC	DSS	PSE	
0.857	1.0	0.942	0.935

Notes:

(1) $\cos\bar{\phi} = 0.92$ for PSC, and $\cos\bar{\phi} = 0.90$ for PSE

(2) $SCF = \dfrac{q\cos\bar{\phi} \text{ at max \% of Avg. Peak Shear Strength}}{q_f\cos\bar{\phi}}$

(3) Avg. SCF = 1/3 (SCF(PSC) + SCF(DSS) + SCF(PSE))

(4) % of Avg. Peak Shear Strength =

$$\frac{1/3\,(q(V)\cos\bar{\phi} + \tau_h + q(H)\cos\bar{\phi})}{\text{Ave. normalized peak shear strength}}$$

(5) Stress-strain data are presented in Chapter 5.

Table 8.4

DETERMINATION OF S_u AT OCR = 1.0 FROM TRIAXIAL LOADING USING STRAIN COMPATIBILITY METHOD

ε %	γ %	$\overline{CK_oUC}$ $q(V)\cos\bar{\phi}$ (1)	$\overline{CK_oUDSS}$ τ_h	$\overline{CK_oUE}$ $q(H)\cos\bar{\phi}$ (1)	Avg.	% of Avg. Peak Shear Strength
4.00	6	0.221	0.160	0.126	0.169	88
5.33	8	0.212 (2)	0.160 (2)	0.171 (2)	0.181	94 (max)
11.33	17	0.193	0.150	0.189	0.177	92

Avg. normalized peak shear strength = $1/3(q_f(V)\cos\bar{\phi} + \tau_{hmax} + q_f(H)\cos\bar{\phi})$

$= 1/3\ (0.25x0.92+0.16+0.2x0.9)$

$= 0.193$

From above results:

SCF (3)			Avg. SCF (4)
$\overline{CK_oUC}$	$\overline{CK_oUDSS}$	$\overline{CK_oUE}$	
0.922	1.0	0.864	0.930

Notes: (1) $\cos\bar{\phi} = 0.92$ for $\overline{CK_oUC}$ and $\cos\bar{\phi} = 0.90$ for $\overline{CK_oUE}$ test.

(2) Equal to S_u (no adjustment for strain rate effect).

(3) $SCF = \dfrac{q\cos\bar{\phi} \text{ at max. \% of Avg. Peak Shear Strength}}{q_f\cos\bar{\phi}}$

(4) Stress-strain data are presented in Chapter 5.

Table 8.5

DETERMINATION[1] OF S_u BY USING STRAIN COMPATIBILITY METHOD FROM TRIAXIAL LOADING AT OCR = 2

At OCR = 2; avg. normalized peak shear strength

$$= 1/3\ (0.40 \times 0.90 + 0.28 + 0.38 \times 0.90)$$

$$= 1/3\ (0.360 + 0.28 + 0.342) = 0.327$$

ε %	γ %	Vertical $q(V)\cos\bar{\phi}$[2]	Inclined τ_h	Horizontal $q(H)\cos\bar{\phi}$[2]	Avg.	% of Avg. Peak Shear Strength
5.33	8	0.351	0.280	0.315	0.315	96
8.00	12	0.342[3]	0.280[1]	0.333[3]	0.318	97 (max)
10.00	15	0.333	0.280	0.342	0.318	97
12.00	18	0.324	0.270	0.342	0.312	95

Avg SCF = $1/3\ (\frac{0.342}{0.360} + \frac{0.28}{0.28} + \frac{0.318}{0.342}) = 0.96$

Notes: (1) Tabulated stress strain data from $\overline{CK_oUC}$, $\overline{CK_oUDSS}$ and $\overline{CK_oUE}$ are presented in Appendix D.

(2) $\cos\bar{\phi} = 0.9$ in all cases.

(3) Equal to S_u (no strain rate adjustment).

Table 8.6(a)

DETERMINATION[1] OF S_u BY USING STRAIN COMPATIBILITY METHOD FROM TRIAXIAL LOADING AT OCR = 4

Avg. normalized peak shear strength = 1/3 (0.66x0.9+0.46+0.64x0.9)

= 1/3 (0.594 + 0.46 + 0.576)

= 0.543

ε %	γ %	Vertical $q(V)\cos\bar{\phi}$[2]	Inclined τ_h	Horizontal $q(H)\cos\bar{\phi}$[2]	Avg.	% of Avg. Peak Shear Strength
8.00	12	0.594	0.450	0.531	0.525	97
10.67	16	0.594[3]	0.460[3]	0.538[3]	0.537	99 (max)
12.00	18	0.585	0.380	0.576	0.514	95

Avg. SCF = 1/3 $(\frac{0.594}{0.594} + \frac{0.46}{0.46} + \frac{0.558}{0.576})$ = 0.99 $\approx$ 1.00

Notes: (1) Tabulated stress strain data from $\overline{CK_oUC}$, $\overline{CK_oUDSS}$, and $\overline{CK_oUE}$ are presented in Appendix D.
(2) $\cos\bar{\phi} = 0.9$ in all cases.
(3) Equal to S_u (no strain rate adjustment).

Table 8.6(b)

DETERMINATION OF S_u BY THE AUTHOR'S APPROACH

OCR	PSC			DSS			PSE			$\frac{S_u(H)}{S_u(V)}$
	$q_f \cos\bar{\phi}$ (2)	SCF	S_u (1)	τ_h	SCF	S_u (1)	$q_f \cos\bar{\phi}$ (2)	SCF	S_u (1)	
1	0.258	0.86	0.210	0.160	1.0	0.15	0.225	0.94	0.20	1.05
2	0.423	0.94	0.38	0.280	1.0	0.27	0.396	0.95	0.36	1.05
4	0.864	1.0	0.65	0.460	1.0	0.44	0.648	1.00	0.62	1.05

Notes: (1) $S_u = q_f \cos\bar{\phi}$ x SCF x 0.95 (strain rate factor)

(2) $\cos\bar{\phi}$ = 0.92 for NC PSC mode and $\cos\bar{\phi}$ = 0.90 for NC PSE Mode. For OC soil, for both PSC and PSE mode $\cos\bar{\phi}$ = 0.90.

Table 8.7

COMPARISON OF S_u VALUES FROM LADD (1975), AUTHOR AND CORRECTED FIELD VANE STRENGTHS

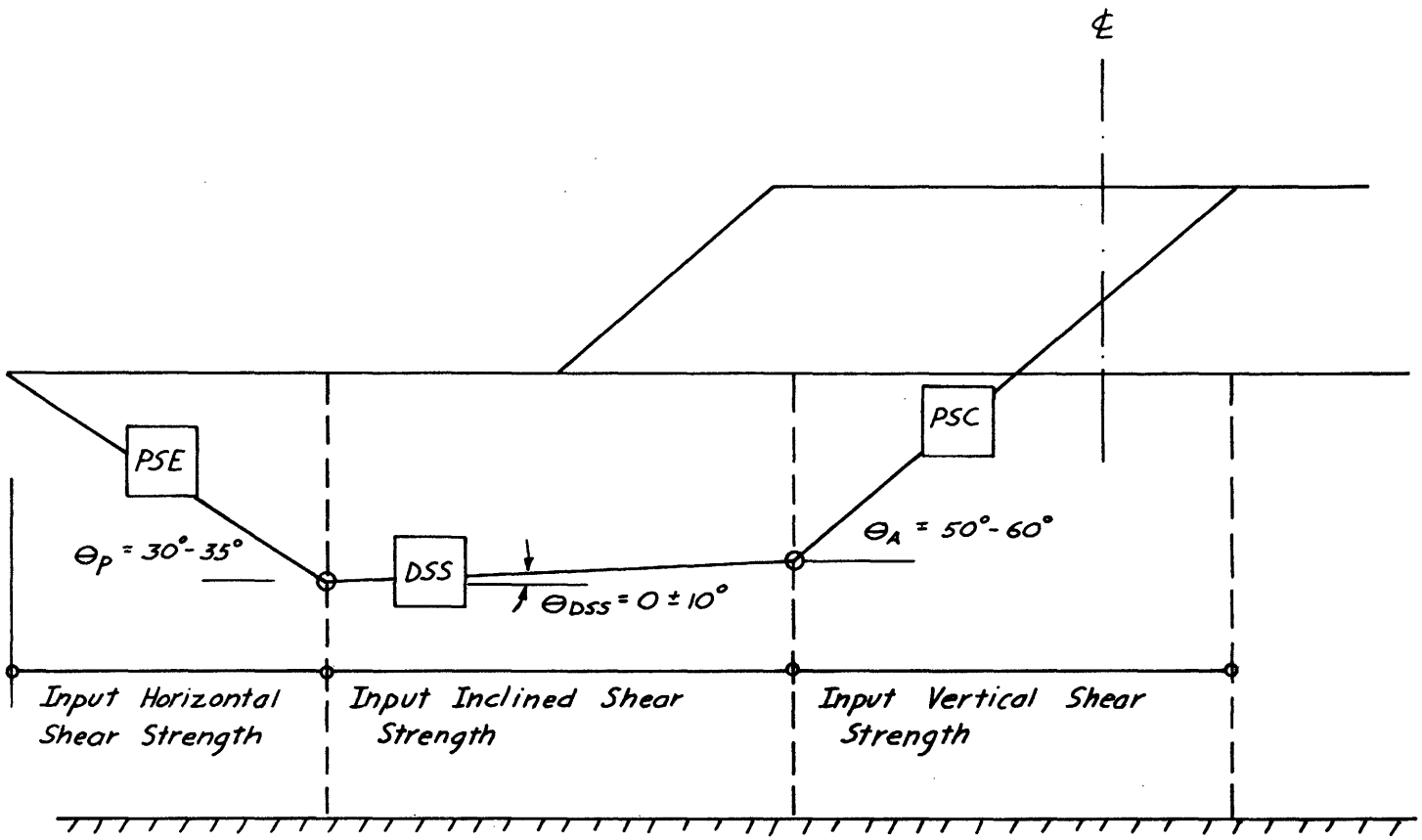
OCR	Ladd (1975)				Author				Corrected (1) FV Strengths
	S_u (PSC)	S_u (DSS)	S_u (PSE)	Avg.	S_u (PSC)	S_u (DSS)	S_u (PSE)	Avg.	
1	0.21	0.15	0.20	0.187	0.210	0.15	0.20	0.187	0.19
2	0.38	0.27	0.34	0.330	0.38	0.27	0.36	0.337	0.34
4	0.65	0.44	0.57	0.555	0.65	0.44	0.62	0.570	0.63

Note: (1) FV versus OCR from Ladd and Foott (1977); see Fig. 8.4.

μ = 0.85 for OC soil;

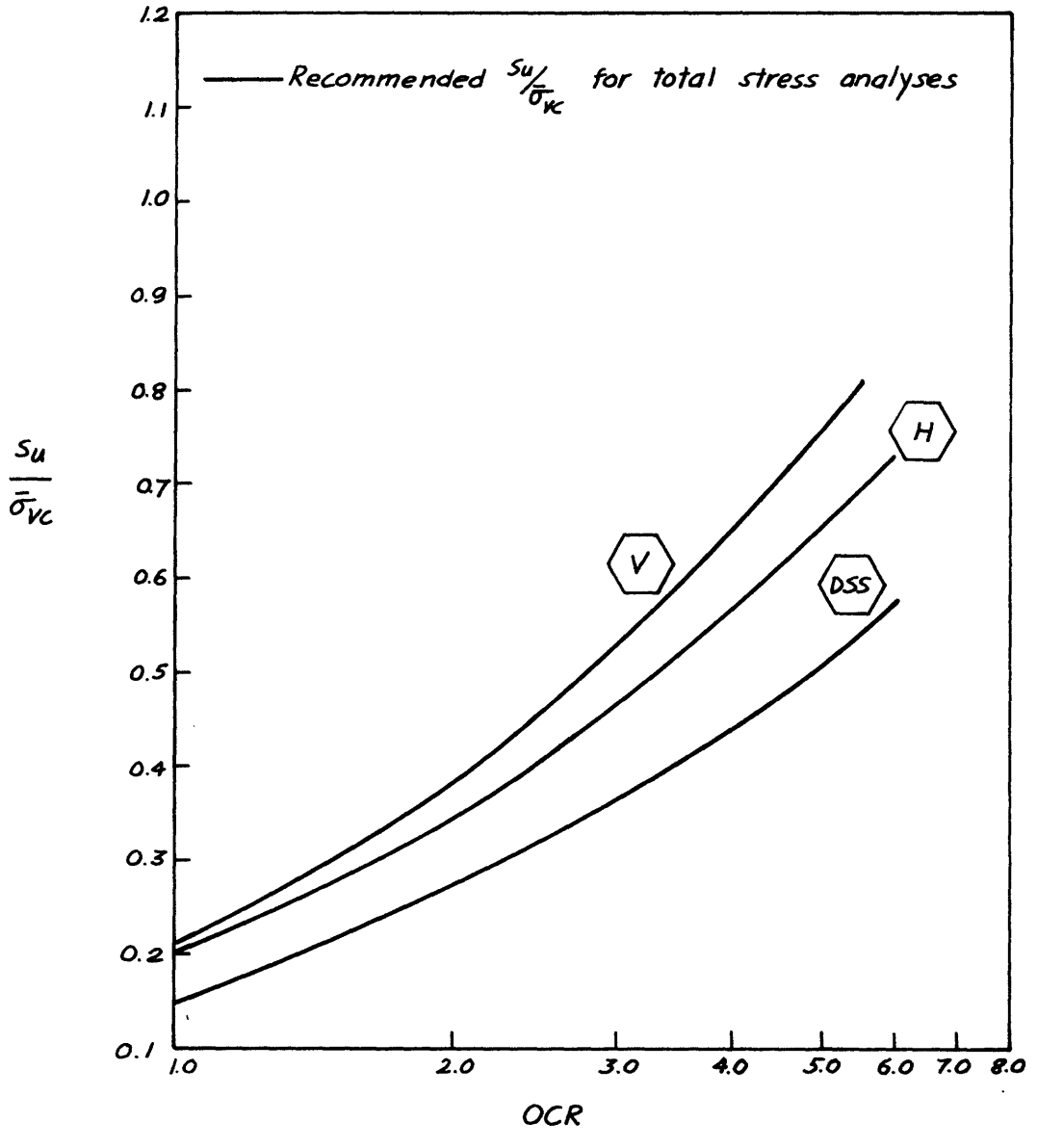
μ = 1.0 for NC soil.

Table 8.8



CONSIDERATION OF ANISOTROPY IN UNDRAINED SHEAR STRENGTH IN MORGENSTERN-PRICE ANALYSES (After Ladd, 1975)

FIGURE 8.1



RECOMMENDED $\frac{S_u}{\bar{\sigma}_{vc}}$ FOR TOTAL STRESS ANALYSES BY LADD (1975)

FIGURE 8.2

METHOD	UNDRAINED STRENGTH	MINIMUM F.S.
Bishop Circular Arc	Corrected FV	1.06
M-P	SHANSEP	1.00

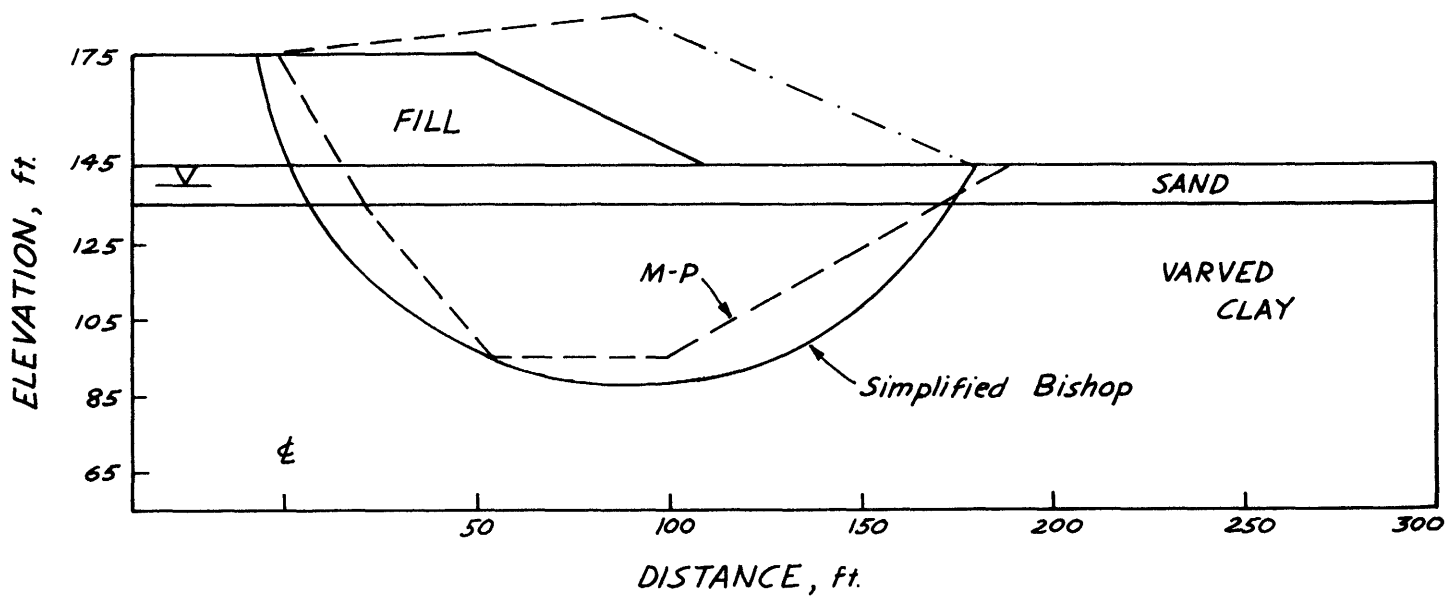
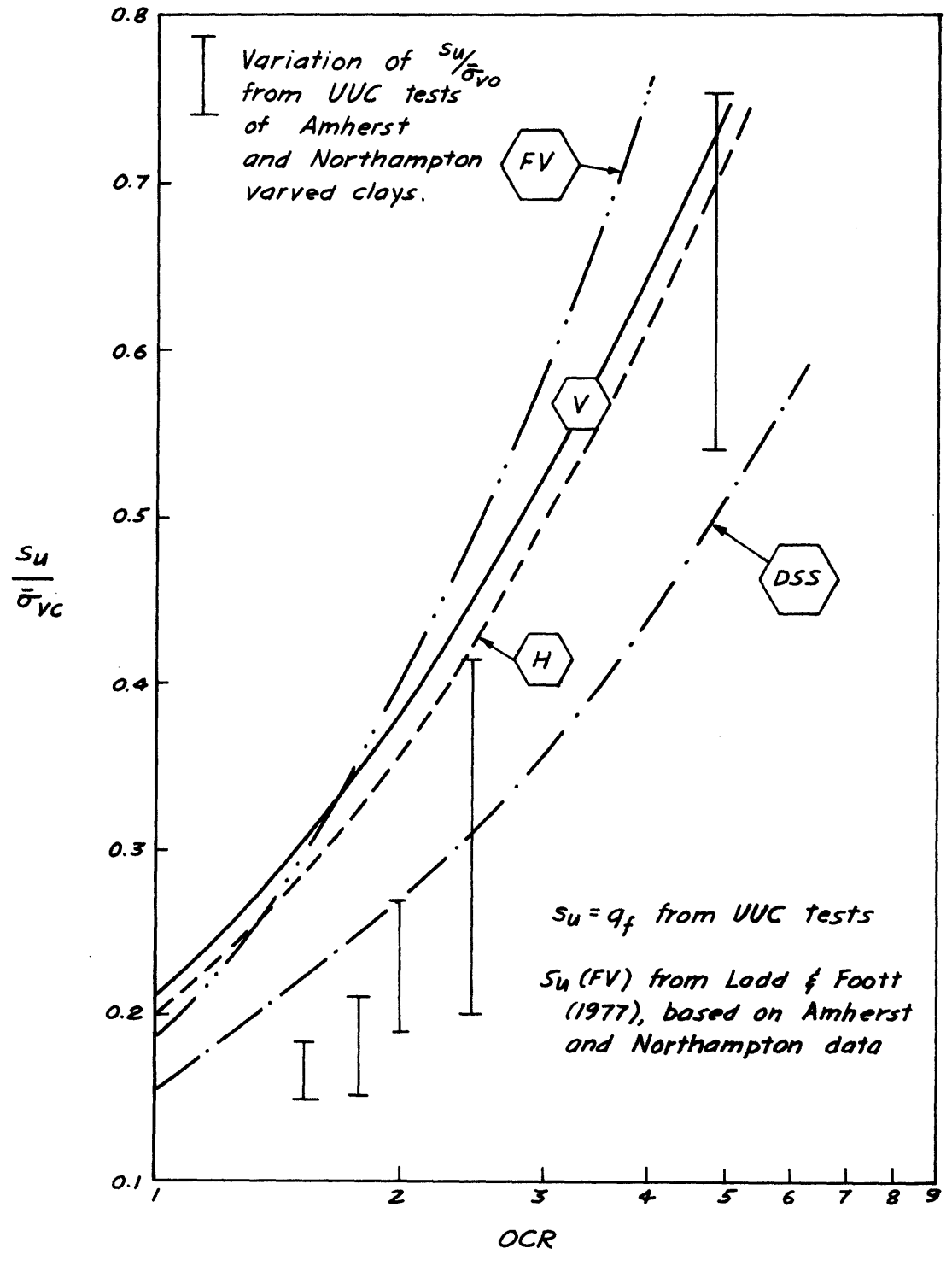
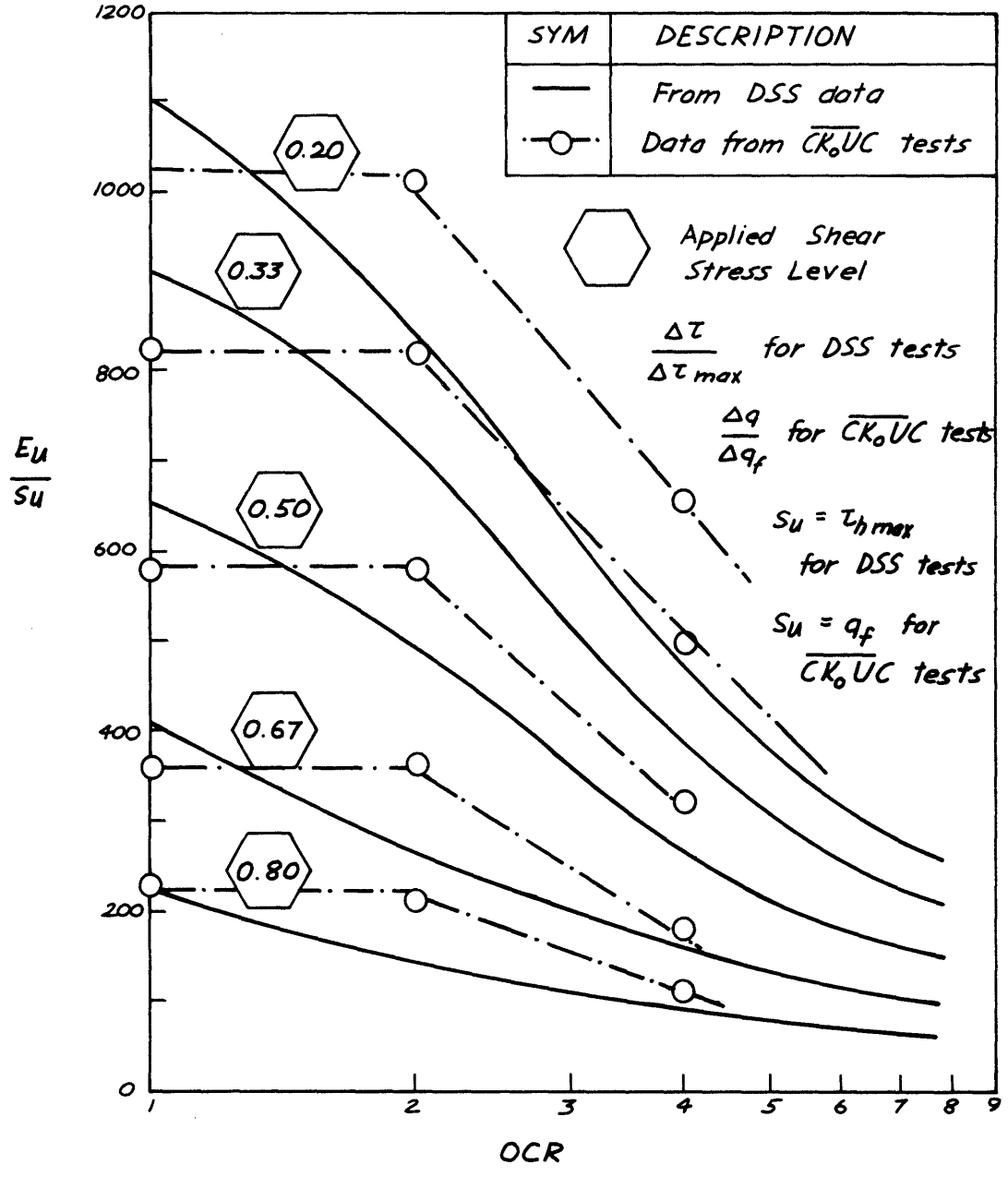


FIGURE 8.3 COMPARISON OF SIMPLIFIED BISHOP CIRCULAR ARC AND MORGENSTERN-PRICE ANALYSES (From Ladd & Foott, 1977)



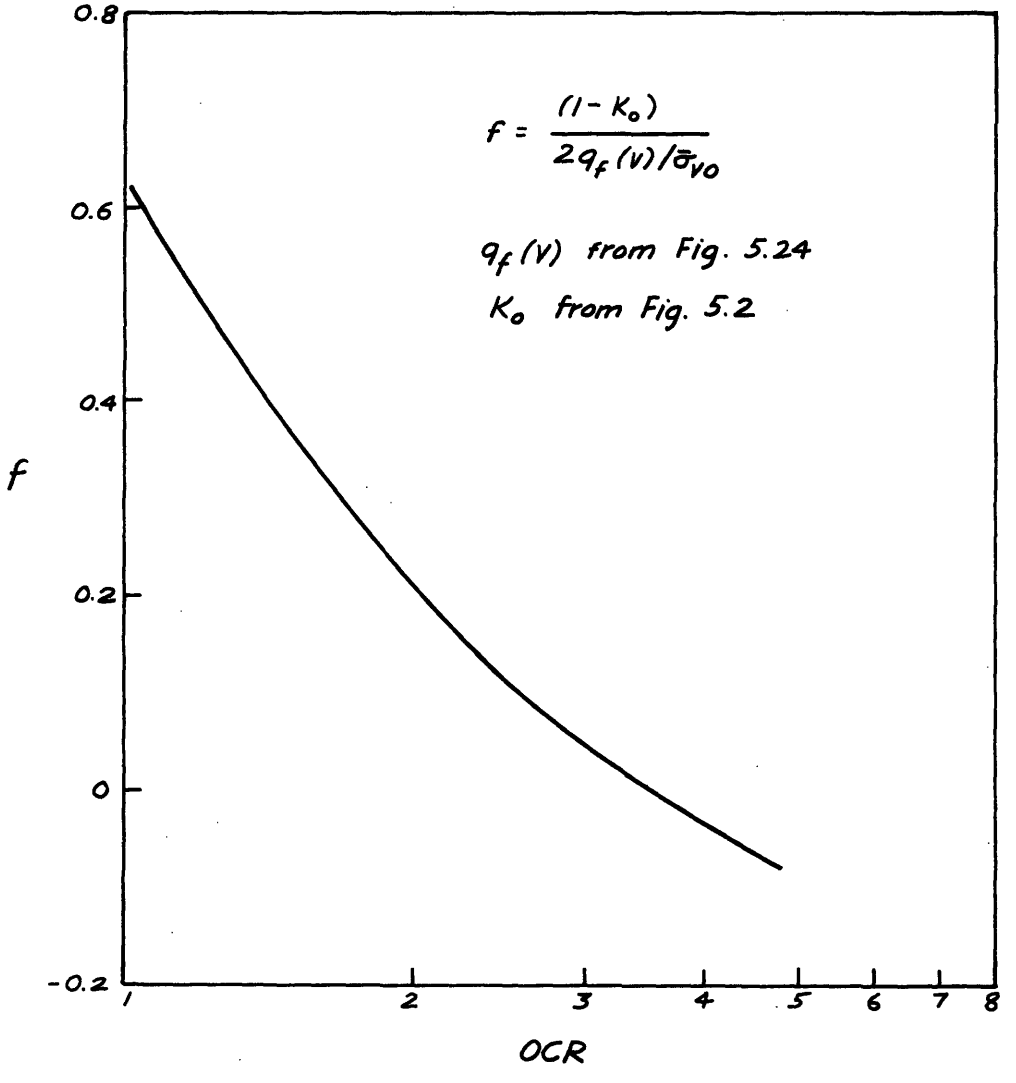
COMPARISON OF NORMALIZED UNDRAINED SHEAR STRENGTH DATA FROM UU_v, FV AND THESIS' RECOMMENDED UNDRAINED SHEAR STRENGTH PARAMETERS FOR DESIGN

FIGURE 8.4



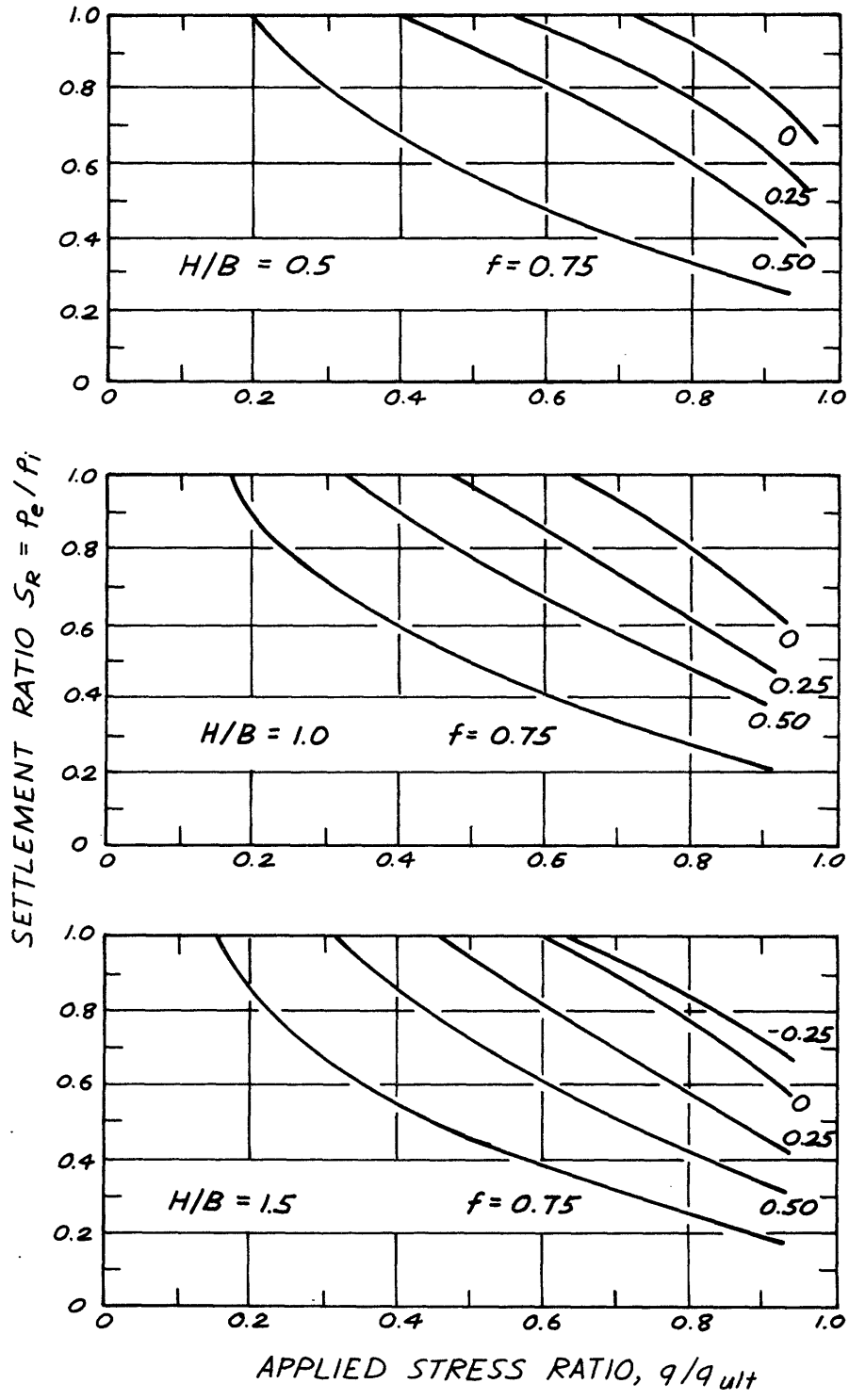
SHANSEP $\frac{E_u}{S_u}$ VERSUS OCR FOR CONNECTICUT VALLEY VARVED CLAYS

FIGURE 8.5



RECOMMENDED INITIAL STRESS RATIO VS OCR FOR CONNECTICUT VALLEY VARVED CLAYS FOR PLANE STRAIN LOADING

FIGURE 8.6



SETTLEMENT RATIO VS APPLIED STRESS RATIO FOR STRIP LOAD ON ISOTROPIC HOMOGENEOUS FOUNDATION (From D'Appolonia, Poulos and Ladd, 1971)

FIGURE 8.7

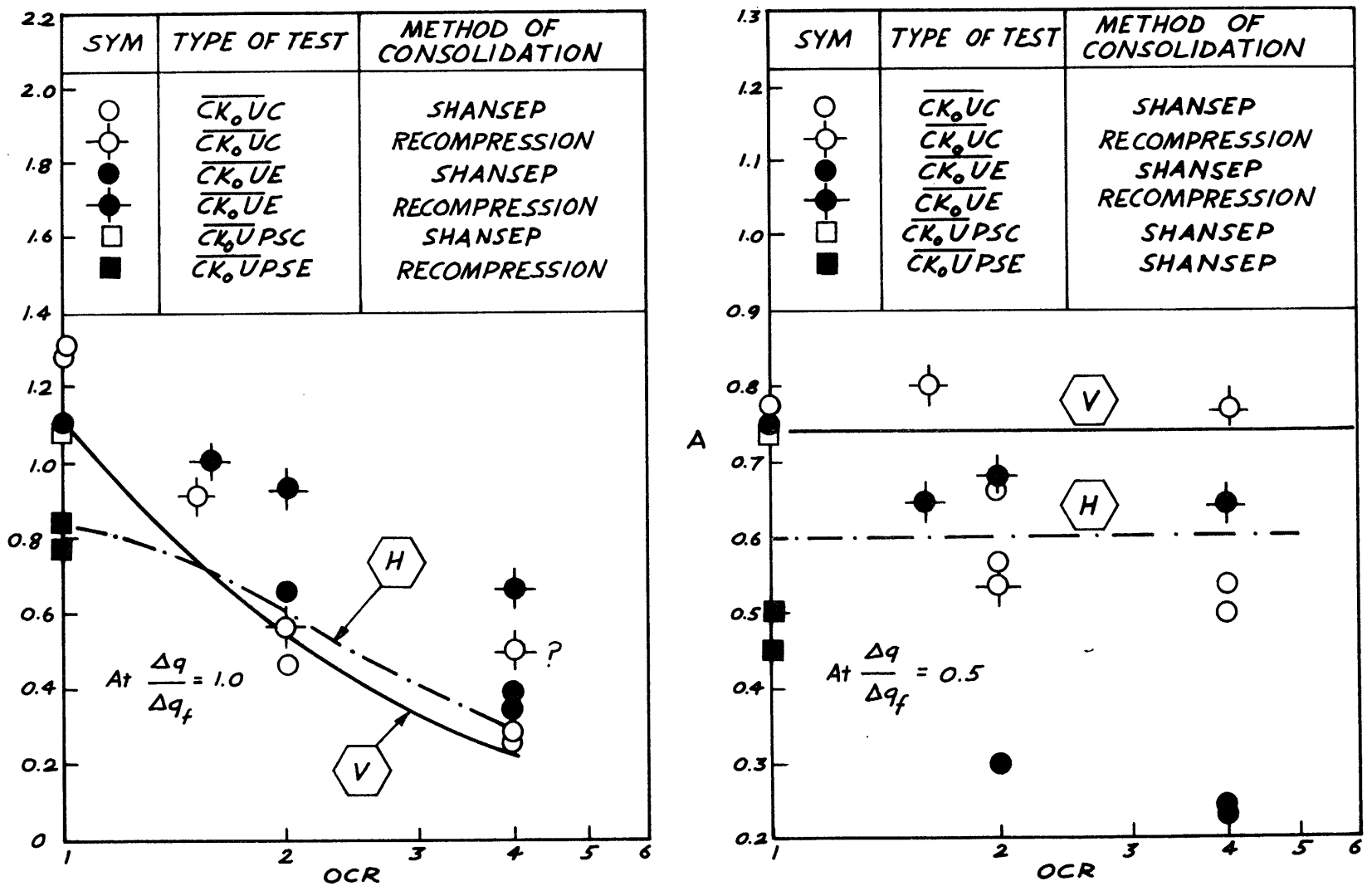
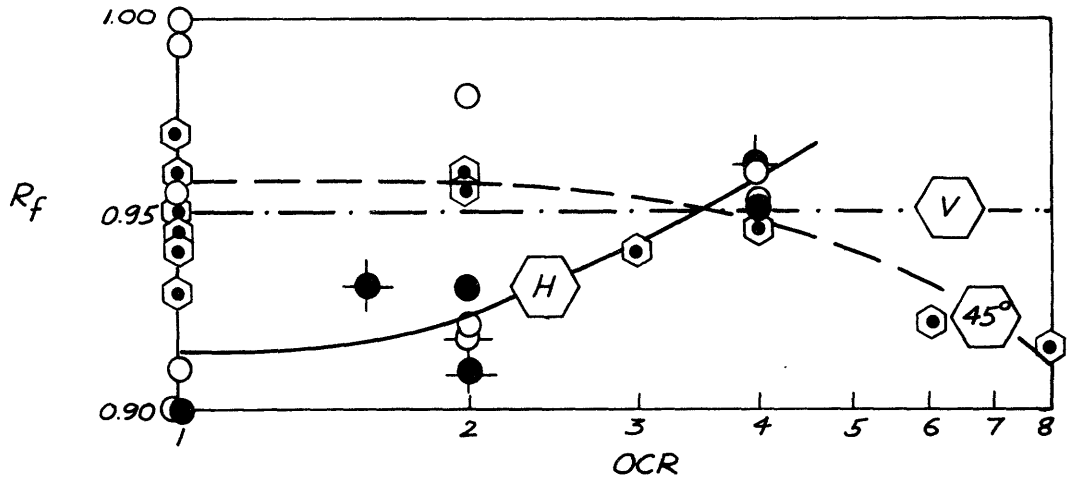
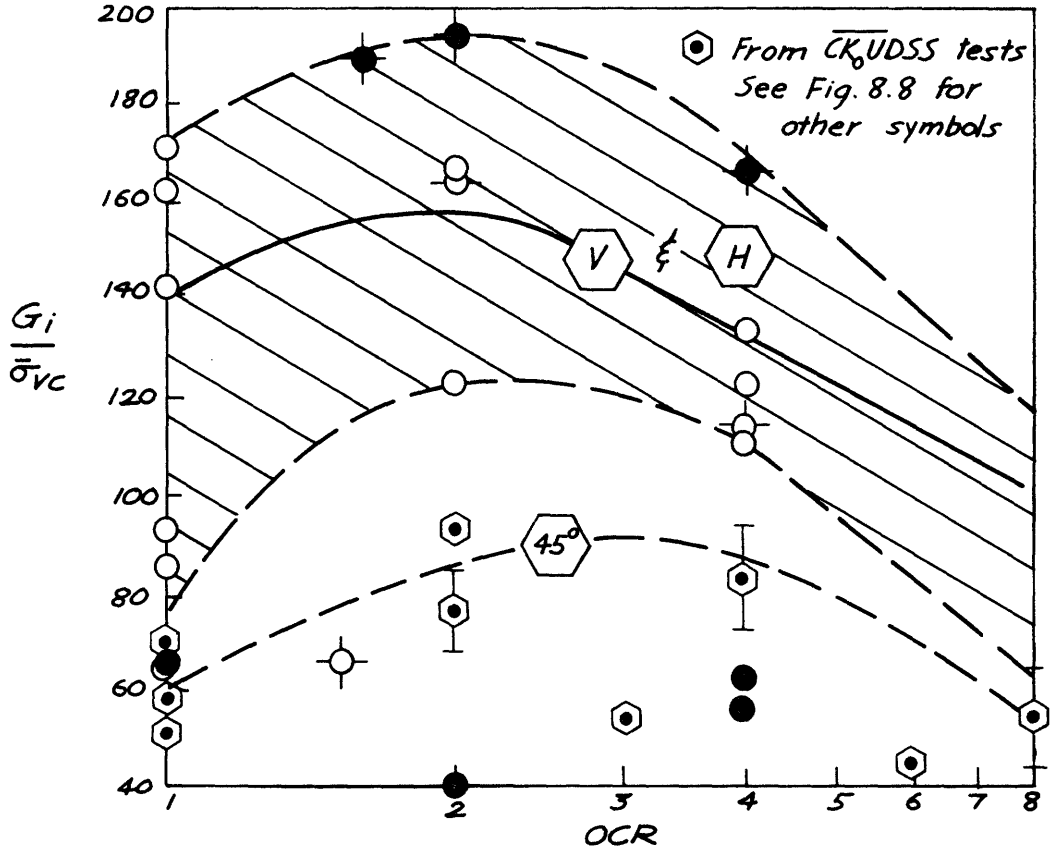


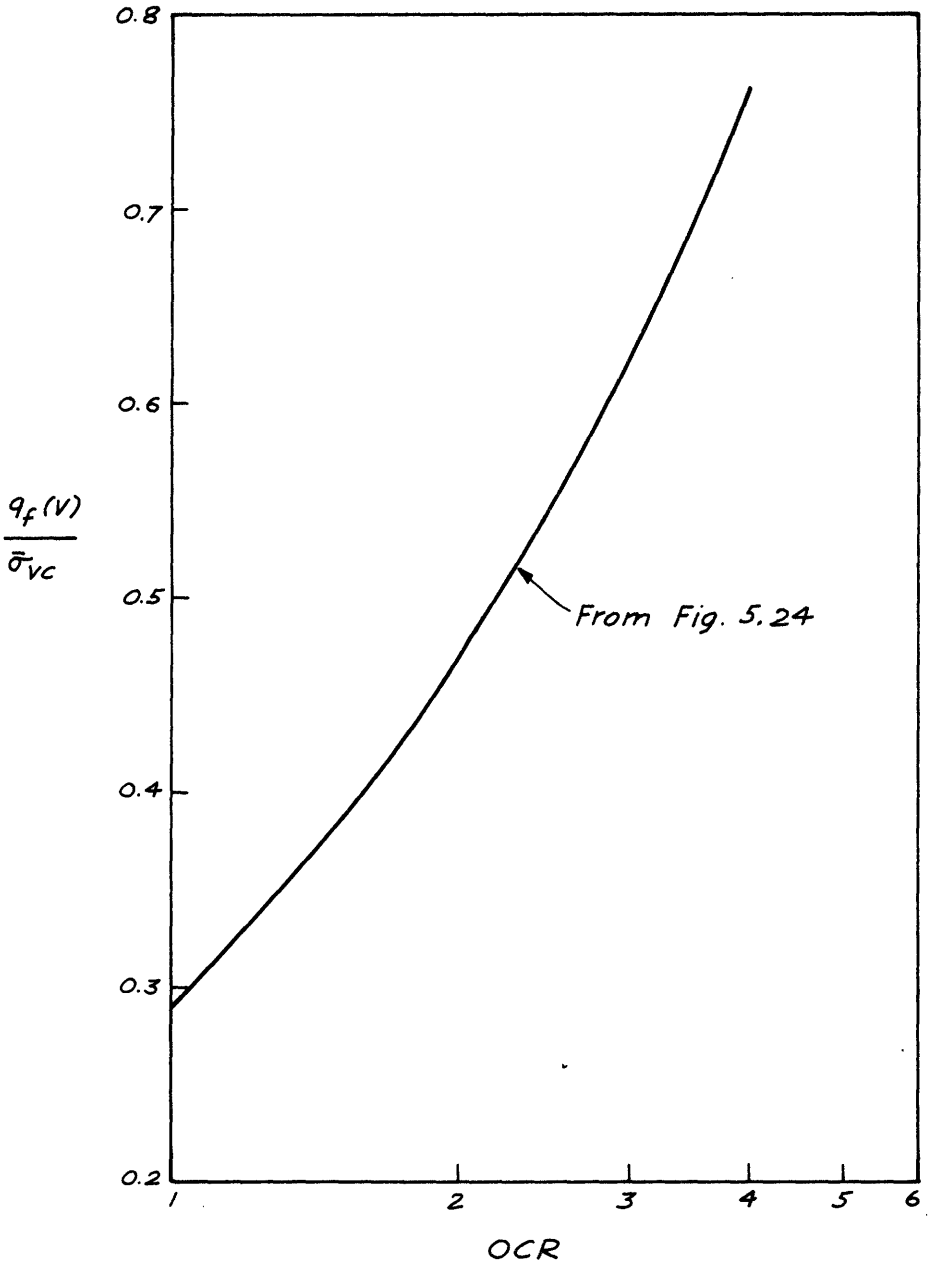
FIGURE 8.8 RECOMMENDED PORE PRESSURE PARAMETER A VS OCR FOR PREDICTIONS OF PORE PRESSURE IN CONN. VALLEY CLAYS FOR PLANE STRAIN LOADING





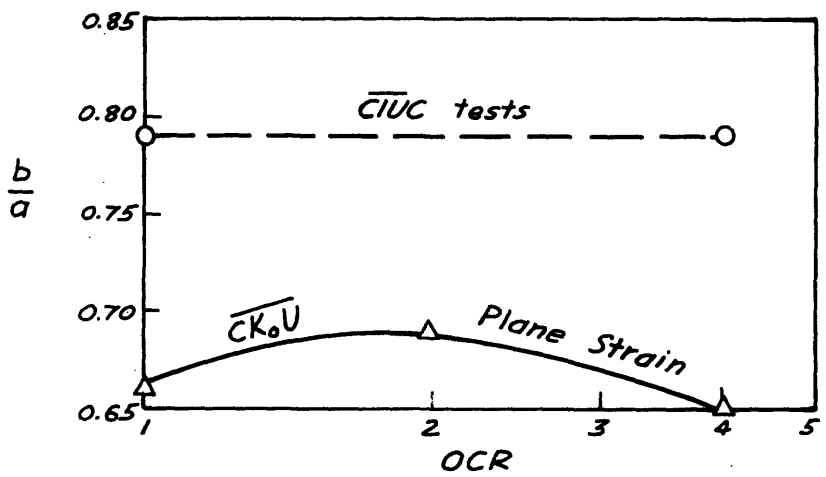
RECOMMENDED UNDRAINED HYPERBOLIC STRESS-STRAIN PARAMETERS FOR USE IN FINITE ELEMENT ANALYSES WITH CONNECTICUT VALLEY VARVED CLAYS

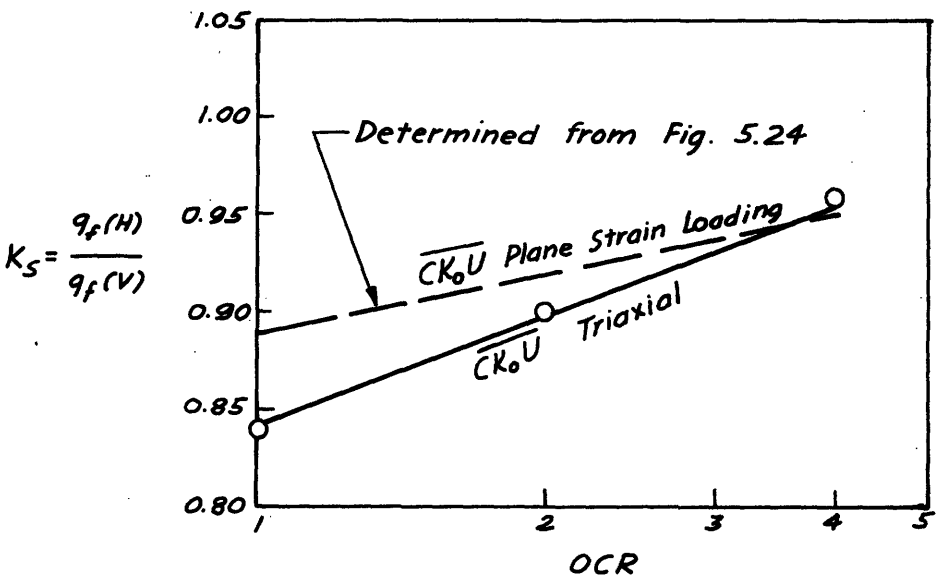
FIGURE 8.9



RECOMMENDED VERTICAL UNDRAINED STRENGTH FOR USE IN FINITE ELEMENT ANALYSES WITH CONN. VALLEY VARVED CLAYS IN PLANE STRAIN LOADING

FIGURE 8.10





RECOMMENDED ELLIPTICAL ANISOTROPIC UNDRAINED STRENGTH PARAMETERS FOR USE IN FINITE ELEMENT ANALYSES WITH CONNECTICUT VALLEY VARVED CLAYS

FIGURE 8.11

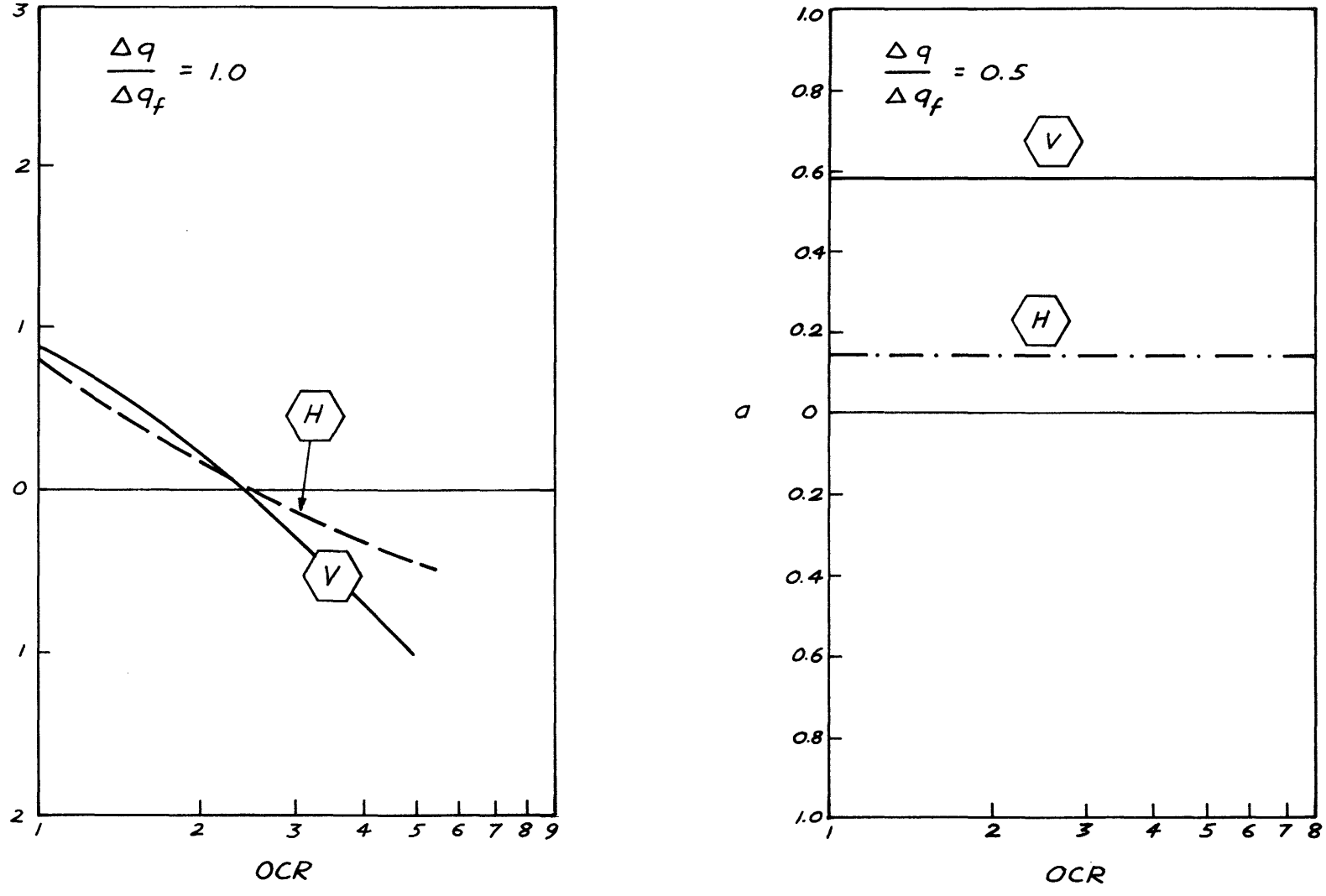


FIGURE 8.12 RECOMMENDED PORE PRESSURE PARAMETER a VS OCR FOR USE IN FINITE ELEMENT ANALYSES WITH CONNECTICUT VALLEY VARVED CLAYS

9 SUMMARY AND CONCLUSIONS

The objectives of this thesis are:

1. To contribute to a fundamental understanding of the nature of undrained shear strength anisotropy of varved clays;
2. To present best estimates of normalized stress-strain strength parameters for undrained total stress analyses and for predictions of pore pressures and undrained deformations used in the design of structures on Connecticut Valley varved clays, with background information showing how these parameters were developed; and
3. To recommend suitable testing procedures for developing normalized soil parameters for other varved clay deposits.

9.1 THE NATURE OF UNDRAINED SHEAR STRENGTH ANISOTROPY OF VARVED CLAYS

The factors that control the anisotropic strength behavior of varved clay are: the inherent anisotropy, which includes anisotropy due to the layered nature of varved clay (material anisotropy) and that due to the prior K_o consolidation (structural anisotropy); and when K_o is not equal to one, the stress system induced anisotropy. The latter results from the fact that different increments of shear stress are required to

produce failure as the major principal stress varies between the vertical and horizontal directions. The major result of stress system induced anisotropy is a change in the effective stress at failure as σ_1 varies in direction during shear.

The anisotropy in $q_f/\bar{\sigma}_{vm}$ of normally consolidated (NC) and overconsolidated (OC) varved clays results from anisotropy in the pore pressure response and in the normalized effective stress envelope* (NESE) at q_f. As shown in Figure 9.1, due to the material anisotropy component, $q_f(H)$ is larger than $q_f(V)$ and q_f from inclined loading is the smallest. With the additional effects of structure and stress system induced anisotropy, i.e., combined anisotropy, $q_f(V)$ is larger than $q_f(H)$ while q_f from inclined loading is still the smallest. (Note, however, that in Figure 9.1 only the effect of the structural anisotropy component is shown since K_o is equal to one). The fact that q_f is smallest in inclined loading is partly due to the material anisotropy component the failure envelope for inclined loading being the lowest because failure takes place within a clay layer (Curve D, Fig. 9.2) whereas for vertical and horizontal loading failure takes place across the varves. With combined anisotropy, i.e., total effects of inherent and stress induced anisotropy, $q_f(H)$ is slightly lower than $q_f(V)$ even though the NESE at q_f is much higher for horizontal loading (Curve A vs. Curve B' in Fig. 9.2). The smaller $q_f(H)$ compared to $q_f(V)$ results from significantly lower effective stresses at failure.

The anisotropy in NESE at q_f shown in Fig. 9.2 when it is

*The plot of $q/\bar{\sigma}_{vm}$ versus $\bar{p}/\bar{\sigma}_{vm}$.

not equal to that at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$, is due to both inherent anisotropy and to variation in the degree of mobilization of friction and cohesion during shear. Because of the latter, the NESE at q_f for vertical loading (Curve B', Fig. 9.2) is lower than that at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$, i.e., Curve A in Fig. 9.2, which holds for both vertical and horizontal loading.

The anisotropy in developed pore pressures is the result of anisotropy in the pore pressure parameter A due to inherent anisotropy (see Fig. 7.9) and differences in the magnitude of Δq due to the stress system induced component.

The anisotropy in NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ is found to be due to material anisotropy. Thus, there are three possible values of NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ for varved clay which depend only on the shearing direction., i.e., across the varves or parallel to the varves. These are: (1) for failure across the varves (Fig. 9.2, Curve A); (2) for failure within a silt layer ($\bar{\phi} \geq 24°$ based on CD direct shear tests); and (3) for failure within a clay layer (Figure 9.2, Curve E), the latter two occurring when shearing is parallel to the varves.

Based on several types of strength tests (see Chapter 5), the NESE at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ for failure across the varves (Curve A in Fig. 9.2 and Fig. 5.26) and for failure within a clay layer (Curve E in Fig. 9.2 and Fig. 5.31) are considered to be "basic material" properties since they were shown to be independent of:

1. The stress system during consolidation;
2. The stress system during shear;

3. The magnitude of $\bar{\sigma}_{vm}$;

4. The drainage conditions; and

5. The OCR of the samples.

9.2 DEVELOPMENT OF NORMALIZED STRESS-STRAIN-STRENGTH PARAMETERS FOR USE IN THE DESIGN OF STRUCTURES ON CONNECTICUT VALLEY VARVED CLAYS

Best estimates of normalized stress-strain-strength parameters for use in the design of structures on Connecticut Valley varved clays, and recommended testing methods for checking these design parameters, were presented in Chapter 8. The background for developing the normalized soil parameters for Connecticut Valley varved clays is as follows:

1. Anisotropic S_u parameters for plane strain loading--τ_{ff} at the q_f condition from SHANSEP K_o consolidated plane strain compression, direct simple shear, and plane strain extension ($\overline{CK_oU}$PSC, $\overline{CK_oU}$DSS, $\overline{CK_oU}$PSE) tests corrected for strain compatibility (Section 8.2.2 are recommended as design values (Fig. 9.3) for Morgenstern and Price undrained total stress analyses. The reasons for considering these values to be best estimates are:

   a. K_o consolidated plane strain and direct simple shear tests simulate representative in situ modes of fialure, i.e., the PSC, DSS, and PSE elements shown in Fig. 9.3.

   b. Values of $q_f/\bar{\sigma}_{vc}$ from several types of SHANSEP

strength tests (e.g., $\overline{CK_oUC}$, $\overline{CK_oUDSS}$, and $\overline{CK_oUE}$ tests) have been shown to be the same for three locations (Amherst, Northampton, and East Windsor) within the Connecticut Valley (e.g., Fig. 4.14).

c. On the basis of $\overline{CK_oU}$ triaxial data on block samples, the SHANSEP method of consolidation is thought to yield reasonable estimates of NC and OC in situ q_f values, especially at low OCR, though it causes more heavily OC samples to behave in a less brittle and sensitive manner compared to the in situ soil (see Fig. 9.4 and 6.5). Note that in Fig. 6.5 the lower q_f value from the RECOMPRESSION* $\overline{CK_oUE}$ OC (OCR=4 the in situ OCR) sample compared to that from the SHANSEP sample probably results mainly from disturbance due to separation of varves, though when σ_{1f} acts perpendicular to the varves or when the effect of disturbance due to separation of varves is small, it is thought that OC RECOMPRESSION data on block samples give the best indication of the in situ soil behavior.

d. In order to reflect the in situ behavior, and since the effective stress rather than total

*RECOMPRESSION--the method of consolidation whereby the sample is consolidated of the desired OCR with the vertical consolidation stress that is less than the in situ maximum past pressure.

stresses control the soil strength behavior, the undrained shear strength is defined as τ_{ff} rather than q_f.

NOTE: In Fig. 9.3, the S_u values for NC vertical and horizontal loading are based on plane strain compression (PSC) and plane strain extension (PSE) tests. However, the variation in strength with OCR was obtained from trends measured in triaxial compression and extension test (see Fig. 5.23 and Section 8.2.3).

2. The recommended values of pre-failure pore pressure parameter A (Fig. 8.8) were based upon SHANSEP plane strain loading tests for NC soil (since they are believed to yield better results than RECOMPRESSION tests) and on estimated RECOMPRESSION tests data for OC soil since RECOMPRESSION tests on block samples are thought to yield realistic pre-failure stress-strain pore pressure behavior for overconsolidated deposits (Figs. 9.4 and 6.5).

The other recommended normalized soil parameters plotted versus OCR are E_u/S_u (Fig. 8.5), the initial shear stress ratio, f, (Fig. 8.6) and hyperbolic stress-strain parameters (G_i and R_f (Fig. 8.9).

9.3 RECOMMENDED TESTING PROCEDURES FOR DEVELOPING NORMALIZED SOIL PARAMETERS FOR OTHER VARVED CLAY DEPOSITS

For undrained total stress stability analyses, SHANSEP K_o

consolidated undrained triaxial compression, direct simple shear and triaxial extension ($\overline{CK_oU}C$, $\overline{CK_oU}DSS$ and $\overline{CK_oU}E$) tests were recommended for obtaining anisotropic plane strain S_u values, with an adjustment to convert triaxial S_u to plane strain S_u values. (Note that use of triaxial tests will lead to lower strength thus yielding somewhat conservative results for field situations involving plane strain loading (see Section 8.3.3)). The adjustment can be done by assuming that the ratios $S_u(V)/q_f$ from $\overline{CK_oU}C$ tests and $S_u(H)/q_f$ from $\overline{CK_oU}E$ tests for the Connecticut Valley varved clay also hold for other deposits. That is, for a given mode of failure (PSC or PSE), the design:

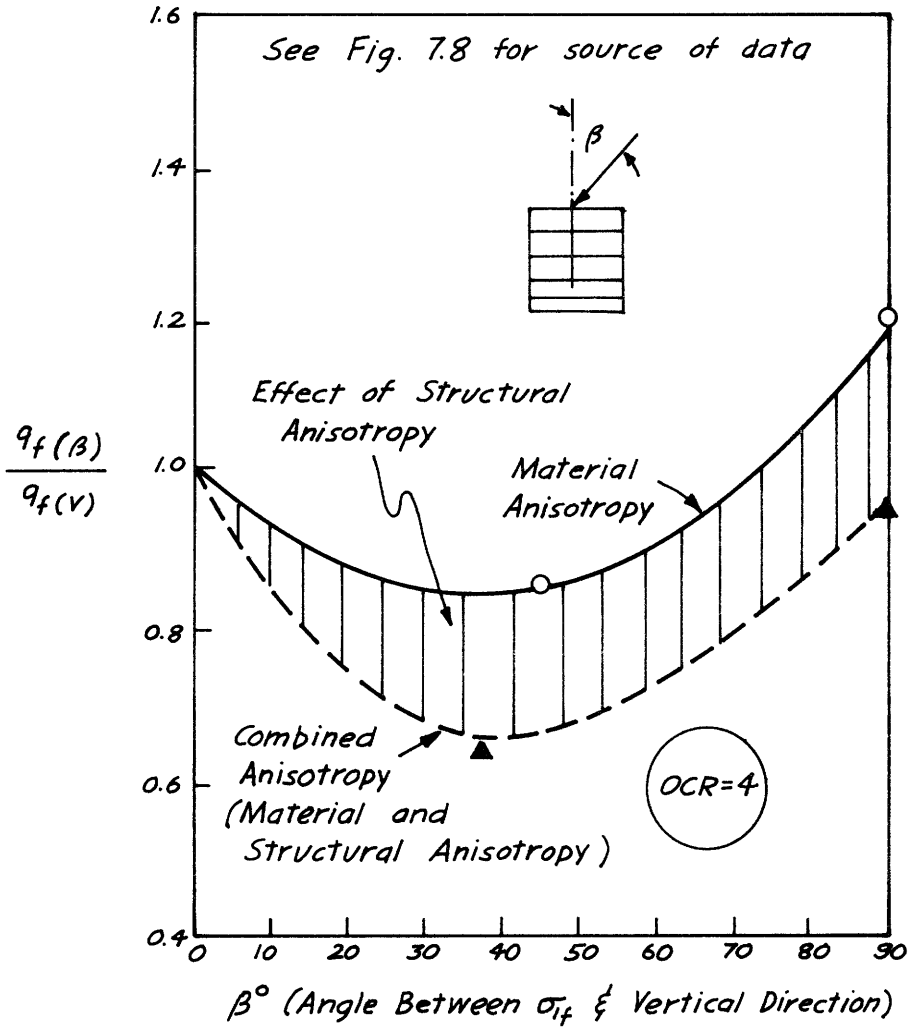
$$S_u = \frac{\text{triaxial } q_f \text{ (measured for other deposit)} \times \text{ design } S_u \text{ value for Conn. Valley}}{\text{triaxial } q_f \text{ (measured from Conn. Valley deposit)}}$$

S_u values for PSC and PSE failure modes for Connecticut Valley varved clays are presented in Fig. 9.3. q_f values from $\overline{CK_oU}C$ and $\overline{CK_oU}E$ tests on Connecticut Valley varved clays are presented in Fig. 4.14. τ_{hmax} from the $\overline{CK_oU}DSS$ test can be used directly as S_u value for DSS mode.

Due to distrubance from structural damage, especially due to separation of varves, UUC tests and RECOMPRESSION $\overline{CIU}C$ tests on samples with different inclinations are not recommended (see Chapter 6). SHANSEP $\overline{CIU}C$ tests on samples with different inclinations are not recommended because they yield unconservative q_f values for plane strain horizontal and inclined loadings (see Chapter 5).

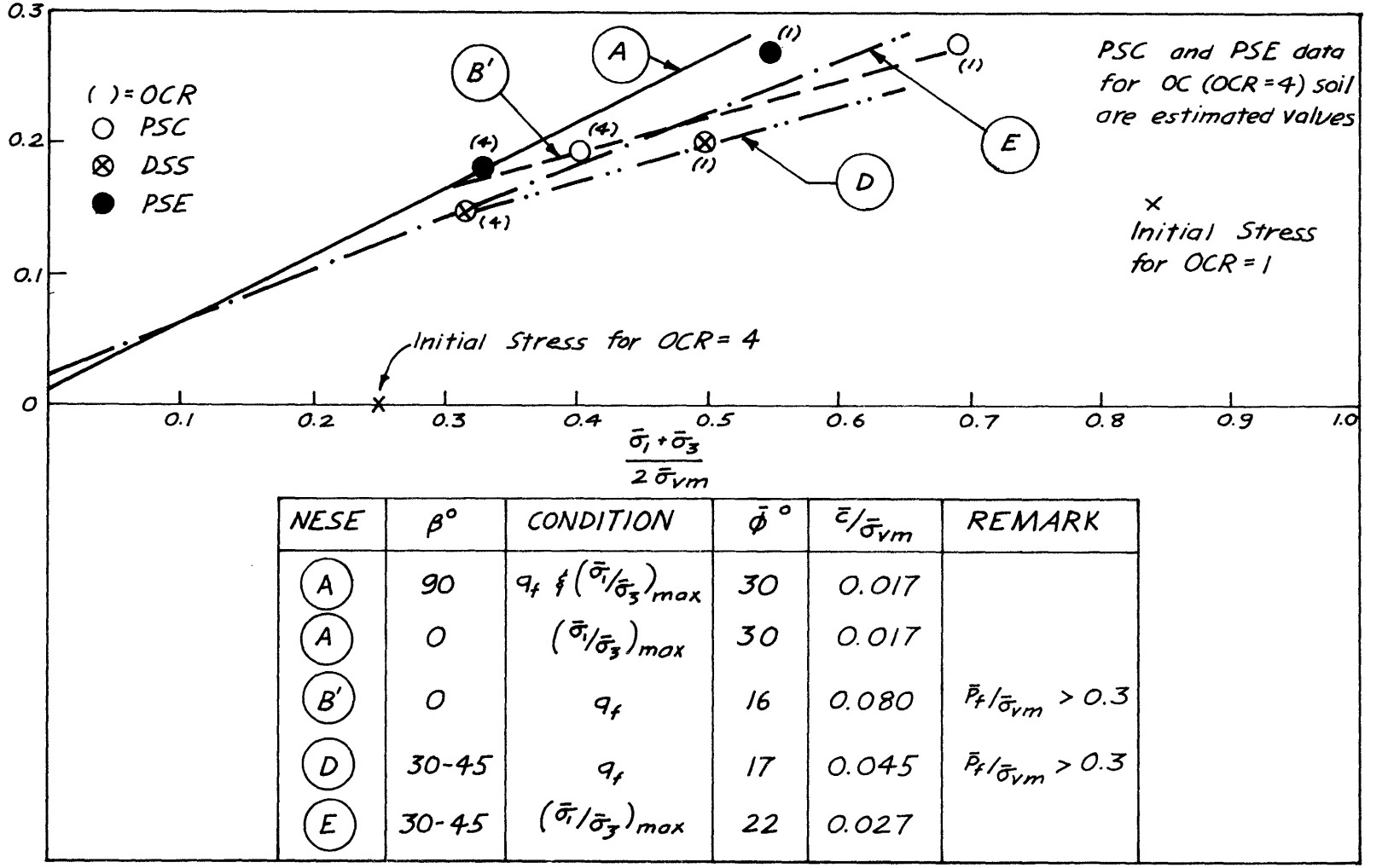
In cases where predictions of pore pressures and deformations

are important, in OC deposits, RECOMPRESSION $\overline{CK_oU}C$, $\overline{CK_oU}$DSS and $\overline{CK_oU}$E tests are recommended, rather than the SHANSEP method if the samples are of good quality.



MATERIAL AND COMBINED ANISOTROPY IN q_f OF CONNECTICUT VALLEY VARVED CLAYS AT OCR = 4

FIGURE 9.1

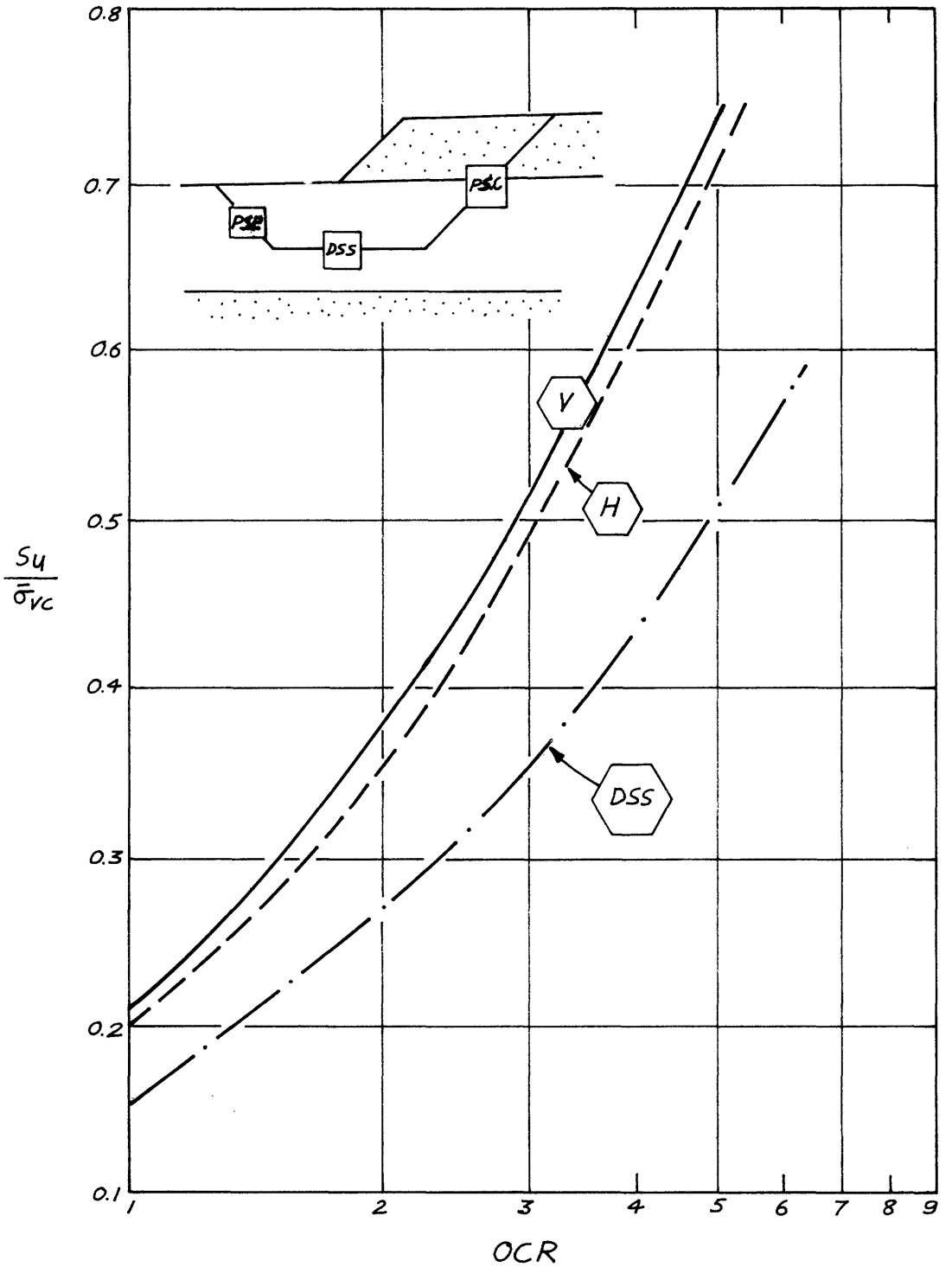


$\frac{\sigma_1 - \sigma_3}{2\bar{\sigma}_{vm}}$

$\frac{\bar{\sigma}_1 + \bar{\sigma}_3}{2\bar{\sigma}_{vm}}$

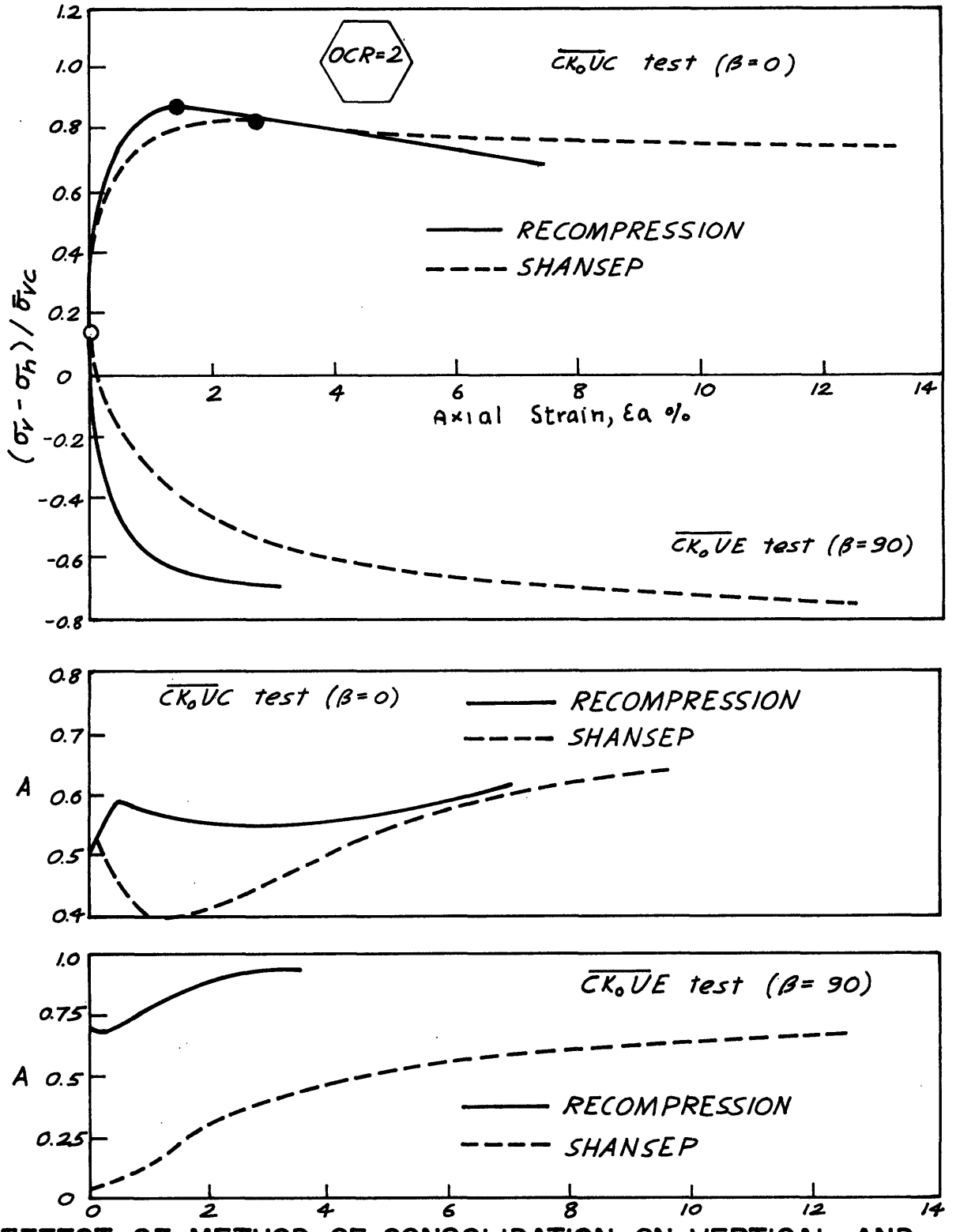
NESE	β°	CONDITION	$\bar{\phi}^\circ$	$\bar{c}/\bar{\sigma}_{vm}$	REMARK
A	90	q_f & $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$	30	0.017	
A	0	$(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$	30	0.017	
B'	0	q_f	16	0.080	$\bar{P}_f/\bar{\sigma}_{vm} > 0.3$
D	30-45	q_f	17	0.045	$\bar{P}_f/\bar{\sigma}_{vm} > 0.3$
E	30-45	$(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$	22	0.027	

FIGURE 9.2 COMBINED ANISOTROPY IN q_f AND IN NESE AT q_f AND AT $(\frac{\bar{\sigma}_1}{\bar{\sigma}_3})_{max}$



RECOMMENDED ANISOTROPIC UNDRAINED STRENGTH PARAMETERS FOR USE IN TOTAL STRESS MORGENSTERN-PRICE STABILITY ANALYSES WITH CONNECTICUT VALLEY VARVED CLAYS

FIGURE 9.3



EFFECT OF METHOD OF CONSOLIDATION ON VERTICAL AND HORIZONTAL σ-ε-STRENGTH BEHAVIOR FROM $\overline{CK_oUC}$ AND $\overline{CK_oUE}$ TESTS ON CONNECTICUT VALLEY VARVED CLAYS AT OCR = 2

FIGURE 9.4

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. EIC-1-3.
SOIL EAST WINDSOR VARVED CLAY
PROJECT 80142
TESTED BY SAM. DATE 1 Sept. 1975
ALL STRESSES IN kg/cm².

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL	56.0			79.19	7.97	9.93
PRESHEAR	—			71.05	9.64	9.29

G_s = TYPE CELL GEO.

PRESHEAR: $\bar{\sigma}_{1c}$ = 1.00, t_c = 1 day; $\bar{\sigma}_{3c}$ = 1.00, P.P.R. = — %; $\bar{\sigma}_{ac}$ = 1.00, u_B = 2.00

DURING SHEAR: CONTROLLED STRAIN ✓ STRESS ___; RATE 0.7% /hr; PATH compression loading.

PRESHEAR STRESS HISTORY ISOTROPIC CONSOLIDATED TO $\bar{\sigma}_c$ = 4.0 kg/cm² and rebound to 1.0 kg/cm².

ELAPSED TIME	AXIAL STRAIN, %	$(\bar{\sigma}_1 - \bar{\sigma}_3)$ (1)	$\frac{(\bar{\sigma}_1 - \bar{\sigma}_3)}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu (2)	$\frac{\Delta u}{\bar{\sigma}_{1c}}$ (2)	A (3)	q	$\bar{p}$	$\bar{\sigma}_r$	$\bar{\sigma}_a$	$\frac{q}{\bar{\sigma}_{vc}}$	$\frac{F}{\bar{\sigma}_{vc}}$	$\frac{q}{\bar{\sigma}_{vm}}$	$\frac{\bar{p}}{\bar{\sigma}_{vm}}$
0	0	0	0	1.000	1.000	1.000	0	0	—	0	1.000			0	1.000	0	0.250
19	0.13	0.199	0.199	0.876	1.075	1.227	0.123	0.123	0.618	0.099	0.976			0.099	0.976	0.025	0.244
45	0.29	0.404	0.404	0.786	1.189	1.514	0.214	0.214	0.530	0.202	0.988			0.202	0.988	0.050	0.247
115	0.96	0.786	0.786	0.700	1.487	2.124	0.299	0.299	0.376	0.393	1.094			0.392	1.094	0.098	0.273
164	1.44	0.939	0.939	0.690	1.629	2.361	0.309	0.309	0.329	0.469	1.156			0.469	1.156	0.117	0.289
214	1.98	1.041	1.041	0.711	1.752	2.464	0.288	0.288	0.277	0.521	1.231			0.521	1.231	0.130	0.307
304	3.00	1.138	1.138	0.724	1.862	2.571	0.275	0.275	0.242	0.569	1.293			0.569	1.293	0.142	0.323
394	4.01	1.192	1.192	0.724	1.916	2.646	0.275	0.275	0.231	0.596	1.320	$\bar{\sigma}_r = \bar{\sigma}_3$	$\bar{\sigma}_a = \bar{\sigma}_1$	0.596	1.320	0.149	0.330
454	4.70	1.239	1.239	0.722	1.961	2.716	0.278	0.278	0.224	0.619	1.341			0.619	1.341	0.155	0.335
514	5.53	1.273	1.273	0.718	1.991	2.774	0.281	0.281	0.221	0.636	1.354			0.636	1.354	0.159	0.338
574	6.22	1.298	1.298	0.716	2.014	2.813	0.283	0.283	0.218	0.649	1.365			0.649	1.365	0.162	0.341
641	6.99	1.320	1.320	0.682	2.002	2.936	0.317	0.317	0.240	0.660	1.342			0.660	1.342	0.165	0.335
723	7.93	1.335	1.335	0.687	2.022	2.944	0.312	0.312	0.234	0.667	1.354			0.667	1.354	0.167	0.338
843	9.35	1.352	1.352	0.669	2.002	3.022	0.330	0.330	0.244	0.676	1.345			0.676	1.346	0.169	0.336
923	10.31	1.364	1.364	0.669	2.003	3.030	0.330	0.330	0.242	0.682	1.351			0.682	1.351	0.170	0.337
1003	11.24	1.366	1.366	0.669	2.003	3.040	0.330	0.330	0.242	0.683	1.352			0.683	1.352	0.171	0.338
1103	12.39	1.362	1.362	0.667	2.029	3.040	0.332	0.332	0.244	0.681	1.348			0.681	1.348	0.170	0.337
1203	13.59	1.358	1.358	0.654	2.012	3.070	0.345	0.345	0.259	0.679	1.333			0.679	1.333	0.169	0.333
1363	15.46	1.369	1.369	0.667	2.036	3.050	0.332	0.332	0.243	0.684	1.351			0.684	1.351	0.171	0.337

(1) CORRECTED FOR ___

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

REMARKS:

CIUC, OCR = 20

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. EIC-1-4
SOIL EAST WINDSOR VARVED CLAY
PROJECT 80142.
TESTED BY SAM., JNL DATE 24 Jan 1975
ALL STRESSES IN kg/cm².

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL	56.3			79.46	8.60	9.93
PRESHEAR	51.4			75.36	7.84	9.61

G_s = ____ TYPE CELL GEO.

PRESHEAR: $\bar{\sigma}_{1c}$ = 1.00, t_c = 1 day.; $\bar{\sigma}_{3c}$ = 1.00, P.P.R. = – %; $\bar{\sigma}_{oc}$ = 1.0, u_B = 2.0

DURING SHEAR: CONTROLLED STRAIN ✓ STRESS ____ RATE 0.7%/hr PATH Compression loading.

PRESHEAR STRESS HISTORY Isotropic consolidated to $\bar{\sigma}_{vc}$ = 4.0 kg/cm² and rebound to 1.0 kg/cm².

ELAPSED TIME	AXIAL STRAIN, %	$(\bar{\sigma}_1 - \bar{\sigma}_3)$ (1)	$\frac{(\bar{\sigma}_1 - \bar{\sigma}_3)}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu (2)	$\frac{\Delta u}{\bar{\sigma}_{1c}}$ (2)	A (3)	q	p̄	$\bar{\sigma}_r$	$\bar{\sigma}_c$	$\frac{q}{\bar{\sigma}_{vc}}$	$\frac{\bar{p}}{\bar{\sigma}_{vc}}$	$\frac{q}{\bar{\sigma}_{vm}}$	$\frac{\bar{p}}{\bar{\sigma}_{vm}}$
0	0	0	0	0.200	0.200	1.000	0	0	–	0	0.200	↑	↑	0	1.000	0	0.050
12	0.052	0.042	0.208	0.169	0.211	1.246	0.031	0.155	0.747	0.021	0.189			0.104	0.949	0.005	0.047
24	0.149	0.089	0.445	0.143	0.232	1.621	0.057	0.284	0.639	0.044	0.187			0.222	0.938	0.011	0.047
42	0.309	0.131	0.653	0.133	0.263	1.983	0.067	0.336	0.515	0.065	0.198			0.326	0.990	0.016	0.050
60	0.492	0.165	0.827	0.130	0.296	2.271	0.070	0.349	0.422	0.083	0.213			0.414	1.064	0.021	0.053
96	0.886	0.228	1.140	0.117	0.345	2.943	0.083	0.414	0.363	0.114	0.231			0.569	1.156	0.028	0.058
141	1.351	0.289	1.446	0.120	0.409	3.412	0.080	0.401	0.277	0.145	0.265			0.723	1.322	0.036	0.066
186	1.846	0.349	1.747	0.128	0.477	3.737	0.073	0.362	0.207	0.175	0.302			0.873	1.512	0.044	0.076
226	2.257	0.393	1.965	0.135	0.528	3.903	0.065	0.323	0.164	0.197	0.332	same as $\bar{\sigma}_3$	same as $\bar{\sigma}_1$	0.982	1.659	0.049	0.083
246	2.493	0.418	2.089	0.135	0.553	4.086	0.065	0.323	0.155	0.209	0.344			1.045	1.721	0.052	0.086
266	2.725	0.439	2.193	0.148	0.587	3.958	0.052	0.258	0.118	0.219	0.368			1.097	1.838	0.055	0.092
306	3.166	0.478	2.389	0.164	0.642	3.918	0.036	0.181	0.076	0.239	0.403			1.195	2.014	0.060	0.101
346	3.652	0.515	2.577	0.177	0.692	3.916	0.023	0.116	0.045	0.258	0.434			1.289	2.172	0.064	0.109
387	4.129	0.547	2.737	0.189	0.737	3.886	0.010	0.051	0.019	0.274	0.463			1.369	2.317	0.068	0.116
429	4.625	0.574	2.867	0.200	0.774	3.867	0.000	0.000	0.000	0.287	0.487			1.433	2.434	0.072	0.122
470	5.137	0.602	3.010	0.208	0.810	3.898	-0.008	-0.039	-0.013	0.301	0.509			1.505	2.544	0.075	0.127
510	5.592	0.613	3.066	0.223	0.837	3.747	-0.023	-0.116	-0.038	0.307	0.530			1.533	2.649	0.077	0.132
570	6.314	0.641	3.203	0.231	0.872	3.773	-0.031	-0.155	-0.048	0.321	0.551			1.602	2.757	0.080	0.138
630	7.034	0.628	3.139	0.254	0.882	3.469	-0.054	-0.271	-0.086	0.314	0.568			1.569	2.841	0.078	0.142
730	8.221	0.622	3.108	0.267	0.889	3.327	-0.067	-0.336	-0.108	0.311	0.578			1.554	2.890	0.078	0.145
850	9.610	0.612	3.061	0.275	0.887	3.227	-0.075	-0.374	-0.122	0.306	0.581			1.531	2.905	0.077	0.145
970	11.044	0.613	3.063	0.285	0.898	3.148	-0.085	-0.426	-0.139	0.306	0.592			1.532	2.958	0.077	0.148
1090	12.505	0.619	3.098	0.290	0.910	3.133	-0.090	-0.452	-0.146	0.309	0.600			1.549	3.001	0.077	0.150
1210	13.960	0.623	3.114	0.296	0.918	3.106	-0.096	-0.478	-0.154	0.311	0.607	↓	↓	1.557	3.035	0.078	0.152

(1) CORRECTED FOR ____

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

REMARKS:

CIUC 45° sample.

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT. OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. EIC-2-1
SOIL EAST WINDSOR VARVED CLAY
PROJECT 80142
TESTED BY JNL DATE 2 May 1975
ALL STRESSES IN kg/cm²

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL	58.7			79.16	7.97	9.93
PRESHEAR	48.2			69.85	7.47	9.35

G_s = ____ TYPE CELL ____

PRESHEAR: $\bar{\sigma}_{1c}$ = 4.0 t_c = 2 days; $\bar{\sigma}_{3c}$ = 4.0 P.P.R. = ____ %; $\bar{\sigma}_{oc}$ = 4.0 u_b = 2.0

DURING SHEAR: CONTROLLED STRAIN ✓ STRESS ____ RATE 0.7 %/hr PATH Compression loading

PRESHEAR STRESS HISTORY ISOTROPIC CONSOLIDATED TO $\bar{\sigma}_{vc}$ = 4.0 kg/cm²

ELAPSED TIME	AXIAL STRAIN, %	$(\bar{\sigma}_1 - \bar{\sigma}_3)$ (1)	$\frac{(\bar{\sigma}_1 - \bar{\sigma}_3)}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu (2)	$\frac{\Delta u}{\bar{\sigma}_{1c}}$ (2)	A (3)	q	$\bar{p}$	$\bar{\sigma}_r$	$\bar{\sigma}_c$	$\frac{q}{\bar{\sigma}_{vc}}$	$\frac{\bar{p}}{\bar{\sigma}_{vc}}$	$\frac{q}{\bar{\sigma}_{vm}}$	$\frac{\bar{p}}{\bar{\sigma}_{vm}}$
	0	0	0	4.000	4.000	1.000	0	0	–	0	4.000	↑	↑	0	1.000	0	1.000
	0.19	0.365	0.091	3.489	3.854	1.104	0.510	0.128	1.398	0.183	3.672			0.046	0.918	0.046	0.918
	0.24	0.414	0.103	3.441	3.854	1.120	0.559	0.139	1.350	0.207	3.648			0.052	0.911	0.052	0.911
	0.33	0.438	0.109	3.417	3.855	1.128	0.583	0.146	1.33	0.219	3.636			0.055	0.909	0.055	0.909
	0.40	0.547	0.137	3.323	3.869	1.164	0.677	0.169	1.23	0.273	3.596			0.068	0.899	0.068	0.899
	0.46	0.734	0.184	3.202	3.936	1.229	0.798	0.199	1.08	0.367	3.569			0.092	0.892	0.092	0.892
	0.51	0.855	0.214	3.119	3.970	1.274	0.881	0.220	1.03	0.428	3.545			0.107	0.886	0.107	0.886
	0.61	1.030	0.258	2.969	4.000	1.347	1.031	0.258	1.00	0.515	3.485			0.129	0.871	0.129	0.871
	0.74	1.166	0.291	2.831	3.966	1.401	1.168	0.292	1.00	0.583	3.399			0.146	0.849	0.146	0.849
	0.85	1.239	0.310	2.732	3.971	1.453	1.268	0.317	1.02	0.619	3.352			0.155	0.838	0.155	0.838
	1.16	1.395	0.349	2.498	3.893	1.558	1.502	0.375	1.076	0.697	3.196			0.174	0.799	0.174	0.799
	1.34	1.449	0.363	2.383	3.832	1.608	1.617	0.404	1.11	0.724	3.108			0.181	0.777	0.181	0.777
	1.64	1.517	0.379	2.230	3.747	1.680	1.770	0.443	1.166	0.758	2.988			0.189	0.747	0.189	0.747
	2.37	1.601	0.400	1.929	3.530	1.830	2.071	0.518	1.293	0.800	2.730	$\bar{\sigma}_r = \bar{\sigma}_3$	$\bar{\sigma}_1 = \bar{\sigma}_a$	0.200	0.682	0.200	0.682
	3.03	1.626	0.406	1.755	3.380	1.926	2.246	0.561	1.381	0.813	2.567			0.203	0.642	0.203	0.642
	3.68	1.632	0.408	1.609	3.241	2.010	2.391	0.598	1.464	0.816	2.425			0.204	0.606	0.204	0.606
	4.71	1.639	0.410	1.435	3.074	2.142	2.565	0.641	1.565	0.819	2.254			0.205	0.564	0.205	0.564
	4.95	1.641	0.410	1.403	3.043	2.169	2.598	0.649	1.583	0.820	2.222			0.205	0.556	0.205	0.556
	5.24	1.643	0.411	1.370	3.013	2.198	2.629	0.657	1.600	0.822	2.191			0.205	0.548	0.205	0.548
	5.44	1.642	0.411	1.343	2.985	2.222	2.657	0.664	1.618	0.821	2.164			0.205	0.541	0.205	0.541
	5.63	1.639	0.410	1.322	2.961	2.239	2.678	0.669	1.634	0.819	2.141			0.205	0.535	0.205	0.535
	6.30	1.627	0.407	1.255	2.882	2.296	2.745	0.686	1.687	0.814	2.068			0.204	0.517	0.204	0.517
	6.63	1.627	0.407	1.228	2.855	2.32	2.772	0.693	1.704	0.813	2.041			0.203	0.510	0.203	0.510
	7.60	1.602	0.400	1.152	2.754	2.391	2.847	0.712	1.777	0.801	1.953			0.200	0.489	0.200	0.489
	8.84	1.553	0.388	1.080	2.634	2.437	2.919	0.730	1.879	0.777	1.857			0.194	0.464	0.194	0.464
	9.61	1.529	0.382	1.045	2.575	2.463	2.955	0.739	1.932	0.765	1.810			0.191	0.453	0.191	0.453
	10.31	1.493	0.373	0.990	2.483	2.508	3.01	0.753	2.019	0.746	1.736			0.187	0.434	0.187	0.434
	10.64	1.471	0.368	0.990	2.461	2.486	3.01	0.753	2.046	0.735	1.698	↓	↓	0.184	0.425	0.184	0.425

(1) CORRECTED FOR ____

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

REMARKS:

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

CIUC, 45 VARVE SAMPLE

SOIL MECHANICS LABORATORY, DEPT. OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. EIC-2-3.

SOIL EAST WINDSOR VARVED CLAY

PROJECT 8014.2

TESTED BY SAM, JNL DATE 29 Jan 1975

ALL STRESSES IN kg/cm²

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL	57.1			79.1	7.97	9.93
PRESHEAR	49.5			70.73	7.71	9.17

G_s = _____ TYPE CELL GEO

PRESHEAR: $\bar{\sigma}_{1c}$ = 1.0 t_c = 1 day; $\bar{\sigma}_{3c}$ = 1.0 P.P.R. = _____ %; $\bar{\sigma}_{ac}$ = 1.0 u_B = 2.0

DURING SHEAR: CONTROLLED STRAIN _____ STRESS _____ RATE 0.7%/hr PATH Comp. Loading

PRESHEAR STRESS HISTORY ISOTROPIC CONSOLIDATED TO $\bar{\sigma}_{vc}$ = 4.0 kg/cm² and rebound to 1.0 kg/cm²

ELAPSED TIME	AXIAL STRAIN, %	$(\bar{\sigma}_1 - \bar{\sigma}_3)$ (1)	$\frac{(\bar{\sigma}_1 - \bar{\sigma}_3)}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu (2)	$\frac{\Delta u}{\bar{\sigma}_{1c}}$ (2)	A (3)	q	p̄	$\bar{\sigma}_r$	$\bar{\sigma}_a$	$\frac{q}{\bar{\sigma}_{vc}}$	$\frac{\bar{p}}{\bar{\sigma}_{vc}}$	$\frac{q}{\bar{\sigma}_{vm}}$	$\frac{\bar{p}}{\bar{\sigma}_{vm}}$
0	0	0	0	1.000	1.000	1.000	0	0	–	0	1.000	↑	↑	0	1.000	0	0.250
22	0.053	0.190	0.190	0.940	1.130	1.202	0.059	0.059	0.314	0.095	1.035			0.095	1.035	0.024	0.259
32	0.109	0.302	0.302	0.907	1.209	1.333	0.094	0.094	0.309	0.151	1.058			0.151	1.058	0.038	0.265
44	0.191	0.399	0.399	0.886	1.286	1.451	0.114	0.114	0.286	0.199	1.086			0.199	1.086	0.050	0.272
56	0.279	0.484	0.484	0.868	1.351	1.557	0.132	0.132	0.274	0.242	1.109			0.242	1.109	0.061	0.277
72	0.409	0.584	0.584	0.855	1.438	1.683	0.145	0.145	0.249	0.292	1.147			0.292	1.147	0.073	0.287
92	0.589	0.684	0.684	0.842	1.526	1.813	0.158	0.158	0.232	0.342	1.184			0.342	1.184	0.086	0.296
112	0.790	0.772	0.772	0.837	1.608	1.922	0.164	0.164	0.212	0.386	1.222			0.386	1.222	0.097	0.306
136	1.016	0.843	0.843	0.824	1.667	2.024	0.177	0.177	0.209	0.422	1.245			0.422	1.245	0.106	0.311
176	1.449	0.956	0.956	0.826	1.782	2.157	0.174	0.174	0.182	0.478	1.304			0.478	1.304	0.120	0.326
216	1.893	1.031	1.031	0.816	1.847	2.264	0.184	0.184	0.179	0.515	1.331			0.515	1.331	0.129	0.333
256	2.366	1.076	1.076	0.813	1.889	2.323	0.187	0.187	0.174	0.538	1.351		$\bar{\sigma}_a = \bar{\sigma}_1$	0.538	1.351	0.135	0.338
296	2.860	1.103	1.103	0.803	1.906	2.374	0.197	0.197	0.179	0.552	1.354			0.552	1.354	0.138	0.339
336	3.338	1.118	1.118	0.798	1.915	2.401	0.203	0.203	0.181	0.559	1.356	$\bar{\sigma}_r = \bar{\sigma}_3$		0.559	1.356	0.140	0.339
396	4.060	1.139	1.139	0.785	1.924	2.452	0.215	0.215	0.189	0.569	1.354			0.569	1.354	0.142	0.339
464	4.871	1.148	1.148	0.772	1.919	2.488	0.228	0.228	0.199	0.574	1.346			0.574	1.346	0.144	0.337
544	5.826	1.138	1.138	0.751	1.889	2.516	0.249	0.249	0.219	0.569	1.320			0.569	1.320	0.142	0.330
664	7.266	1.112	1.112	0.722	1.835	2.541	0.278	0.278	0.249	0.556	1.279			0.556	1.279	0.139	0.320
784	8.721	1.102	1.101	0.704	1.805	2.564	0.296	0.296	0.269	0.550	1.255			0.550	1.255	0.138	0.314
904	10.215	1.051	1.051	0.678	1.729	2.549	0.322	0.322	0.306	0.525	1.203			0.525	1.203	0.131	0.301
1024	11.714	0.992	0.992	0.668	1.659	2.486	0.322	0.322	0.335	0.496	1.164			0.496	1.164	0.124	0.291
1144	13.173	0.979	0.979	0.660	1.639	2.484	0.340	0.340	0.347	0.489	1.149			0.489	1.149	0.122	0.287
1264	14.609	0.983	0.983	0.665	1.648	2.477	0.335	0.335	0.341	0.491	1.157	↓	↓	0.491	1.157	0.123	0.289

(1) CORRECTED FOR _____

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

REMARKS:

CIUC 45' sample., OCR: 20

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT. OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. EIC-Z-4
SOIL EAST WINDSOR VARVED CLAY
PROJECT 80142
TESTED BY JNL DATE 18 Feb. 1975
ALL STRESSES IN kg/cm²

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL	57.1			79.58	8.01	9.93
PRESHEAR	52.4		100	75.13	7.84	9.58

G_s = _____ TYPE CELL GEO.

PRESHEAR: $\bar{\sigma}_{1c}$ = 0.20 t_c = 1 day. $\bar{\sigma}_{3c}$ = 0.20 P.P.R. = _____ % $\bar{\sigma}_{ac}$ = 0.20 u_B = 2.0

DURING SHEAR: CONTROLLED STRAIN ✓ STRESS _____ RATE 0.7%/hr. PATH compression loading

PRESHEAR STRESS HISTORY Isotropicly consolidated to 4.0 kg/cm² and rebound to 0.2 kg/cm²

ELAPSED TIME	AXIAL STRAIN, %	$(\bar{\sigma}_1 - \bar{\sigma}_3)$ (1)	$\frac{(\bar{\sigma}_1 - \bar{\sigma}_3)}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu (2)	$\frac{\Delta u}{\bar{\sigma}_{1c}}$ (2)	A (3)	q	$\bar{p}$	$\bar{\sigma}_r$	$\bar{\sigma}_a$	$\frac{q}{\bar{\sigma}_{vc}}$	$\frac{\bar{p}}{\bar{\sigma}_{vc}}$	$\frac{q}{\bar{\sigma}_{vm}}$	$\frac{\bar{p}}{\bar{\sigma}_{vm}}$
0	0	0	0	0.200	0.200	1.000	0	0	—	0	0.200	↑	↑	0	1.000	0	0.050
49	0.495	0.201	1.006	0.160	0.361	2.257	0.040	0.199	0.199	0.101	0.261			0.503	1.303	0.025	0.065
57	0.590	0.217	1.087	0.158	0.375	2.377	0.042	0.210	0.194	0.109	0.267			0.544	1.333	0.027	0.067
65	0.686	0.233	1.165	0.157	0.386	2.516	0.046	0.231	0.199	0.117	0.270			0.583	1.351	0.029	0.068
80	0.867	0.258	1.292	0.158	0.416	2.637	0.042	0.210	0.163	0.129	0.287			0.646	1.436	0.032	0.072
100	1.109	0.289	1.448	0.166	0.456	2.741	0.034	0.168	0.116	0.145	0.311			0.724	1.556	0.036	0.078
120	1.354	0.317	1.584	0.160	0.477	2.980	0.040	0.199	0.126	0.158	0.319			0.792	1.592	0.040	0.080
160	1.853	0.365	1.823	0.173	0.537	3.112	0.027	0.137	0.075	0.182	0.356			0.912	1.775	0.046	0.089
197	2.395	0.408	2.042	0.185	0.594	3.204	0.015	0.074	0.036	0.204	0.389			1.021	1.947	0.051	0.097
238	2.904	0.445	2.227	0.194	0.639	3.299	0.006	0.032	0.014	0.222	0.416			1.114	2.082	0.056	0.104
278	3.405	0.473	2.364	0.206	0.679	3.292	-0.006	-0.032	-0.013	0.236	0.442			1.182	2.214	0.059	0.111
328	4.038	0.506	2.530	0.229	0.736	3.205	-0.029	-0.147	-0.058	0.253	0.483			1.265	2.412	0.063	0.121
388	4.801	0.532	2.661	0.244	0.776	3.179	-0.044	-0.221	-0.083	0.266	0.510	$\bar{\sigma}_r = \bar{\sigma}_3$	$\bar{\sigma}_a = \bar{\sigma}_1$	1.330	2.551	0.067	0.128
478	5.991	0.518	2.588	0.261	0.778	2.983	-0.061	-0.305	-0.118	0.259	0.519			1.294	2.599	0.065	0.130
598	7.578	0.472	2.358	0.263	0.735	2.793	-0.063	-0.316	-0.134	0.236	0.499			1.179	2.495	0.059	0.125
718	9.476	0.454	2.268	0.269	0.723	2.684	-0.069	-0.347	-0.153	0.227	0.496			1.134	2.481	0.057	0.124
838	10.692	0.466	2.333	0.276	0.742	2.692	-0.076	-0.378	-0.162	0.233	0.509			1.166	2.545	0.058	0.127
958	12.248	0.481	2.404	0.282	0.763	2.705	-0.082	-0.410	-0.171	0.240	0.522			1.202	2.612	0.060	0.131
1078	13.819	0.485	2.427	0.297	0.782	2.636	-0.097	-0.484	-0.199	0.243	0.539			1.213	2.697	0.061	0.135
1198	15.388	0.523	2.613	0.301	0.824	2.737	-0.101	-0.505	-0.193	0.261	0.562			1.306	2.811	0.065	0.141
1318	16.943	0.543	2.714	0.299	0.842	2.816	-0.099	-0.494	-0.182	0.271	0.570	↓	↓	1.357	2.851	0.068	0.143

(1) CORRECTED FOR _____

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

REMARKS:

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. EIC-3-1

SOIL EAST WINDSOR VARVED CLAY

PROJECT 80142

TESTED BY JNL DATE 28 April 75

ALL STRESSES IN kg/cm²

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL	53.7			79.16	7.97	9.93
PRESHEAR	45.8			71.33	7.77	9.17

G_s = ____ TYPE CELL GEO

PRESHEAR: $\bar{\sigma}_{1c}$ = 4.0 t_c = 2 days; $\bar{\sigma}_{3c}$ = 4.0 P.P.R. = - %; $\bar{\sigma}_{oc}$ = 4.0 u_s = 2.0

DURING SHEAR: CONTROLLED STRAIN ✓ STRESS ____ RATE 0.7%/hr PATH Compression loading.

PRESHEAR STRESS HISTORY Isotropic Consolidated $\bar{\sigma}_{vc}$ = 4.0 kg/cm².

ELAPSED TIME	AXIAL STRAIN, %	$(\sigma_1 - \sigma_3)$ (1)	$\frac{(\bar{\sigma}_1 - \bar{\sigma}_3)}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu (2)	$\frac{\Delta u}{\bar{\sigma}_{1c}}$ (2)	A (3)	q	$\bar{p}$	$\bar{\sigma}_r$	$\bar{\sigma}_a$	$\frac{q}{\bar{\sigma}_{vc}}$	$\frac{\bar{p}}{\bar{\sigma}_{vc}}$	$\frac{q}{\bar{\sigma}_{vm}}$	$\frac{\bar{p}}{\bar{\sigma}_{vm}}$
	0	0	0	4.000	4.000	1.000	0	0	-	0	4.000			0	1.000	0	1.000
	0.04	0.117	0.029	3.966	4.083	1.029	0.034	0.009	0.289	0.058	4.020			0.015	1.005	0.015	1.005
	0.07	0.248	0.062	3.927	4.175	1.063	0.073	0.018	0.293	0.124	4.051			0.031	1.01	0.031	1.01
	0.10	0.334	0.084	3.868	4.202	1.086	0.133	0.033	0.396	0.167	3.944			0.042	0.986	0.042	0.986
	0.14	0.528	0.132	3.806	4.328	1.138	0.200	0.050	0.379	0.264	4.064			0.066	1.016	0.066	1.016
	0.17	0.639	0.159	3.729	4.348	1.174	0.270	0.068	0.422	0.319	4.049			0.079	1.010	0.079	1.010
	0.24	0.846	0.212	3.592	4.438	1.236	0.408	0.102	0.481	0.423	4.015			0.106	1.000	0.106	1.000
	0.44	1.207	0.302	3.286	4.492	1.367	0.714	0.179	0.592	0.603	3.888			0.151	0.972	0.151	0.972
	0.74	1.513	0.378	2.948	4.461	1.513	1.052	0.263	0.695	0.756	3.704			0.189	0.926	0.189	0.926
	1.01	1.688	0.422	2.714	4.402	1.622	1.286	0.322	0.761	0.844	3.558			0.211	0.889	0.211	0.889
	1.36	1.844	0.461	2.470	4.314	1.746	1.530	0.383	0.829	0.922	3.391			0.231	0.848	0.231	0.848
	2.06	2.020	0.505	2.153	4.173	1.938	1.847	0.462	0.914	1.010	3.163	$\bar{\sigma}_r = \bar{\sigma}_3$	$\bar{\sigma}_a = \bar{\sigma}_1$	0.253	0.790	0.253	0.790
	2.32	2.060	0.515	2.062	4.122	2.127	1.938	0.485	0.940	1.030	3.092			0.258	0.773	0.258	0.773
	2.82	2.114	0.529	1.912	4.026	2.105	2.088	0.522	0.987	1.057	2.969			0.264	0.742	0.264	0.742
	3.75	2.169	0.542	1.709	3.878	2.269	2.291	0.572	1.056	1.085	2.793			0.271	0.698	0.271	0.698
	4.32	2.223	0.556	1.618	3.841	2.237	2.382	0.596	1.071	1.112	2.729			0.278	0.682	0.278	0.682
	5.33	2.265	0.566	1.475	3.740	2.535	2.525	0.631	1.114	1.132	2.607			0.283	0.651	0.283	0.651
	6.65	2.279	0.569	1.395	3.674	2.633	2.606	0.652	1.143	1.139	2.534			0.284	0.633	0.284	0.633
	6.84	2.295	0.574	1.370	3.660	2.671	2.629	0.657	1.145	1.147	2.515			0.287	0.628	0.287	0.628
	7.39	2.299	0.575	1.334	3.633	2.723	2.665	0.666	1.159	1.149	2.483			0.287	0.620	0.287	0.620
	8.68	2.310	0.578	1.287	3.597	2.79	2.712	0.678	1.174	1.155	2.442			0.288	0.610	0.288	0.610
	9.29	2.311	0.578	1.266	3.577	2.83	2.734	0.684	1.183	1.156	2.421			0.288	0.605	0.288	0.605
	9.99	2.306	0.577	1.253	3.560	2.838	2.746	0.687	1.190	1.153	2.406			0.283	0.601	0.283	0.601
	10.75	2.290	0.573	1.240	3.530	2.846	2.759	0.689	1.204	1.145	2.385			0.286	0.596	0.286	0.596
	11.29	2.271	0.567	1.231	3.503	2.845	2.768	0.692	1.218	1.136	2.367			0.284	0.591	0.284	0.591
	11.93	2.238	0.559	1.222	3.460	2.832	2.777	0.694	1.240	1.119	2.341			0.279	0.585	0.279	0.585

(1) CORRECTED FOR ____

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

REMARKS:

CIUC, OCR: 2.0

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT. OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. EIC3-2
SOIL EAST WINDSOR VARVED CLAY
PROJECT 80142
TESTED BY JNL DATE 21 May 1975
ALL STRESSES IN kg/cm².

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL	57.6			78.79	7.93	9.93
PRESHEAR	45.4		100	69.49	7.70	9.02

G_s = _____ TYPE CELL _____

PRESHEAR: $\bar{\sigma}_{1c}$ = 2.0 kg/cm², $\bar{\sigma}_{3c}$ = 2.0 kg/cm², $\bar{\sigma}_{ac}$ = 2.0 kg/cm², t_c = 1 day, P.P.R. = _____ %, u_B = 2.0 kg/cm²

DURING SHEAR: CONTROLLED STRAIN ✓ STRESS _____, RATE 0.7% hr, PATH compression, loading

PRESHEAR STRESS HISTORY: ISOTROPICALLY CONSOLIDATED TO 4.0 kg/cm² and unload to 2.0 kg/cm².

ELAPSED TIME	AXIAL STRAIN, %	$(\bar{\sigma}_1 - \bar{\sigma}_3)$ (1)	$\frac{(\bar{\sigma}_1 - \bar{\sigma}_3)}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu (2)	$\frac{\Delta u}{\bar{\sigma}_{1c}}$ (2)	A (3)	q	$\bar{p}$	$\bar{\sigma}_r$	$\bar{\sigma}_a$	$\frac{q}{\bar{\sigma}_{vc}}$	$\frac{\bar{p}}{\bar{\sigma}_{vc}}$	$\frac{q}{10\bar{\sigma}_{vm}}$	$\frac{\bar{p}}{10\bar{\sigma}_{vm}}$
0	0	0	0	2.000	2.000	1.000	0	0	–	0	2.000	↑	↑	0	1.000	0	0.050
13	0.036	0.209	0.105	1.971	2.180	1.106	0.029	0.0145	0.139	0.105	2.076			0.052	1.038	0.003	0.052
28	0.083	0.431	0.216	1.937	2.368	1.222	0.063	0.032	0.147	0.215	2.152			0.108	1.076	0.005	0.054
46	0.167	0.683	0.341	1.918	2.601	1.356	0.082	0.041	0.119	0.341	2.259			0.171	1.130	0.009	0.057
63	0.348	1.017	0.508	1.949	2.965	1.522	0.051	0.026	0.050	0.508	2.457			0.254	1.229	0.013	0.061
88	0.443	1.116	0.558	1.891	3.006	1.590	0.169	0.055	0.098	0.558	2.448			0.279	1.224	0.014	0.061
118	0.671	1.278	0.639	1.822	3.100	1.701	0.178	0.089	0.139	0.639	2.461			0.319	1.230	0.016	0.062
158	1.001	1.337	0.669	1.699	3.036	1.787	0.301	0.150	0.225	0.669	2.367			0.334	1.184	0.017	0.059
195	1.391	1.458	0.729	1.614	3.072	1.903	0.386	0.193	0.265	0.729	2.343			0.365	1.172	0.018	0.059
274	2.052	1.579	0.789	1.557	3.136	2.015	0.443	0.222	0.281	0.789	2.346			0.395	1.173	0.020	0.059
334	2.667	1.736	0.868	1.469	3.204	2.181	0.507	0.265	0.306	0.867	2.337			0.434	1.169	0.022	0.058
394	3.300	1.808	0.904	1.384	3.192	2.306	0.615	0.308	0.341	0.904	2.288		$\bar{\sigma}_a = \bar{\sigma}_1$	0.452	1.144	0.023	0.057
454	3.944	1.881	0.940	1.310	3.191	2.436	0.689	0.345	0.367	0.940	2.250	$\bar{\sigma}_r = \bar{\sigma}_3$		0.470	1.125	0.024	0.056
514	4.619	1.967	0.984	1.284	3.251	2.533	0.716	0.358	0.364	0.984	2.267			0.492	1.133	0.025	0.057
574	5.254	1.973	0.987	1.285	3.208	2.496	0.764	0.382	0.387	0.986	2.222			0.493	1.111	0.025	0.056
634	5.890	1.991	0.995	1.201	3.192	2.657	0.798	0.399	0.401	0.995	2.196			0.498	1.098	0.025	0.055
694	6.547	2.031	1.016	1.183	3.214	2.717	0.817	0.409	0.402	1.016	2.199			0.508	1.099	0.025	0.055
754	7.061	1.873	0.937	1.103	2.976	2.698	0.897	0.448	0.479	0.937	2.040			0.468	1.020	0.023	0.051
838	7.929	1.913	0.957	1.075	2.987	2.779	0.925	0.463	0.484	0.956	2.031			0.478	1.016	0.024	0.051
958	9.252	1.952	0.976	1.059	3.011	2.843	0.941	0.470	0.482	0.971	2.035			0.488	1.018	0.024	0.051
1078	10.596	1.954	0.977	1.067	3.021	2.831	0.933	0.467	0.478	0.977	2.044			0.488	1.022	0.024	0.051
1198	11.992	1.894	0.947	1.069	2.964	2.771	0.930	0.465	0.491	0.947	2.017			0.474	1.008	0.024	0.050
1318	13.305	1.765	0.882	1.057	2.822	2.670	0.943	0.472	0.534	0.882	1.939	↓	↓	0.441	0.969	0.022	0.048

(1) CORRECTED FOR _____

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

REMARKS:

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT. OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. EIC-3-3
SOIL EAST WINDSOR
PROJECT 80142
TESTED BY SAM. DATE 7 Feb 1975
ALL STRESSES IN kg/cm²

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL	60.9			79.4	7.99	9.93
PRESHEAR	51.7			71.6	7.74	9.18

G_s = _____ TYPE CELL _____

PRESHEAR: $\bar{\sigma}_{1c}$ = 1.0 t_c = 1 day; $\bar{\sigma}_{3c}$ = 1.0 P.P.R. = – %; $\bar{\sigma}_{ac}$ = 1.0 u_B = 2.0

DURING SHEAR: CONTROLLED STRAIN ✓ STRESS _____ RATE 0.7%/hr PATH Compression loading

PRESHEAR STRESS HISTORY isotropically consolidated to $\bar{\sigma}_{vc}$ = 4.0 kg/cm² and Rebound to 1.0 kg/cm².

ELAPSED TIME	AXIAL STRAIN, %	$(\bar{\sigma}_1 - \bar{\sigma}_3)$ (1)	$\frac{(\bar{\sigma}_1 - \bar{\sigma}_3)}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu (2)	$\frac{\Delta u}{\bar{\sigma}_{1c}}$ (2)	A (3)	q	$\bar{p}$	$\bar{\sigma}_r$	$\bar{\sigma}_a$	$q/\bar{\sigma}_{vc}$	$\bar{p}/\bar{\sigma}_{vc}$	$q/\bar{\sigma}_{vm}$	$\bar{p}/\bar{\sigma}_{vm}$
0	0	0	0	1.00	1.00	1.00	0	0	–	0	1.00			0	1.00	0	0.250
19	0.046	0.357	0.357	0.979	1.336	1.365	0.021	0.021	0.059	0.179	1.157			0.179	1.157	0.045	0.289
31	0.121	0.522	0.522	0.968	1.490	1.539	0.032	0.032	0.060	0.261	1.229			0.261	1.229	0.065	0.307
43	0.215	0.649	0.649	0.959	1.609	1.677	0.040	0.040	0.062	0.325	1.285			0.325	1.285	0.081	0.321
63	0.396	0.814	0.814	0.956	1.770	1.852	0.044	0.044	0.054	0.407	1.363			0.407	1.363	0.102	0.341
85	0.617	0.948	0.948	0.951	1.899	1.997	0.049	0.049	0.051	0.474	1.426			0.474	1.426	0.119	0.357
105	0.832	1.045	1.045	0.947	1.992	2.103	0.053	0.053	0.051	0.523	1.469			0.523	1.469	0.131	0.367
125	1.056	1.121	1.121	0.949	2.066	2.186	0.055	0.055	0.049	0.561	1.506			0.561	1.506	0.140	0.377
145	1.287	1.182	0.939	2.121	2.121	2.259	0.061	0.061	0.052	0.591	1.529			0.591	1.529	0.148	0.382
165	1.529	1.236	1.236	0.928	2.164	2.331	0.072	0.072	0.058	0.618	1.546			0.618	1.546	0.155	0.387
185	1.765	1.283	1.283	0.924	2.207	2.389	0.076	0.076	0.059	0.642	1.566			0.642	1.566	0.161	0.392
205	2.009	1.322	1.322	0.916	2.238	2.445	0.085	0.085	0.064	0.661	1.577			0.662	1.577	0.165	0.394
225	2.257	1.356	1.356	0.909	2.265	2.491	0.091	0.091	0.067	0.678	1.587			0.678	1.587	0.170	0.397
265	2.761	1.406	1.406	0.892	2.298	2.576	0.108	0.108	0.077	0.703	1.596			0.703	1.595	0.176	0.399
325	3.523	1.460	1.460	0.869	2.329	2.681	0.131	0.131	0.089	0.730	1.599			0.730	1.599	0.183	0.400
365	4.019	1.496	1.496	0.856	2.352	2.747	0.144	0.144	0.096	0.748	1.604			0.748	1.604	0.187	0.401
405	4.554	1.522	1.522	0.844	2.366	2.805	0.156	0.156	0.103	0.761	1.645			0.761	1.605	0.190	0.401
504	5.847	1.549	1.549	0.820	2.369	2.888	0.179	0.179	0.116	0.775	1.595			0.775	1.595	0.196	0.399
624	7.432	1.568	1.568	0.799	2.367	2.962	0.201	0.201	0.128	0.784	1.583			0.784	1.583	0.195	0.396
744	9.042	1.559	1.559	0.789	2.348	2.977	0.211	0.211	0.136	0.779	1.568			0.779	1.568	0.195	0.392
864	10.632	1.531	1.531	0.778	2.309	2.968	0.222	0.222	0.145	0.766	1.544			0.766	1.544	0.192	0.386
984	12.271	1.456	1.456	0.768	2.234	2.897	0.232	0.232	0.159	0.728	1.496			0.728	1.496	0.182	0.374
1104	13.918	1.336	1.336	0.751	2.087	2.780	0.249	0.249	0.187	0.668	1.418			0.668	1.419	0.167	0.355
1224	15.536	1.245	1.245	0.732	1.976	2.701	0.268	0.268	0.216	0.622	1.354			0.622	1.354	0.156	0.339
1344	17.137	1.204	1.204	0.711	1.914	2.694	0.289	0.289	0.241	0.602	1.312			0.602	1.312	0.151	0.328

(1) CORRECTED FOR _____

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

REMARKS:

CIUC 90° VANE, OCR: 20

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT. OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. EIC-3-4.
SOIL EAST WINDSOR VARVED CLAY
PROJECT 80142
TESTED BY JNL DATE 2 Jan. 1975
ALL STRESSES IN kg/cm²

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL	61.0			79.38	7.99	9.93
PRESHEAR	54.1		100	74.4	7.77	9.578

G_s = ____ TYPE CELL GEO.

PRESHEAR: $\bar{\sigma}_{1c}$ = 0.20, t_c = 1 day; $\bar{\sigma}_{3c}$ = 0.20, P.P.R. = ____ %; $\bar{\sigma}_{dc}$ = ____, u_B = 2.0

DURING SHEAR: CONTROLLED STRAIN ✓ STRESS ____; RATE 0.7%/hr; PATH LOADING (comp)

PRESHEAR STRESS HISTORY Isotropic consolidated to $\bar{\sigma}_{rm}$ = 4.0 kg/cm² and then rebound to 0.2 kg/cm².

ELAPSED TIME	AXIAL STRAIN, %	$(\bar{\sigma}_1 - \bar{\sigma}_3)$ (1)	$\frac{(\bar{\sigma}_1 - \bar{\sigma}_3)}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu (2)	$\frac{\Delta u}{\bar{\sigma}_{1c}}$ (2)	A (3)	q	$\bar{p}$	$\bar{\sigma}_r$	$\bar{\sigma}_c$	$\frac{q}{\bar{\sigma}_{1c}}$	$\frac{\bar{p}}{\bar{\sigma}_{1c}}$	$\frac{q}{\bar{\sigma}_{rm}}$	$\frac{\bar{p}}{\bar{\sigma}_{rm}}$
0	0	0	0	0.200	0.200	1.000	0	0	—	0	0.200			0	1.000	0	0.050
2	0.012	0.019	0.098	0.198	0.217	1.099	0.002	0.011	0.108	0.009	0.208			0.049	1.036	0.002	0.052
10	0.062	0.113	0.567	0.185	0.299	1.613	0.015	0.074	0.130	0.057	0.242			0.284	1.210	0.014	0.061
18	0.120	0.192	0.958	0.175	0.366	2.096	0.025	0.126	0.132	0.096	0.271			0.479	1.352	0.024	0.068
38	0.305	0.335	1.675	0.164	0.499	3.040	0.036	0.179	0.107	0.168	0.332			0.838	1.658	0.042	0.083
58	0.521	0.409	2.048	0.173	0.582	3.373	0.027	0.137	0.067	0.205	0.378			1.024	1.887	0.051	0.094
78	0.756	0.455	2.278	0.181	0.636	3.516	0.019	0.095	0.042	0.228	0.409			1.139	2.044	0.057	0.102
108	1.115	0.506	2.532	0.194	0.700	3.615	0.063	0.032	0.013	0.253	0.447			1.266	2.234	0.063	0.112
138	1.489	0.555	2.776	0.204	0.759	3.719	-0.004	-0.021	-0.008	0.278	0.482			1.388	2.409	0.069	0.120
168	1.849	0.597	2.984	0.213	0.809	3.807	-0.013	-0.063	-0.024	0.298	0.511			1.492	2.555	0.075	0.128
229	2.604	0.650	3.250	0.232	0.881	3.807	-0.032	-0.158	-0.049	0.325	0.557	$\bar{\sigma}_r = \bar{\sigma}_3$	$\bar{\sigma}_c = \bar{\sigma}_1$	1.625	2.783	0.081	0.139
289	3.352	0.703	3.516	0.248	0.952	3.831	-0.048	-0.242	-0.069	0.352	0.600			1.758	3.000	0.088	0.150
345	4.069	0.729	3.646	0.263	0.992	3.771	-0.063	-0.316	-0.087	0.365	0.628			1.822	3.138	0.091	0.157
405	4.831	0.755	3.773	0.276	1.030	3.737	-0.076	-0.379	-0.100	0.377	0.653			1.887	3.265	0.094	0.163
465	5.599	0.770	3.852	0.284	1.054	3.711	-0.084	-0.421	-0.109	0.385	0.669			1.926	3.347	0.096	0.167
525	6.368	0.781	3.905	0.297	1.078	3.632	-0.097	-0.484	-0.124	0.391	0.687			1.952	3.436	0.098	0.172
585	7.134	0.783	3.915	0.305	1.088	3.566	-0.105	-0.526	-0.134	0.392	0.697			1.958	3.484	0.098	0.174
645	7.908	0.780	3.902	0.314	1.094	3.488	-0.114	-0.568	-0.146	0.390	0.704			1.951	3.519	0.098	0.176
705	8.689	0.775	3.874	0.318	1.093	3.438	-0.118	-0.589	-0.152	0.387	0.705			1.937	3.526	0.097	0.176
795	9.891	0.742	3.709	0.316	1.058	3.350	-0.116	-0.579	-0.156	0.371	0.687			1.855	3.433	0.093	0.172
915	11.478	0.712	3.559	0.314	1.025	3.270	-0.114	-0.568	-0.159	0.356	0.669			1.779	3.348	0.089	0.167
1035	13.080	0.681	3.406	0.309	0.991	3.202	-0.109	-0.547	-0.161	0.341	0.650			1.703	3.250	0.085	0.163

(1) CORRECTED FOR ____

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

REMARKS:

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT. OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. EIE-1-1
SOIL EAST WINDSOR JAMESDCLAY
PROJECT 80142
TESTED BY SAM. DATE 27 July 1975
ALL STRESSES IN kg/cm²

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL	57.1			79.33	7.99	9.93
PRESHEAR	47.0			70.58	7.49	9.42

σ_3 = ____ TYPE CELL GEO

PRESHEAR: σ_{ic} = 4.10 t_c = 2 days; σ_{3c} = 4.10 P.P.R. = — %; σ_{oc} = — u_0 = 2.00

PRESHEAR STRESS HISTORY Isotropic consolidated to σ_{vc} = 4.10 kg/cm²

DURING SHEAR: CONTROLLED STRAIN ✓ STRESS ____ RATE 0.7 % /hr PATH EXTENSION UNLOADING

ELAPSED TIME	AXIAL STRAIN, %	$\sigma_v - \sigma_h$	$\frac{\sigma_v - \sigma_h}{\bar{\sigma}_{vc}}$	$\bar{\sigma}_v$	$\bar{\sigma}_h$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu (2)	$\frac{\Delta u - \Delta\sigma_3}{\bar{\sigma}_{vc}}$	A (3)	q	p	σ_r	σ_a	$\frac{2q}{\bar{\sigma}_{vc}}$	$\frac{\bar{p}}{\bar{\sigma}_{vc}}$	$\frac{2q}{\bar{\sigma}_{vm}}$	$\frac{\bar{p}}{\bar{\sigma}_{vm}}$
0	0	0	0	4.000	4.000	1.000	0	0	—	0	4.000	4.000	4.000	0	1.000	0	1.000
10	0.025	-0.165	0.0414	3.934	4.099	1.016	-0.099	0.397	0.397	-0.082	4.010	4.099	3.934	0.041	1.004	0.041	1.004
24	0.080	-0.392	0.098	3.805	4.197	1.103	-0.197	0.497	0.497	-0.196	4.001	4.197	3.805	0.098	1.000	0.098	1.000
44	0.206	-0.665	0.166	3.584	4.249	1.185	-0.249	0.626	0.626	-0.333	3.916	4.249	3.584	0.166	0.979	0.166	0.979
60	0.308	-0.851	0.212	3.402	4.253	1.250	-0.253	0.703	0.703	-0.426	3.827	4.253	3.402	0.212	0.956	0.212	0.956
98	0.628	-1.184	0.296	3.085	4.189	1.394	-0.189	0.840	0.840	-0.592	3.597	4.189	3.085	0.296	0.899	0.296	0.899
118	0.823	-1.309	0.327	2.826	4.135	1.462	-0.135	0.897	0.897	-0.654	3.481	4.135	2.826	0.327	0.870	0.327	0.870
173	1.470	-1.547	0.387	2.406	3.953	1.643	0.047	1.030	1.030	-0.773	3.179	3.953	2.406	0.387	0.794	0.387	0.794
228	2.16	-1.684	0.421	2.080	3.764	1.809	0.236	1.140	1.140	-0.842	2.922	3.764	2.080	0.421	0.730	0.421	0.730
253	2.28	-1.705	0.426	2.026	3.733	1.841	0.267	1.156	1.156	-0.857	2.880	3.733	2.028	0.426	0.720	0.426	0.720
383	2.61	-1.752	0.438	1.912	3.664	1.916	0.336	1.192	1.192	-0.876	2.799	3.664	1.912	0.438	0.697	0.438	0.697
513	4.42	-1.882	0.470	1.495	3.377	2.258	0.623	1.331	1.331	-0.941	2.436	3.377	1.495	0.470	0.609	0.470	0.609
553	4.95	-1.895	0.474	1.416	3.311	2.338	0.689	1.364	1.364	-0.947	2.363	3.311	1.416	0.474	0.590	0.474	0.590
613	5.75	-1.925	0.481	1.317	3.242	2.462	0.758	1.394	1.394	-0.962	2.279	3.242	1.317	0.481	0.569	0.481	0.569
673	6.56	-1.958	0.485	1.243	3.201	2.575	0.799	1.408	1.408	-0.979	2.222	3.201	1.243	0.485	0.555	0.485	0.555
733	7.40	-1.982	0.495	1.137	3.119	2.743	0.881	1.445	1.445	-0.991	2.128	3.119	1.137	0.495	0.532	0.495	0.532
793	8.24	-1.997	0.499	1.071	3.068	2.864	0.932	1.467	1.467	-0.998	2.069	3.068	1.071	0.499	0.517	0.499	0.517
813	9.05	-2.010	0.503	1.013	3.030	2.991	0.970	1.483	1.483	-1.005	2.021	3.030	1.013	0.503	0.505	0.503	0.505
873	9.86	-2.017	0.504	0.983	3.000	3.056	0.996	1.494	1.494	-1.008	1.991	3.000	0.983	0.504	0.497	0.504	0.497
933	10.66	-2.013	0.503	0.975	2.988	3.060	1.012	1.503	1.503	-1.006	1.981	2.988	0.975	0.503	0.495	0.503	0.495
993	11.48	-2.028	0.507	0.945	2.973	3.146	1.026	1.506	1.506	-1.014	1.959	2.973	0.945	0.507	0.489	0.507	0.489

(1) CORRECTED FOR ____

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_3}{\Delta\sigma_1 - \Delta\sigma_3}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_3}{\Delta\sigma_3 - \Delta\sigma_1}$ FOR EXTENSION TESTS

REMARKS:

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT. OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. EIE-1-2

SOIL EAST WINDSOR VARVED CLAY

PROJECT 80142.

TESTED BY SAM. DATE

ALL STRESSES IN

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL	57.4			78.19	7.87	9.93
PRESHEAR	46.6			69.82	7.47	9.35

G_s = TYPE CELL

PRESHEAR

$\bar{\sigma}_{1c}$ = 2.0 t_c = 1 day.

$\bar{\sigma}_{3c}$ = 2.0 P.P.R. = — %

$\bar{\sigma}_{oc}$ = 2.0 u_b = 2.0

DURING SHEAR

CONTROLLED STRAIN ✓ STRESS

RATE 0.7%/hr

PATH EXTENSION UNLOADING

PRESHEAR STRESS HISTORY ISOTROPIC consolidated to $\bar{\sigma}_{VC}$ = 4.0 kg/cm² and rebound to 2.0 kg/cm²

ELAPSED TIME	AXIAL STRAIN, %	$\sigma_v - \sigma_h$	$\frac{\sigma_v - \sigma_h}{\bar{\sigma}_{VC}}$	$\bar{\sigma}_v$	$\bar{\sigma}_h$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu [2]	$\frac{\Delta u - \Delta \sigma_3}{\bar{\sigma}_{VC}}$	A [3]	q	p	$\bar{\sigma}_r$	$\bar{\sigma}_a$	$\frac{q}{\bar{\sigma}_{VC}}$	$\frac{\bar{p}}{\bar{\sigma}_{VC}}$	$\frac{q}{\bar{\sigma}_{VM}}$	$\frac{\bar{p}}{\bar{\sigma}_{VM}}$
	0	0	0	2.000	2.000	1.000	0	0	—	0	2.000	2.000	2.000	-0	1.000	0	0.500
	0.020	-0.156	-0.078	1.961	2.117	1.079	-0.117	0.059	0.250	-0.078	2.039	2.117	1.961	-0.039	1.019	0.020	0.509
	0.036	-0.262	-0.131	1.936	2.198	1.135	-0.198	0.032	0.244	-0.131	2.067	2.198	1.936	-0.066	1.033	0.033	0.517
	0.051	-0.332	-0.166	1.919	2.251	1.175	-0.251	0.040	0.122	-0.166	2.085	2.251	1.919	-0.083	1.042	0.042	0.521
	0.097	-0.492	-0.246	1.885	2.377	1.261	-0.377	0.057	0.117	-0.246	2.131	2.377	1.885	-0.123	1.066	0.062	0.533
	0.163	-0.634	-0.316	1.856	2.490	1.342	-0.490	0.072	0.227	-0.317	2.173	2.490	1.856	-0.158	1.086	0.079	0.543
	0.237	-0.760	-0.380	1.828	2.588	1.416	-0.588	0.086	0.226	-0.380	2.208	2.588	1.828	-0.190	1.104	0.095	0.552
	0.515	-1.039	-0.519	1.757	2.796	1.591	-0.796	0.122	0.234	-0.519	2.276	2.796	1.757	-0.260	1.138	0.130	0.569
	0.962	-1.245	-0.622	1.634	2.879	1.762	-0.897	0.174	0.279	-0.622	2.257	2.879	1.634	-0.311	1.128	0.156	0.564
	1.52	-1.406	-0.703	1.524	2.930	1.922	-0.936	0.238	0.339	-0.703	2.227	2.930	1.524	-0.352	1.113	0.176	0.556
	2.18	-1.502	-0.751	1.407	2.909	2.067	-0.909	0.297	0.395	-0.751	2.158	2.909	1.407	-0.375	1.079	0.188	0.539
	3.02	-1.575	-0.787	1.276	2.851	2.234	-0.851	0.362	0.459	-0.788	2.063	2.851	1.276	-0.383	1.031	0.197	0.516
	3.83	-1.625	-0.812	1.175	2.800	2.383	-0.800	0.413	0.509	-0.812	1.988	2.800	1.175	-0.406	0.994	0.203	0.497
	5.06	-1.690	-0.845	1.061	2.751	2.593	-0.751	0.469	0.555	-0.845	1.906	2.751	1.061	-0.422	0.953	0.211	0.476
	6.01	-1.708	-0.855	0.996	2.704	2.714	-0.704	0.502	0.587	-0.854	1.850	2.704	0.996	-0.427	0.925	0.218	0.462
	8.46	-1.832	-0.916	0.826	2.658	3.218	-0.658	0.587	0.641	-0.916	1.742	2.658	0.926	-0.458	0.871	0.229	0.436
	9.79	-1.850	-0.925	0.806	2.656	3.295	-0.656	0.597	0.645	-0.925	1.731	2.656	0.806	-0.462	0.866	0.231	0.433

[1] CORRECTED FOR

[2] Δu FOR $\Delta\sigma_3 = 0$

[3] $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_3}{\Delta\sigma_a - \Delta\sigma_3}$ FOR EXTENSION TESTS

REMARKS:

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT. OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. EIE-1-3

SOIL EAST WINDSOR VARVED CLAY

PROJECT 8042

TESTED BY SAM. DATE 6 July 1975

ALL STRESSES IN kg/cm²

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL	56.3			79.24	7.98	9.93
PRESHEAR	47.8			72.17	7.65	9.44

σ_3 = ____ TYPE CELL GEO

PRESHEAR: σ_{1c} = 1.0 t_c = 1 day; σ_{3c} = 1.0 P.P.R. = ____ %; σ_{oc} = 1.0 u_B = 2.0

DURING SHEAR: CONTROLLED STRAIN ✓ STRESS ____ RATE 0.7 %/hr PATH EXTENSION UNLOADING

PRESHEAR STRESS HISTORY ISOTROPIC CONSOLIDATED TO $\bar{\sigma}_{VC}$ = 4.0 kg/cm² and rebound to 1.0 kg/cm².

ELAPSED TIME	AXIAL STRAIN, %	$\sigma_v - \sigma_H$	$\frac{\sigma_v - \sigma_H}{\bar{\sigma}_{VC}}$	$\bar{\sigma}_V$	$\bar{\sigma}_H$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu (2)	$\frac{\Delta u - \Delta\sigma_3}{\bar{\sigma}_{VC}}$	A (3)	q	p̄	$\bar{\sigma}_r$	$\bar{\sigma}_a$	$2q/\bar{\sigma}_{VC}$	$\bar{p}/\bar{\sigma}_{VC}$	$2q/\bar{\sigma}_{VM}$	$\bar{p}/\bar{\sigma}_{VM}$
0	0	0	0	1.000	1.000	1.000	0	0	—	0	1.000	1.000	1.000	0	1.000	0	0.250
10	0.101	-0.246	-0.246	0.922	1.168	1.266	-0.168	0.078	0.317	0.246	1.045	1.168	0.922	0.246	1.045	0.031	0.261
22	0.177	-0.398	-0.398	0.892	1.290	1.446	-0.290	0.108	0.271	0.398	1.091	1.290	0.892	0.398	1.091	0.049	0.272
37	0.319	-0.552	-0.552	0.865	1.417	1.638	-0.417	0.135	0.244	0.552	1.141	1.417	0.865	0.552	1.141	0.069	0.285
63	0.586	-0.732	-0.732	0.835	1.567	1.877	-0.567	0.165	0.225	0.732	1.201	1.567	0.835	0.732	1.201	0.091	0.300
99	0.974	-0.909	-0.909	0.800	1.709	2.136	-0.709	0.200	0.220	0.909	1.254	1.709	0.800	0.909	1.254	0.114	0.313
168	1.734	-1.069	-1.069	0.765	1.834	2.397	-0.834	0.235	0.219	1.069	1.299	1.834	0.765	1.069	1.299	0.134	0.325
222	2.497	-1.153	-1.153	0.733	1.886	2.573	-0.886	0.267	0.232	1.153	1.309	1.886	0.733	1.153	1.309	0.144	0.327
282	3.220	-1.217	-1.217	0.704	1.921	2.729	-0.921	0.296	0.243	1.217	1.313	1.921	0.704	1.217	1.313	0.152	0.328
342	4.027	-1.275	-1.275	0.666	1.941	2.757	-0.941	0.334	0.262	1.275	1.303	1.941	0.666	1.275	1.303	0.159	0.326
402	4.835	-1.317	-1.317	0.635	1.952	3.074	-0.952	0.365	0.277	1.317	1.294	1.952	0.635	1.317	1.294	0.165	0.323
462	5.616	-1.366	-1.366	0.611	1.977	3.236	-0.977	0.389	0.284	1.366	1.294	1.977	0.611	1.366	1.294	0.171	0.324
522	6.405	-1.392	-1.392	0.594	1.986	3.343	-0.986	0.406	0.292	1.392	1.290	1.986	0.594	1.392	1.290	0.174	0.323
577	7.122	-1.408	-1.408	0.584	1.992	3.410	-0.992	0.416	0.295	1.408	1.288	1.992	0.584	1.408	1.288	0.176	0.322
637	7.890	-1.454	-1.454	0.566	2.020	3.569	-1.020	0.434	0.298	1.454	1.293	2.020	0.566	1.454	1.293	0.182	0.323
757	9.51	-1.490	-1.490	0.548	2.038	3.719	-1.038	0.452	0.303	1.490	1.293	2.038	0.548	1.490	1.293	0.186	0.323
847	10.70	-1.495	-1.495	0.556	2.051	3.689	-1.051	0.444	0.297	1.493	1.303	2.051	0.556	1.495	1.303	0.187	0.325

(1) CORRECTED FOR ____

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_3}{\Delta\sigma_1 - \Delta\sigma_3}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_3}{\Delta\sigma_3 - \Delta\sigma_1}$ FOR EXTENSION TESTS

REMARKS:

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT. OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. EIE-3-1
SOIL EAST WINDSOR
PROJECT
TESTED BY SAM. DATE 16 May 1975
ALL STRESSES IN kg/cm²

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL	56.6			71.2	7.98	9.93
PRESHEAR	45.6			70.58	7.72	9.14

G_s = TYPE CELL

PRESHEAR STRESS HISTORY

PRESHEAR
$\bar{\sigma}_{ic}$ = 4.0 t_c = 2 days
$\bar{\sigma}_{3c}$ = 4.0 P.P.R. = — %
$\bar{\sigma}_{ac}$ = 4.0 u_s = 2.0

DURING SHEAR
CONTROLLED STRAIN ✓ STRESS
RATE 0.7%/hr.
PATH Extension unloading.

ELAPSED TIME	AXIAL STRAIN, %	$\sigma_v - \sigma_h$	$\frac{\sigma_v - \sigma_h}{\bar{\sigma}_{vc}}$	$\bar{\sigma}_h$	$\bar{\sigma}_v$	$\bar{\sigma}_1 / \bar{\sigma}_3$	Δu (2)	$\frac{\Delta u - \Delta\sigma_h}{\bar{\sigma}_{vc}}$	A (3)	q	$\bar{p}$	$\bar{\sigma}_r$	$\bar{\sigma}_a$	$q/\bar{\sigma}_{ic}$	$\bar{p}/\bar{\sigma}_{vc}$		
	0	0	0	4.00	4.00	1.00	0	0	—	0	4.00	4.00	4.00	0	1.000		
	0.010	0.032	0.008	3.98	4.01	1.00	-0.010	0.006	0.688	.016	3.99	4.01	3.98	0.004	0.999		
	0.026	0.216	0.054	3.81	4.03	1.06	-0.029	0.047	0.866	.108	3.93	4.03	3.81	0.027	0.981		
	0.089	0.777	0.194	3.28	4.06	1.24	-0.057	0.180	0.924	.388	3.668	4.06	3.28	0.097	0.917		
	0.149	1.004	0.251	3.010	4.05	1.35	-0.045	0.240	0.955	.504	3.528	4.05	3.010	0.126	0.882		
	0.227	1.238	0.310	2.789	4.03	1.44	-0.027	0.303	0.978	.626	3.408	4.03	2.789	0.155	0.852		
	0.343	1.456	0.364	2.528	3.98	1.57	+0.015	0.368	1.016	.728	3.256	3.98	2.528	0.182	0.814		
	0.720	1.767	0.442	2.062	3.83	1.86	+0.171	0.485	1.097	.884	2.944	3.83	2.062	0.221	0.736		
	1.48	1.978	0.495	1.581	3.56	2.25	0.441	0.605	1.213	.988	2.584	3.56	1.581	0.247	0.646		
	2.27	2.046	0.512	1.319	3.37	2.55	0.635	0.670	1.310	1.024	2.344	3.27	1.319	0.256	0.586		
	3.05	2.076	0.519	1.148	3.22	2.81	0.776	0.713	1.374	1.040	2.188	3.22	1.148	0.260	0.547		
	3.79	2.110	0.528	1.042	3.15	3.02	0.848	0.740	1.402	1.056	2.096	3.15	1.042	0.264	0.524		
	4.46	2.130	0.532	0.963	3.09	3.21	0.907	0.759	1.426	1.064	2.028	3.09	0.963	0.266	0.507		
	4.60	2.133	0.533	0.949	3.08	3.25	0.918	0.763	1.430	1.068	2.016	3.08	0.949	0.267	0.504		
	5.01	2.137	0.534	0.911	3.05	3.35	0.952	0.772	1.445	1.068	1.980	3.05	0.911	0.267	0.495		
	5.42	2.138	0.535	0.878	3.02	3.44	0.984	0.781	1.466	1.068	1.948	3.02	0.878	0.267	0.487		
	6.24	2.140	0.535	0.826	2.96	3.61	1.040	0.795	1.486	1.072	1.892	2.96	0.826	0.268	0.473		

(1) CORRECTED FOR

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

REMARKS:

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT. OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. ETE-3-2

SOIL EAST WINDSOR VARVED CLAY

PROJECT 80142

TESTED BY SAM. DATE 25 July.

ALL STRESSES IN

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL	59.0			79.24	7.98	9.93
PRESHEAR	49.2			71.66	7.73	9.18

G_s = TYPE CELL CLO.

PRESHEAR

$\bar{\sigma}_{1c}$ = 2.00 t_c = 1 day.

$\bar{\sigma}_{3c}$ = 2.00 P.P.R. = %

$\bar{\sigma}_{ac}$ = 2.00 u_s =

PRESHEAR STRESS HISTORY Isotropic consolidated $\bar{\sigma}_{vc}$ = 4.0 kg/cm² and rebound to 2.0 kg/cm².

DURING SHEAR

CONTROLLED STRAIN ✓ STRESS

RATE 0.7%/hr

PATH Extension Unloading

ELAPSED TIME	AXIAL STRAIN, %	$\sigma_v - \sigma_h$	$\bar{\sigma}_v$	$\bar{\sigma}_h$	$\frac{\sigma_v - \sigma_h}{\bar{\sigma}_{vc}}$	$\bar{\sigma}_1 / \bar{\sigma}_3$	Δu (2)	$\frac{\Delta u - \Delta \sigma_h}{\bar{\sigma}_{vc}}$	A (3)	q	$\bar{p}$	$\bar{\sigma}_r$	$\bar{\sigma}_a$	$q/\bar{\sigma}_{vc}$	$\bar{p}/\bar{\sigma}_{vc}$	$q/\bar{\sigma}_{vm}$	$\bar{p}/\bar{\sigma}_{vm}$
	0	0	2.00	2.00	0	1.0	0	0	—	0	0	2.0	2.0	0	0	0	0.500
	0.04	0.190	2.05	1.864	0.095	1.10	-0.054	0.068	0.721	0.095	1.960	2.05	1.864	0.048	0.980	0.024	0.490
	0.079	0.424	2.123	1.699	0.212	1.25	-0.113	0.156	0.733	0.212	1.912	2.123	1.699	0.106	0.956	0.053	0.478
	0.149	0.607	2.202	1.595	0.304	1.38	-0.202	0.203	0.677	0.304	1.906	2.202	1.595	0.152	0.950	0.076	0.475
	0.294	0.920	2.304	1.384	0.460	1.66	-0.304	0.308	0.670	0.460	1.844	2.304	1.384	0.230	0.922	0.115	0.461
	0.479	1.118	2.384	1.266	0.559	1.88	-0.384	0.367	0.657	0.559	1.824	2.384	1.266	0.280	0.912	0.140	0.456
	0.849	1.338	2.472	1.134	0.669	2.18	-0.472	0.433	0.647	0.669	1.804	2.472	1.134	0.334	0.902	0.167	0.451
	1.32	1.494	2.523	1.029	0.747	2.45	-0.523	0.486	0.650	0.747	1.776	2.523	1.029	0.374	0.888	0.187	0.444
	2.05	1.620	2.542	0.922	0.810	2.76	-0.542	0.539	0.665	0.810	1.732	2.542	0.922	0.406	0.866	0.203	0.433
	2.86	1.692	2.529	0.837	0.846	3.02	-0.529	0.582	0.687	0.846	1.684	2.529	0.837	0.424	0.842	0.212	0.421
	4.07	1.762	2.499	0.737	0.881	3.39	-0.499	0.632	0.717	0.881	1.620	2.499	0.737	0.440	0.810	0.220	0.405
	5.27	1.792	2.474	0.680	0.896	3.64	-0.474	0.659	0.735	0.896	1.576	2.474	0.680	0.448	0.788	0.224	0.394
	6.05	1.805	2.475	0.670	0.903	3.69	-0.455	0.675	0.748	0.903	1.572	2.475	0.670	0.452	0.786	0.226	0.393

(1) CORRECTED FOR

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

REMARKS:

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT. OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. EIE-3-3
SOIL EAST WINDSOR VARVED CLAY
PROJECT 80142
TESTED BY SAM. DATE 17 July 1975
ALL STRESSES IN kg/cm²

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL	36.2			77.14	7.98	9.93
PRESHEAR	47.0			74.81	7.72	9.30

G_s = _____ TYPE CELL GEO

PRESHEAR: $\bar{\sigma}_{1c}$ = 1.0, t_c = 1 day; $\bar{\sigma}_{3c}$ = 1.0, P.P.R. = — %; $\bar{\sigma}_{oc}$ = 1.0, u_s = 2.0

DURING SHEAR: CONTROLLED STRAIN ✓ STRESS ____; RATE 0.7% / hr.; PATH EXTENSION UNLOADING

PRESHEAR STRESS HISTORY Isotropic Consolidated to $\bar{\sigma}_{VC}$ = 4.0 kg/cm² and rebound to 1.0 kg/cm²

ELAPSED TIME	AXIAL STRAIN, %	$\sigma_v - \sigma_H$	$\frac{\sigma_v - \sigma_H}{\bar{\sigma}_{VC}}$	$\bar{\sigma}_v$	$\bar{\sigma}_H$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu (2)	$\frac{\Delta u - \Delta\sigma_3}{\bar{\sigma}_{VC}}$	A (3)	q	p̄	$\bar{\sigma}_f$	$\bar{\sigma}_a$	$q/\bar{\sigma}_{VC}$	$\bar{p}/\bar{\sigma}_{VC}$	$q/\bar{\sigma}_{VM}$	$\bar{p}/\bar{\sigma}_{VM}$
	0	0	0	1.000	1.000	1.000	0	0	—	0	1.000	1.000	1.000	0	1.000	0	0.250
	0.005	0.090	0.090	1.028	0.937	1.096	-0.028	0.063	0.694	0.045	0.982	1.028	0.937	0.045	0.982	0.011	0.246
	0.012	0.140	0.140	1.041	0.901	1.155	-0.041	0.099	0.705	0.070	0.971	1.041	0.901	0.070	0.971	0.018	0.242
	0.018	0.173	0.173	1.050	0.877	1.197	-0.050	0.123	0.711	0.086	0.963	1.050	0.877	0.086	0.963	0.022	0.240
	0.123	0.383	0.383	1.127	0.745	1.514	-0.128	0.255	0.667	0.192	0.936	1.127	0.745	0.192	0.936	0.048	0.234
	0.275	0.554	0.554	1.206	0.652	1.849	-0.206	0.348	0.628	0.277	0.929	1.206	0.652	0.277	0.929	0.069	0.232
	0.626	0.772	0.772	1.328	0.556	2.388	-0.328	0.444	0.575	0.386	0.942	1.328	0.556	0.386	0.942	0.096	0.236
	1.55	1.082	1.082	1.544	0.462	3.341	-0.544	0.538	0.497	0.541	1.083	1.544	0.462	0.541	1.003	0.135	0.251
	1.64	1.102	1.102	1.559	0.457	3.411	-0.599	0.543	0.493	0.551	1.508	1.559	0.457	0.551	1.008	0.138	0.252
	2.13	1.194	1.194	1.635	0.441	3.707	-0.635	0.559	0.468	0.597	1.038	1.635	0.441	0.597	1.038	0.149	0.260
	3.34	1.334	1.334	1.758	0.424	4.146	-0.758	0.576	0.432	0.667	1.091	1.758	0.424	0.667	1.091	0.167	0.273
	4.40	1.402	1.402	1.829	0.427	4.283	-0.829	0.573	0.409	0.701	1.128	1.829	0.427	0.701	1.128	0.175	0.282
	4.68	1.377	1.377	1.826	0.449	4.067	-0.826	0.551	0.406	0.688	1.138	1.826	0.449	0.688	1.377	0.172	0.284

(1) CORRECTED FOR _____

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_3}{\Delta\sigma_1 - \Delta\sigma_3}$ FOR EXTENSION TESTS

REMARKS:

SECTION D.2-1

TABULATION OF TEST DATA FROM $\overline{CK_oUC}$ AND $\overline{CK_oUE}$ TESTS

ON NC AND OC SHANSEP EAST WINDSOR

SAMPLES

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT. OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. EKC-1.
SOIL EAST WINDSOR VARVED CLAY
PROJECT 80142
TESTED BY J.N.L. DATE 23 March 1975
ALL STRESSES IN kg/cm²

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL	56.3			79.24	7.98	9.93
PRESHEAR	—			72.30	7.33	9.86

G_s = _____ TYPE CELL GEO

PRESHEAR: $\bar{\sigma}_{1c}$ = 3.599 t_c = 2 days; $\bar{\sigma}_{3c}$ = 2.430 P.P.R. = _____ %; $\bar{\sigma}_{oc}$ = _____ u_B = 2.0

DURING SHEAR: CONTROLLED STRAIN ✓ STRESS _____; RATE 0.7%/hr.; PATH LOADING.

PRESHEAR STRESS HISTORY Ko CONSOLIDATION UP TO 3.599 kg/cm².

ELAPSED TIME	AXIAL STRAIN, %	$(\bar{\sigma}_1 - \bar{\sigma}_3)$ (1)	$\frac{(\bar{\sigma}_1 - \bar{\sigma}_3)}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu (2)	$\frac{\Delta u}{\bar{\sigma}_{1c}}$ (2)	A (3)	q	p	$\bar{\sigma}_r$	$\bar{\sigma}_a$	$\frac{q}{\bar{\sigma}_{vc}}$	$\frac{p}{\bar{\sigma}_{vc}}$		
0	0	1.169	0.325	2.430	3.599	1.481	0	0	—	0.585	3.014	↑	↑	0.1624	0.837		
4	0.0007	1.211	0.337	2.401	3.613	1.504	0.028	0.007	0.668	0.606	3.007			0.1683	0.835		
6	0.0023	1.246	0.346	2.373	3.619	1.525	0.041	0.016	0.734	0.623	2.996			0.1731	0.832		
8	0.0041	1.285	0.357	2.344	3.630	1.548	0.085	0.024	0.733	0.643	2.987			0.1785	0.829		
10	0.0087	1.321	0.367	2.319	3.640	1.569	0.111	0.031	0.729	0.661	2.979			0.1835	0.826		
14	0.0160	1.350	0.375	2.296	3.646	1.588	0.134	0.037	0.741	0.675	2.971			0.1875	0.822		
18	0.0322	1.419	0.394	2.241	3.660	1.633	0.189	0.052	0.755	0.709	2.951			0.1971	0.818		
22	0.0488	1.458	0.405	2.213	3.671	1.659	0.217	0.060	0.751	0.729	2.942			0.2025	0.817		
26	0.0687	1.501	0.417	2.169	3.671	1.692	0.261	0.072	0.785	0.751	2.919			0.2086	0.811		
30	0.1001	1.555	0.432	2.125	3.679	1.731	0.305	0.085	0.791	0.771	2.902			0.2156	0.806		
34	0.1171	1.581	0.439	2.094	3.674	1.754	0.336	0.093	0.809	0.790	2.884			0.2195	0.801		
40	0.1686	1.629	0.452	2.045	3.674	1.796	0.385	0.107	0.837	0.814	2.859			0.2262	0.794		
46	0.2165	1.676	0.465	1.983	3.658	1.845	0.447	0.124	0.883	0.837	2.820	$\bar{\sigma}_r = \bar{\sigma}_3$		0.2327	0.783		
58	0.330	1.724	0.479	1.898	3.622	1.908	0.532	0.148	0.959	0.862	2.759			0.2394	0.766		
70	0.437	1.750	0.486	1.818	3.567	1.963	0.612	0.170	1.066	0.875	2.697		$\bar{\sigma}_a = \bar{\sigma}_1$	0.2430	0.748		
82	0.566	1.783	0.495	1.743	3.525	2.023	0.687	0.191	1.119	0.892	2.634			0.2476	0.732		
94	0.7019	1.798	0.499	1.681	3.499	2.080	0.749	0.208	1.189	0.899	2.580			0.2498	0.717		
106	0.8359	1.799	0.500	1.626	3.426	2.107	0.804	0.223	1.276	0.899	2.526			0.2500	0.702		
112	0.9116	1.810	0.503	1.597	3.408	2.133	0.832	0.231	1.297	0.905	2.503			0.2508	0.695		
118	0.9730	1.806	0.502	1.574	3.381	2.147	0.855	0.237	1.342	0.903	2.477			0.2508	0.688		
124	1.0344	1.789	0.497	1.564	3.353	2.144	0.866	0.241	1.396	0.894	2.458			0.2485	0.683		
130	1.1047	1.808	0.502	1.531	3.339	2.182	0.899	0.249	1.406	0.904	2.435			0.2512	0.676		
136	1.1703	1.806	0.502	1.510	3.316	2.196	0.920	0.256	1.444	0.903	2.413			0.2509	0.670		
142	1.2490	1.793	0.498	1.499	3.293	2.196	0.930	0.258	1.490	0.896	2.396			0.2491	0.665		
148	1.3120	1.807	0.502	1.468	3.276	2.230	0.961	0.267	1.507	0.903	2.372			0.2510	0.658		
154	1.3802	1.801	0.500	1.455	3.257	2.237	0.974	0.271	1.542	0.900	2.356			0.2500	0.654		
163	1.4943	1.801	0.500	1.427	3.228	2.262	1.003	0.278	1.588	0.900	2.328	↓	↓	0.2501	0.646		

(1) CORRECTED FOR _____

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

REMARKS:

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. EKC-2
SOIL EAST WINDSOR VARVED CLAY
PROJECT 80142
TESTED BY SAM. DATE 27 Dec 1975
ALL STRESSES IN kg/cm²

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL				77.20	7.78	9.92
PRESHEAR				70.40	7.14	9.86

G_s = _____ TYPE CELL GEO

PRESHEAR
$\bar{\sigma}_{1c}$ = 3.0 t_c = 2 days
$\bar{\sigma}_{3c}$ = 3.0 P.P.R. = — %
$\bar{\sigma}_{oc}$ = 3.0 u_s = 2.00

DURING SHEAR
CONTROLLED STRAIN ✓ STRESS ____
RATE 0.7%/hr
PATH compression loading

PRESHEAR STRESS HISTORY $\bar{C}\bar{K}_oUC$ consolidated to $\bar{\sigma}_{vc}$ = 3.0 kg/cm².

ELAPSED TIME	AXIAL STRAIN, %	$(\sigma_1 - \sigma_3)$ (1)	$\frac{(\bar{\sigma}_1 - \bar{\sigma}_3)}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu (2)	$\frac{\Delta u}{\bar{\sigma}_{1c}}$ (2)	A (3)	q	p̄	$\bar{\sigma}_r$	$\bar{\sigma}_a$	$\frac{q}{\bar{\sigma}_{vc}}$	$\frac{\bar{p}}{\bar{\sigma}_{vc}}$	$\frac{q}{\bar{\sigma}_{vm}}$	$\frac{\bar{p}}{\bar{\sigma}_{vm}}$
0	0	0.965	0.321	2.040	3.005	1.473	0	0	—	0.483	2.523	↑	↑	0.161	0.841	0.161	0.841
14	0.02	1.083	0.360	1.890	2.973	1.573	0.149	0.049	1.267	0.540	2.273			0.180	0.758	0.180	0.758
29	0.17	1.285	0.428	1.750	3.033	1.733	0.291	0.097	0.909	0.642	2.392			0.214	0.797	0.214	0.797
39	0.27	1.333	0.444	1.675	3.008	1.798	0.369	0.123	1.000	0.666	2.336			0.222	0.778	0.222	0.778
72	0.69	1.389	0.462	1.485	2.874	1.935	0.555	0.185	1.309	0.693	2.179			0.231	0.726	0.231	0.726
102	1.09	1.396	0.464	1.397	2.793	1.999	0.643	0.214	1.490	0.696	2.095			0.232	0.698	0.232	0.698
142	1.66	1.394	0.464	1.231	2.626	2.133	0.808	0.268	1.980	0.696	1.968	—	—	0.232	0.656	0.232	0.656
242	3.09	1.366	0.454	1.038	2.404	2.316	1.081	0.333	2.493	0.681	1.892	$\bar{\sigma}_r = \bar{\sigma}_3$	$\bar{\sigma}_a = \bar{\sigma}_1$	0.227	0.631	0.227	0.631
302	3.91	1.376	0.458	0.948	2.324	2.451	1.092	0.363	2.656	0.687	1.636			0.229	0.545	0.229	0.545
422	5.54	1.370	0.456	0.836	2.206	2.639	1.203	0.400	2.770	0.684	1.521			0.228	0.507	0.228	0.507
482	6.36	1.370	0.456	0.798	2.167	2.715	1.242	0.413	3.067	0.684	1.483			0.228	0.494	0.228	0.494
602	8.00	1.347	0.448	0.746	2.093	2.806	1.293	0.430	3.385	0.672	1.419			0.224	0.473	0.224	0.473
722	9.70	1.289	0.429	0.715	2.064	2.862	1.324	0.441	4.080	0.645	1.359			0.215	0.453	0.215	0.453
782	10.54	1.295	0.432	0.702	1.997	2.844	1.337	0.443	4.052	0.648	1.349	↓	↓	0.216	0.449	0.216	0.449

(1) CORRECTED FOR _____

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

REMARKS:

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT. OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. EKC-3
SOIL EAST WINDSOR
PROJECT
TESTED BY Sam DATE 18|Feb|1975.
ALL STRESSES IN kg/cm².

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL				76.72	7.73	
PRESHEAR				71.88	7.35	9.79

G_s = TYPE CELL

PRESHEAR
$\bar{\sigma}_{1c}$ = 2.69 t_c = 3 days
$\bar{\sigma}_{3c}$ = 1.84 P.P.R. = %
$\bar{\sigma}_{ac}$ = u_s = 2.0 kg/cm²

DURING SHEAR
CONTROLLED STRAIN ✓ STRESS
RATE 0.35%/hr
PATH compression.

PRESHEAR STRESS HISTORY

ELAPSED TIME	AXIAL STRAIN, %	$(\bar{\sigma}_1 - \bar{\sigma}_3)$ (1)	$\frac{(\bar{\sigma}_1 - \bar{\sigma}_3)}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu (2)	$\frac{\Delta u}{\bar{\sigma}_{1c}}$ (2)	A (3)	q	$\bar{p}$	σ_r	σ_a	$q/\bar{\sigma}_{vc}$	$\bar{p}/\bar{\sigma}_{vc}$		
	0	0.850	0.316	1.84	2.690	1.462	0	0	-	0.425	2.265			0.158	0.842		
	0.01	0.869	0.323	1.80	2.670	1.483	0.039	0.014		0.434	2.235			0.161	0.831		
	0.07	1.043	0.388	1.644	2.687	1.634	0.196	0.073	1.016	0.522	2.166			0.194	0.805		
	0.40	1.246	0.463	1.338	2.584	1.931	0.502	0.187	1.268	0.623	1.961			0.232	0.729		
	0.73	1.290	0.480	1.189	2.479	2.085	0.651	0.242	1.476	0.645	1.834			0.239	0.682		
	1.06	1.299	0.483	1.089	2.388	2.193	0.751	0.279	1.669	0.650	1.739			0.242	0.646		
	1.40	1.299	0.483	1.017	2.315	2.276	0.823	0.306	1.833	0.649	1.666			0.241	0.619		
	1.85	1.284	0.477	0.950	2.234	2.352	0.890	0.331	2.051	0.642	1.592			0.238	0.592		
	2.65	1.243	0.462	0.865	2.108	2.437	0.975	0.362	2.487	0.621	1.487			0.231	0.553		
	3.34	1.210	0.450	0.811	2.021	2.492	1.029	0.383	2.866	0.605	1.416			0.225	0.526		
	3.91	1.182	0.439	0.769	1.952	2.538	1.070	0.398	3.233	0.591	1.361			0.219	0.506		
	4.93	1.138	0.423	0.718	1.856	2.585	1.122	0.417	3.896	0.569	1.287			0.211	0.478		
	5.85	1.103	0.410	0.682	1.785	2.617	1.158	0.430	4.577	0.551	1.234			0.205	0.459		

(1) CORRECTED FOR

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

REMARKS:

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT. OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. EKC-4

SOIL EAST WINDSOR VARVED CLAY

PROJECT 80142

TESTED BY SAM. DATE 10 Oct 1975

ALL STRESSES IN kg/cm²

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL	58.5			79.38	7.98	9.95
PRESHEAR	48.9		100	71.91	7.32	9.83

G_s = ____ TYPE CELL BEO.

PRESHEAR: $\bar{\sigma}_{1c}$ = 1.800 t_c = 1 day; $\bar{\sigma}_{3c}$ = 1.476 P.P.R. = — %; $\bar{\sigma}_{ac}$ = 1.800 u_s = 2.0

DURING SHEAR: CONTROLLED STRAIN ✓ STRESS ____ RATE 0.7 % / hr. PATH LOADING, compression

PRESHEAR STRESS HISTORY Ko consolidated to $\bar{\sigma}_{vc}$ = 3.6 kg/cm² and rebound to 1.8 kg/cm²

ELAPSED TIME (4)	AXIAL STRAIN, %	$(\bar{\sigma}_1 - \bar{\sigma}_3)$ (1)	$\frac{(\bar{\sigma}_1 - \bar{\sigma}_3)}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1 / \bar{\sigma}_3$	Δu (2)	$\frac{\Delta u}{\bar{\sigma}_{1c}}$ (2)	A (3)	q	$\bar{p}$	$\bar{\sigma}_r$	$\bar{\sigma}_a$	$\frac{q}{\bar{\sigma}_{vc}}$	$\frac{\bar{p}}{\bar{\sigma}_{vc}}$	$\frac{q}{\bar{\sigma}_{vm}}$	$\frac{\bar{p}}{\bar{\sigma}_{vm}}$
0	0	0.324	0.180	1.476	1.800	1.221	0	0	—	0.162	1.638	↑	↑	0.0900	0.910	0.045	0.455
10	0.017	0.467	0.259	1.365	1.832	1.342	0.111	0.062	0.776	0.234	1.598			0.129	0.888	0.065	0.444
36	0.099	0.792	0.440	1.205	1.997	1.657	0.271	0.151	0.579	0.396	1.601			0.220	0.889	0.110	0.447
51	0.1781	0.938	0.521	1.149	2.087	1.816	0.327	0.182	0.533	0.469	1.618			0.260	0.898	0.130	0.449
69	0.293	1.097	0.609	1.082	2.178	2.014	0.394	0.219	0.510	0.548	1.630			0.304	0.906	0.152	0.453
92	0.476	1.239	0.688	1.053	2.292	2.176	0.423	0.235	0.462	0.619	1.673			0.344	0.929	0.172	0.465
116	0.705	1.334	0.741	1.049	2.383	2.271	0.427	0.237	0.422	0.667	1.716			0.370	0.953	0.185	0.477
141	0.972	1.381	0.767	1.066	2.447	2.296	0.410	0.227	0.388	0.690	1.757			0.384	0.976	0.192	0.488
176	1.367	1.429	0.794	1.041	2.470	2.373	0.435	0.242	0.394	0.714	1.755			0.397	0.975	0.198	0.488
212	1.783	1.449	0.805	1.020	2.469	2.421	0.456	0.253	0.405	0.724	1.744			0.403	0.969	0.201	0.484
242	2.144	1.449	0.805	1.015	2.464	2.427	0.461	0.256	0.410	0.724	1.739	$\bar{\sigma}_r = \bar{\sigma}_3$		0.403	0.966	0.201	0.483
302	2.84	1.459	0.810	0.963	2.422	2.515	0.513	0.285	0.452	0.729	1.692		$\bar{\sigma}_a = \bar{\sigma}_1$	0.405	0.940	0.202	0.470
362	3.56	1.444	0.802	0.930	2.374	2.552	0.546	0.303	0.488	0.722	1.652			0.401	0.918	0.200	0.459
482	4.789	1.440	0.800	0.863	2.303	2.668	0.613	0.341	0.549	0.720	1.583			0.400	0.879	0.200	0.439
587	6.23	1.424	0.791	0.814	2.238	2.749	0.662	0.368	0.549	0.712	1.526			0.356	0.848	0.198	0.424
707	7.74	1.410	0.783	0.772	2.182	2.826	0.704	0.391	0.602	0.705	1.477			0.392	0.821	0.196	0.410
857	9.62	1.391	0.773	0.731	2.128	2.887	0.739	0.410	0.648	0.695	1.433			0.386	0.796	0.193	0.398
977	11.08	1.383	0.768	0.713	2.096	2.939	0.763	0.424	0.692	0.691	1.405			0.384	0.780	0.192	0.390
1157	13.25	1.376	0.764	0.695	2.071	2.979	0.781	0.433	0.742	0.688	1.383			0.382	0.768	0.191	0.384
1237	14.24	1.368	0.760	0.677	2.045	3.026	0.799	0.443	0.765	0.684	1.361	↓	↓	0.380	0.756	0.190	0.378

(1) CORRECTED FOR filter paper.

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

(4) Elapsed Time in min.

REMARKS: (1) $\bar{\sigma}_{vm}$ = 3.6 kg/cm².

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. EKC-5
SOIL EAST WINDSOR VARVED CLAY
PROJECT 80142
TESTED BY J.N.L. DATE 30 March 1975
ALL STRESSES IN kg/cm².

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL	56.3			79.24	7.98	9.93
PRESHEAR	—			72.33	7.33	9.863

G_s = ____ TYPE CELL GEO

PRESHEAR: $\bar{\sigma}_{1c}$ = 1.0 kg/cm² t_c = 1 day; $\bar{\sigma}_{3c}$ = 1.0 P.P.R. = ____ %; $\bar{\sigma}_{ac}$ = 1.0 u_B = 2.0

DURING SHEAR: CONTROLLED STRAIN ✓ STRESS ____ RATE 0.7 %/hr PATH LOADING

PRESHEAR STRESS HISTORY Ko consolidation to $\bar{\sigma}_{vm}$ = 4.0 kg/cm² and Rebound to 1.0 kg/cm²

ELAPSED TIME (4)	AXIAL STRAIN, %	$(\bar{\sigma}_1 - \bar{\sigma}_3)$ (1)	$\frac{(\bar{\sigma}_1 - \bar{\sigma}_3)}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu (2)	$\frac{\Delta u}{\bar{\sigma}_{1c}}$ (2)	A (3)	q	$\bar{p}$	$\bar{\sigma}_r$	$\bar{\sigma}_a$	$\frac{q}{\bar{\sigma}_{vc}}$	$\frac{\bar{p}}{\bar{\sigma}_{vc}}$	$\frac{q}{\bar{\sigma}_{vm}}$	$\frac{\bar{p}}{\bar{\sigma}_{vm}}$
0	0	0	0	1.00	1.00	1.000	0	—	—	0	1.000			0	1.000	0	0.250
2	0.012	0.030	0.030	0.977	1.007	1.031	0.023	0.0233	0.773	0.015	0.992			0.0151	0.992	0.004	0.248
6	0.620	0.099	0.099	0.936	1.029	1.106	0.069	0.069	0.706	0.049	0.979			0.0494	0.979	0.012	0.245
12	0.040	0.195	0.195	0.868	1.063	1.224	0.132	0.132	0.676	0.097	0.965			0.097	0.966	0.024	0.242
19	0.073	0.294	0.294	0.811	1.105	1.362	0.189	0.189	0.642	0.147	0.958			0.147	0.958	0.037	0.240
27	0.111	0.376	0.376	0.767	1.143	1.490	0.233	0.233	0.618	0.188	0.956			0.188	0.956	0.047	0.239
32	0.131	0.408	0.408	0.752	1.159	1.542	0.248	0.248	0.608	0.204	0.956			0.204	0.956	0.051	0.239
63	0.345	0.638	0.638	0.664	1.302	1.960	0.336	0.336	0.527	0.319	0.983			0.319	0.983	0.080	0.246
73	0.440	0.702	0.702	0.648	1.351	2.088	0.351	0.351	0.500	0.351	0.999			0.351	0.999	0.088	0.250
88	0.566	0.774	0.774	0.638	1.412	2.212	0.362	0.362	0.467	0.387	1.025			0.387	1.025	0.097	0.256
108	0.743	0.859	0.859	0.630	1.489	2.362	0.369	0.369	0.430	0.429	1.060			0.429	1.059	0.107	0.265
133	0.923	0.947	0.947	0.630	1.577	2.501	0.369	0.370	0.390	0.473	1.104	$\bar{\sigma}_r = \bar{\sigma}_3$	$\bar{\sigma}_a = \bar{\sigma}_1$	0.473	1.104	0.118	0.276
163	1.298	1.043	1.043	0.638	1.681	2.633	0.362	0.362	0.347	0.521	1.159			0.521	1.159	0.130	0.290
225	1.960	1.165	1.165	0.662	1.827	2.761	0.338	0.338	0.291	0.583	1.244			0.583	1.244	0.146	0.311
285	2.620	1.241	1.241	0.687	1.928	2.805	0.313	0.313	0.252	0.621	1.308			0.621	1.308	0.155	0.327
345	3.269	1.284	1.285	0.703	1.987	2.828	0.297	0.297	0.231	0.642	1.345			0.643	1.345	0.161	0.336
405	3.991	1.313	1.313	0.708	2.021	2.854	0.292	0.292	0.222	0.657	1.365			0.657	1.364	0.164	0.341
525	5.476	1.355	1.3552	0.708	2.062	2.914	0.292	0.292	0.215	0.678	1.386			0.678	1.386	0.170	0.347
585	6.221	1.356	1.356	0.710	2.066	2.908	0.289	0.289	0.213	0.678	1.388			0.678	1.389	0.170	0.347
645	6.956	1.351	1.351	0.703	2.053	2.922	0.297	0.297	0.220	0.675	1.378			0.675	1.378	0.169	0.345
762	8.365	1.340	1.340	0.690	2.030	2.943	0.310	0.310	0.231	0.670	1.360			0.670	1.360	0.168	0.340
882	9.829	1.338	1.338	0.682	2.021	2.961	0.318	0.318	0.238	0.669	1.351			0.668	1.351	0.167	0.338
1002	11.273	1.321	1.321	0.672	1.993	2966	0.328	0.328	0.248	0.660	1.332			0.660	1.332	0.165	0.333
1122	12.735	1.305	1.305	0.667	1.972	2.957	0.333	0.333	0.255	0.653	1.319			0.653	1.319	0.163	0.327
1242	14.218	1.285	1.285	0.662	1.947	2.943	0.338	0.338	0.263	0.643	1.304			0.642	1.304	0.161	0.326
1362	15.702	1.239	1.239	0.659	1.898	2.880	0.341	0.341	0.275	0.619	1.278			0.619	1.278	0.155	0.320

(1) CORRECTED FOR ____

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

(4) Elapse Time in Min

REMARKS:

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT. OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. EXRE-1

SOIL EAST WINDSOR VARVED CLAY

PROJECT 80142.

TESTED BY SAM. DATE 26 May 1970

ALL STRESSES IN kg/cm².

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL	56.6			78.95	7.95	9.93
PRESHEAR	47.1			71.16	7.25	9.81

G_s = ____ TYPE CELL CLO

PRESHEAR: $\bar{\sigma}_{ic}$ = 3.615 t_c = 1 day.; $\bar{\sigma}_{3c}$ = 2.448 P.P.R. = — %; $\bar{\sigma}_{oc}$ = 3.615 u_b = 2.00

DURING SHEAR: CONTROLLED STRAIN ✓ STRESS ____ RATE 0.7%/hr PATH EXTENSION UNLOADING

PRESHEAR STRESS HISTORY $\bar{C}K_oU$ CONSOLIDATED TO $\bar{\sigma}_{vc}$ = 3.615 kg/cm².

ELAPSED TIME	AXIAL STRAIN, %	$\sigma_v - \sigma_H$	$\frac{\sigma_v - \sigma_H}{\bar{\sigma}_{vc}}$	$\bar{\sigma}_v$	$\bar{\sigma}_H$	$\bar{\sigma}_H/\bar{\sigma}_v$	Δu (2)	$\frac{\Delta u - \Delta\sigma_3}{\bar{\sigma}_{vc}}$	A (3)	q	p	$\bar{\sigma}_f$	$\bar{\sigma}_o$	$q/\bar{\sigma}_{vc}$	$\bar{p}/\bar{\sigma}_{vc}$	$q/\bar{\sigma}_{vm}$	$\bar{p}/\bar{\sigma}_{vm}$
0	0	1.168	0.324	3.615	2.448	0.677	0	0	—	0.584	3.031	2.448	3.615	0.162	0.838	0.162	0.838
10	0.007	1.048	0.308	3.540	2.492	0.704	-0.044	0.021	0.633	0.524	3.01	2.492	3.540	0.145	0.834	0.145	0.834
38	0.109	0.566	0.156	3.291	2.725	0.828	-0.277	0.090	0.540	0.283	3.00	2.725	3.291	0.078	0.832	0.078	0.832
52	0.200	0.342	0.094	3.130	2.788	0.841	-0.340	0.134	0.588	0.171	2.959	2.788	3.130	0.047	0.818	0.047	0.818
90	0.470	-0.034	-0.016	2.791	2.825	1.010	-0.377	0.228	0.686	-0.017	2.808	2.825	2.791	-0.005	0.777	-0.005	0.777
112	0.653	-0.238	-0.066	2.588	2.826	1.090	-0.378	0.284	0.713	-0.119	2.707	2.826	2.588	-0.033	0.749	-0.033	0.749
162	1.13	-0.595	-0.166	2.206	2.800	1.269	-0.353	0.390	0.800	-0.298	2.503	2.800	2.206	-0.083	0.692	-0.083	0.692
222	1.86	-0.885	-0.244	1.833	2.717	1.482	-0.269	0.493	0.869	-0.443	2.275	2.717	1.833	-0.122	0.629	-0.122	0.629
292	2.79	-1.085	-0.300	1.506	2.591	1.720	-0.144	0.583	0.939	-0.543	2.048	2.591	1.506	-0.150	0.566	-0.150	0.566
442	4.79	-1.303	-0.360	1.107	2.410	2.170	0.037	0.694	1.015	-0.652	1.758	2.410	1.107	-0.180	0.486	-0.180	0.486
502	5.625	-1.362	-0.376	0.997	2.359	2.360	0.089	0.724	1.035	-0.681	1.678	2.359	0.997	-0.188	0.464	-0.188	0.464
592	6.90	-1.443	-0.398	0.868	2.310	2.660	0.137	0.760	1.052	-0.722	1.589	2.310	0.868	-0.199	0.439	-0.199	0.439
653	7.78	-1.471	-0.407	0.817	2.287	2.799	0.160	0.774	1.061	-0.736	1.552	2.287	0.817	-0.203	0.429	-0.203	0.429
713	8.60	-1.500	-0.415	0.763	2.263	2.965	0.185	0.789	1.069	-0.750	1.513	2.263	0.763	-0.208	0.419	-0.208	0.419
773	9.41	-1.544	-0.426	0.720	2.264	3.144	0.184	0.801	1.068.	-0.772	1.492	2.264	0.720	-0.213	0.412	-0.213	0.412

(1) CORRECTED FOR ____

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_3}{\Delta\sigma_1 - \Delta\sigma_3}$ FOR EXTENSION TESTS

REMARKS:

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT. OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. ERE-2

SOIL EAST WINDSOR VARVED CLAY

PROJECT 80142

TESTED BY SAM. DATE 2 Oct. 1975

ALL STRESSES IN kg/cm².

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL	56.4			78.95	7.95	9.93
PRESHEAR	46.3			72.09	7.29	9.88

θ_0 = ____ TYPE CELL CLO.

PRESHEAR STRESS HISTORY CK$_0$UE consolidated to $\bar{\sigma}_{VC}$ = 3.613 kg/cm².

PRESHEAR: σ_{1c} = 3.613, t_c = 2 days; σ_{3c} = 2.448, P.P.R. = ____ %; $\bar{\sigma}_{oc}$ = 3.613, u_0 = 2.00

DURING SHEAR: CONTROLLED STRAIN ✓ STRESS ____; RATE 0.7%/hr.; PATH EXTENSION UNLOADING

ELAPSED TIME	AXIAL STRAIN, %	$\sigma_v - \sigma_H$	$\frac{\sigma_v - \sigma_H}{\bar{\sigma}_{VC}}$	$\bar{\sigma}_v$	$\bar{\sigma}_H$	$\bar{\sigma}_H / \bar{\sigma}_v$	Δu (2)	$\frac{\Delta u - \Delta\sigma_3}{\bar{\sigma}_{VC}}$	A (3)	q	β	σ_f	σ_o	$q/\bar{\sigma}_{VC}$	$\bar{p}/\bar{\sigma}_{VC}$	$q/\bar{\sigma}_{VM}$	$\bar{p}/\bar{\sigma}_{VM}$
0	0	1.165	0.322	3.613	2.448	0.677	0	0	—	0.583	3.030	2.448	3.613	0.161	0.838	0.161	0.838
24	0.036	0.820	0.226	3.490	2.670	0.765	-0.222	0.034	0.655	0.410	3.089	2.670	3.490	0.113	0.852	0.113	0.852
56	0.137	0.349	0.096	3.193	2.844	0.890	-0.396	0.116	0.515	0.175	3.028	2.844	3.193	0.048	0.835	0.048	0.835
85	0.331	0.014	0.004	2.926	2.912	0.995	-0.464	0.190	0.597	0.007	2.919	2.912	2.926	0.002	0.807	0.002	0.807
121	0.673	-0.324	-0.090	2.589	2.913	1.125	-0.465	0.283	0.688	-0.162	2.751	2.913	2.589	-0.045	0.761	-0.045	0.761
144	1.612	-0.795	-0.220	1.972	2.767	1.403	-0.319	0.454	0.837	-0.398	2.369	2.767	1.972	-0.110	0.655	-0.110	0.655
174	1.98	-0.907	-0.252	1.801	2.708	1.504	-0.260	0.502	0.875	-0.454	2.254	2.708	1.801	-0.126	0.624	-0.126	0.624
234	2.742	-1.052	-0.292	1.549	2.601	1.679	-0.153	0.571	0.931	-0.526	2.075	2.601	1.549	-0.146	0.574	-0.146	0.574
294	3.544	-1.145	-0.316	1.364	2.509	1.839	-0.062	0.622	0.973	-0.573	1.936	2.509	1.364	-0.158	0.535	-0.158	0.535
354	4.367	-1.216	-0.336	1.217	2.433	1.999	+0.015	0.663	1.006	-0.608	1.825	2.433	1.217	-0.168	0.505	-0.168	0.505
414	5.20	-1.279	-0.354	1.092	2.371	2.171	0.077	0.697	1.032	-0.640	1.731	2.371	1.092	-0.177	0.479	-0.177	0.479
474	6.03	-1.336	-0.368	0.989	2.325	2.350	0.123	0.726	1.049	-0.668	1.657	2.325	0.989	-0.184	0.458	-0.184	0.458
534	6.86	-1.372	-0.378	0.915	2.287	2.499	0.161	0.746	1.064	-0.686	1.601	2.287	0.915	-0.189	0.443	-0.189	0.443
594	7.68	-1.403	-0.388	0.858	2.261	2.635	0.187	0.763	1.078	-0.702	1.559	2.261	0.858	-0.194	0.432	-0.194	0.432
654	8.49	-1.441	-0.398	0.800	2.241	2.801	0.207	0.778	1.079	-0.721	1.520	2.241	0.800	-0.199	0.421	-0.199	0.421
714	9.34	-1.484	-0.410	0.744	2.228	2.994	0.220	0.794	1.083	-0.742	1.486	2.228	0.744	-0.205	0.411	-0.205	0.411

(1) CORRECTED FOR ____

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_3}{\Delta\sigma_1 - \Delta\sigma_3}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_3}{\Delta\sigma_3 - \Delta\sigma_1}$ FOR EXTENSION TESTS

REMARKS:

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT. OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. EKE-3

SOIL EAST WINDSOR VARVED CLAY

PROJECT 80142

TESTED BY SAM. DATE 19 Oct 1975.

ALL STRESSES IN kg/cm²

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL	58.7			78.95	7.98	9.89
PRESHEAR	50.1			73.86	7.51	9.84

G_s = ____ TYPE CELL CLO

PRESHEAR: $\bar{\sigma}_{1c}$ = 0.900 t_c = 1 day; $\bar{\sigma}_{3c}$ = 0.900 P.P.R. = — %; $\bar{\sigma}_{oc}$ = 0.900 u_s = 2.00

DURING SHEAR: CONTROLLED STRAIN ✓ STRESS ____ RATE 0.7%/hr PATH EXTENSION UNLOADING

PRESHEAR STRESS HISTORY CROVE at $\bar{\sigma}_{vc}$ = 3.6 kg/cm² and rebound to 0.9 kg/cm²

ELAPSED TIME	AXIAL STRAIN, %	$\sigma_v - \sigma_h$	$\frac{\sigma_v - \sigma_h}{\bar{\sigma}_{vc}}$	$\bar{\sigma}_v$	$\bar{\sigma}_h$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu (2)	$\frac{\Delta u - \Delta \sigma_3}{\bar{\sigma}_{vc}}$	A (3)	q	p̄	$\bar{\sigma}_r$	$\bar{\sigma}_a$	$\frac{q}{\bar{\sigma}_{vc}}$	$\frac{\bar{p}}{\bar{\sigma}_{vc}}$	$\frac{q}{\bar{\sigma}_{vm}}$	$\frac{\bar{p}}{\bar{\sigma}_{vm}}$
0	0	0	0	0.900	0.900	1.000	0	0	—	0	0.900	0.900	0.900	0	1.000	0	0.250
10	0.052	-0.092	0.102	0.874	0.966	1.106	-0.066	0.028	0.277	-0.046	0.920	0.966	0.874	-0.051	1.023	-0.013	0.256
28	0.201	-0.254	0.282	0.843	1.097	1.301	-0.197	0.064	0.226	-0.127	0.969	1.097	0.843	-0.141	1.077	-0.035	0.269
56	0.481	-0.411	0.456	0.807	1.218	1.509	-0.318	0.103	0.226	-0.206	1.013	1.218	0.807	-0.228	1.125	-0.057	0.281
116	1.199	-0.609	0.677	0.742	1.351	1.821	-0.451	0.175	0.259	-0.305	1.047	1.351	0.742	-0.338	1.163	-0.085	0.291
176	1.956	-0.719	0.798	0.692	1.411	2.039	-0.511	0.231	0.289	-0.360	1.052	1.411	0.692	-0.399	1.168	-0.099	0.292
229	2.728	-0.798	0.886	0.646	1.444	2.235	-0.544	0.282	0.318	-0.399	1.045	1.444	0.646	-0.443	1.161	-0.100	0.290
329	4.08	-0.893	0.992	0.578	1.471	2.545	-0.571	0.357	0.359	-0.447	1.024	1.471	0.578	-0.495	1.138	-0.124	0.285
459	5.80	-0.966	1.074	0.532	1.498	2.816	-0.598	0.409	0.381	-0.483	1.015	1.498	0.532	-0.537	1.128	-0.134	0.282
579	7.45	-1.033	1.148	0.492	1.525	3.100	-0.625	0.453	0.395	-0.517	1.008	1.525	0.492	-0.574	1.121	-0.143	0.280
699	9.03	-1.069	1.187	0.485	1.554	3.204	-0.654	0.461	0.388	-0.535	1.019	1.554	0.485	-0.594	1.133	-0.148	0.283
819	11.39	-1.154	1.282	0.431	1.585	3.677	-0.685	0.521	0.406	-0.577	1.008	1.585	0.431	-0.641	1.120	-0.160	0.280
882	11.50	-1.164	1.293	0.447	1.611	3.604	-0.700	0.515	0.398	-0.582	1.029	1.611	0.447	-0.646	1.143	-0.161	0.285
938	12.28	-1.171	1.301	0.438	1.609	3.670	-0.709	0.513	0.394	-0.586	1.024	1.609	0.438	-0.650	1.137	-0.162	0.284

(1) CORRECTED FOR ____

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

REMARKS:

SECTION D.2-3

TABULATION OF TEST DATA FROM $\overline{CK_oUC}$, $\overline{CIUC}$ AND $\overline{CK_oUE}$

TESTS ON RECOMPRESSION EAST WINDSOR SAMPLES

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. ERKC-1
SOIL EAST WINDSOR VARVED CLAY
PROJECT 80142
TESTED BY SAM. DATE
ALL STRESSES IN kg/cm²

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL	57.4			79.12	7.98	9.92
PRESHEAR	54.9			76.75	7.82	9.81

G_s = TYPE CELL GEO.

PRESHEAR: $\bar{\sigma}_{1c}$ = 1.153, t_c = 1 day; $\bar{\sigma}_{3c}$ = 0.943, P.P.R. = %; $\bar{\sigma}_{ac}$ = , u_B = 2.0

DURING SHEAR: CONTROLLED STRAIN ✓ STRESS; RATE 0.7%/hr; PATH Compression loading.

PRESHEAR STRESS HISTORY Anisotropic consolidated to $\bar{\sigma}_{1c}$ = 1.153 kg/cm²

ELAPSED TIME	AXIAL STRAIN, %	$(\bar{\sigma}_1 - \bar{\sigma}_3)$ (1)	$\frac{(\bar{\sigma}_1 - \bar{\sigma}_3)}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu (2)	$\frac{\Delta u}{\bar{\sigma}_{1c}}$ (2)	A (3)	q	$\bar{p}$	$\bar{\sigma}_r$	$\bar{\sigma}_c$	$\frac{q}{\sigma_{vc}}$	$\frac{\bar{p}}{\bar{\sigma}}$	$\frac{q}{\sigma_{vm}}$	$\frac{\bar{p}}{\sigma_{vm}}$
0	0	0.210	0.183	0.943	1.153	1.222	0	0	–	0.210	1.048			0.091	0.911	0.046	0.456
27	0.090	0.497	0.432	0.730	1.227	1.681	0.213	0.185	0.428	0.447	0.979			0.216	0.851	0.108	0.426
60	0.324	0.732	0.637	0.525	1.257	2.394	0.418	0.363	0.571	0.732	0.891			0.318	0.774	0.159	0.387
90	0.596	0.857	0.745	0.431	1.288	2.988	0.512	0.445	0.597	0.857	0.860			0.373	0.747	0.186	0.374
120	0.909	0.965	0.839	0.400	1.365	3.413	0.543	0.472	0.563	0.965	0.883			0.420	0.767	0.210	0.384
184	1.603	0.990	0.861	0.392	1.382	3.526	0.551	0.479	0.556	0.990	0.887			0.430	0.771	0.215	0.386
244	2.226	0.985	0.856	0.403	1.388	3.444	0.541	0.470	0.548	0.985	0.896	Same		0.428	0.778	0.214	0.389
304	2.946	0.944	0.820	0.423	1.367	3.232	0.520	0.452	0.550	0.944	0.895	as $\bar{\sigma}_3$	same as	0.410	0.778	0.205	0.389
364	3.618	0.923	0.802	0.426	1.349	3.167	0.517	0.450	0.560	0.923	0.888		$\bar{\sigma}_1$	0.401	0.771	0.200	0.386
447	4.570	0.894	0.777	0.421	1.315	3.124	0.522	0.453	0.583	0.894	0.894			0.388	0.754	0.194	0.377
507	5.267	0.867	0.754	0.436	1.303	2.988	0.507	0.441	0.585	0.867	0.870			0.377	0.756	0.188	0.378
567	5.986	0.852	0.741	0.436	1.288	2.954	0.507	0.447	0.595	0.852	0.862			0.370	0.749	0.185	0.375
627	6.706	0.823	0.716	0.436	1.259	2.888	0.507	0.441	0.616	0.823	0.848			0.358	0.737	0.179	0.369
657	7.050	0.820	0.713	0.429	1.249	2.911	0.514	0.447	0.626	0.820	0.839			0.356	0.729	0.178	0.364

(1) CORRECTED FOR

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

REMARKS:

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. ERIC-1-1
SOIL EAST WINDSOR VARVE CLAY
PROJECT 80142
TESTED BY SAM. DATE
ALL STRESSES IN kg/cm².

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL				77.65	7.89	9.82
PRESHEAR				76.43	7.77	9.84

G_s = TYPE CELL GEO

PRESHEAR: $\bar{\sigma}_{ic}$ = 0.60, $\bar{\sigma}_{3c}$ = 0.60, $\bar{\sigma}_{ac}$ = 0.60, t_c = 1 day, P.P.R = %, u_B = 2.0

DURING SHEAR: CONTROLLED STRAIN ✓ STRESS, RATE 0.35 %/hr., PATH Compression loading.

PRESHEAR STRESS HISTORY $\bar{c}\bar{I}UC$ test $\bar{\sigma}_{vc}$ = 0.60 kg/cm².

ELAPSED TIME	AXIAL STRAIN, %	$(\bar{\sigma}_1 - \bar{\sigma}_3)$ (1)	$\frac{(\bar{\sigma}_1 - \bar{\sigma}_3)}{\bar{\sigma}_{ic}}$	$\bar{\sigma}_3$	$\bar{\sigma}_r$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu (2)	$\frac{\Delta u}{\bar{\sigma}_{ic}}$ (2)	A (3)	q	$\bar{p}$	$\bar{\sigma}_r$	$\bar{\sigma}_a$	$\frac{q}{\bar{\sigma}_{vc}}$	$\frac{\bar{p}}{\bar{\sigma}_{vc}}$	$\frac{q}{\bar{\sigma}_{vm}}$	$\frac{\bar{p}}{\bar{\sigma}_{vm}}$
	0	0	0	0.600	0.600	1.000	0	0	-	0	0.600	↑	↑	0	1.000	0	0.250
	0.10	0.140	0.059	0.460	0.600	1.306	0.139	0.232	0.993	0.070	0.530			0.117	0.883	0.029	0.221
	0.22	0.236	0.392	0.396	0.632	1.597	0.204	0.340	0.864	0.118	0.514			0.196	0.857	0.049	0.214
	0.46	0.416	0.694	0.283	0.699	2.469	0.317	0.528	0.762	0.208	0.491			0.347	0.818	0.087	0.205
	1.13	0.689	1.148	0.175	0.864	4.930	0.425	0.708	0.617	0.344	0.520			0.574	0.866	0.144	0.216
	1.82	0.763	1.272	0.200	0.963	4.810	0.399	0.665	0.523	0.381	0.582			0.636	0.969	0.159	0.242
	2.24	0.767	1.278	0.218	0.985	4.520	0.381	0.635	0.497	0.383	0.602	same	same	0.639	1.003	0.160	0.251
	3.26	0.764	1.272	0.244	1.008	4.130	0.356	0.593	0.466	0.382	0.626	=	as $\bar{\sigma}_1$	0.636	1.043	0.159	0.261
	4.33	0.756	1.260	0.247	1.003	4.060	0.353	0.588	0.467	0.378	0.625	as $\bar{\sigma}_3$		0.630	1.042	0.158	0.260
	5.40	0.753	1.254	0.262	1.015	3.875	0.338	0.563	0.449	0.376	0.639			0.627	1.064	0.157	0.266
	6.94	0.749	1.250	0.270	1.019	3.774	0.329	0.548	0.439	0.375	0.645			0.625	1.074	0.156	0.269
	7.44	0.747	1.246	0.278	1.025	3.686	0.322	0.537	0.431	0.374	0.651			0.623	1.086	0.156	0.271
	9.42	0.740	1.234	0.272	1.012	3.721	0.327	0.600	0.442	0.370	0.647			0.617	1.078	0.154	0.269
	11.95	0.736	1.226	0.270	1.006	3.726	0.329	0.548	0.447	0.368	0.638			0.613	1.063	0.153	0.266
	14.46	0.711	1.186	0.281	0.992	3.530	0.319	0.532	0.449	0.356	0.637	↓	↓	0.593	1.061	0.148	0.265

(1) CORRECTED FOR

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

REMARKS:

$\overline{CIUC}$ 45° VARVE SAMPLE

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. ERIC-2-1
SOIL EAST WINDSOR VARVE CLAY
PROJECT 80142
TESTED BY SAM DATE 18 Dec 1975
ALL STRESSES IN kg/cm²

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL				79.13	7.97	9.93
PRESHEAR				77.58	7.85	9.87

G_s = ____ TYPE CELL GEO

PRESHEAR: $\bar{\sigma}_{1c}$ = 0.60, t_c = 1 day; $\bar{\sigma}_{3c}$ = 0.60, P.P.R. = — %; $\bar{\sigma}_{dc}$ = —, u_B = 2.0

DURING SHEAR: CONTROLLED STRAIN ✓ STRESS ____; RATE 0.35%/hr; PATH compression loading.

PRESHEAR STRESS HISTORY CIUC, at OCR = 4.0, consolidated below max. past pressure

ELAPSED TIME	AXIAL STRAIN, %	$(\bar{\sigma}_1 - \bar{\sigma}_3)$ (1)	$\frac{(\bar{\sigma}_1 - \bar{\sigma}_3)}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu (2)	$\frac{\Delta u}{\bar{\sigma}_{1c}}$	A (3)	q	$\bar{p}$	$\bar{\sigma}_r$	$\bar{\sigma}_z$	$q/\bar{\sigma}_{VC}$	$\bar{p}/\bar{\sigma}_{VC}$	$q/\bar{\sigma}_{VM}$	$\bar{p}/\bar{\sigma}_{VM}$
0	0	0	0	0.600	0.600	1.00	0	0	—	0	0.600	↑	↑	0	1.000	0	0.250
24	0.047	0.124	0.207	0.538	0.662	1.230	0.062	0.103	0.500	0.124	0.600			0.103	1.000	0.026	0.250
49	0.141	0.221	0.368	0.474	0.695	1.466	0.126	0.210	0.570	0.221	0.585			0.184	0.975	0.046	0.244
119	0.454	0.379	0.632	0.378	0.757	2.003	0.222	0.370	0.586	0.379	0.568			0.316	0.947	0.079	0.237
239	1.080	0.491	0.818	0.314	0.805	2.564	0.286	0.477	0.582	0.491	0.559	$\bar{\sigma}_r = \bar{\sigma}_3$	$\bar{\sigma}_q = \bar{\sigma}_1$	0.409	0.932	0.102	0.233
359	1.750	0.523	0.872	0.288	0.811	2.816	0.312	0.520	0.597	0.523	0.550			0.436	0.917	0.109	0.229
479	2.490	0.543	0.905	0.286	0.829	2.899	0.314	0.523	0.578	0.543	0.566			0.453	0.943	0.113	0.236
599	3.21	0.555	0.925	0.291	0.846	2.907	0.309	0.515	0.557	0.555	0.568			0.463	0.947	0.116	0.237
719	3.92	0.566	0.943	0.293	0.859	2.937	0.307	0.512	0.542	0.566	0.576			0.472	0.960	0.118	0.240
839	4.63	0.568	0.947	0.299	0.867	2.900	0.301	0.502	0.530	0.568	0.583			0.473	0.972	0.118	0.243
959	5.41	0.457	0.762	0.286	0.743	2.598	0.314	0.523	0.687	0.457	0.514			0.381	0.857	0.095	0.214
1004	5.61	0.546	0.910	0.273	0.819	3.008	0.327	0.545	0.599	0.546	0.546			0.455	0.910	0.114	0.218
1199	6.72	0.591	0.985	0.273	0.864	3.165	0.327	0.545	0.553	0.591	0.568			0.493	0.947	0.123	0.237
1367	7.70	0.580	0.967	0.286	0.866	3.028	0.314	0.523	0.541	0.580	0.576	↓	↓	0.483	0.960	0.121	0.240
		↑															

Note failure plane already developed.
Double failure plane, the second one is in the process of forming.

(1) CORRECTED FOR ____

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

REMARKS:

$\bar{C}IUC$ 90° sample, OCR = 2.0 CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. ERIC-3-1

SOIL EAST WINDSOR VARVED CLAY

PROJECT 80142

TESTED BY SAM. DATE

ALL STRESSES IN kg/cm²

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL	57.8			75.37	7.67	9.83
PRESHEAR	53.3			72.26	7.49	9.64

G_s = TYPE CE GEO

PRESHEAR: $\bar{\sigma}_{1c}$ = 1.2, t_c = 1 day; $\bar{\sigma}_{3c}$ = 1.2, P.P.R. = —; $\bar{\sigma}_{ac}$ = —, u_b = 2.0

DURING SHEAR: CONTROLLED STRAIN ✓ STRESS; RATE = 0.7% /hr.; PATH Compression loading

PRESHEAR STRESS HISTORY Isotropic consolidated to $\bar{\sigma}_{vc}$ = 1.2 kg/cm², to obtain OCR = 2.0.

ELAPSED TIME (4)	AXIAL STRAIN, %	$(\bar{\sigma}_1 - \bar{\sigma}_3)$ (1)	$\frac{(\bar{\sigma}_1 - \bar{\sigma}_3)}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu (2)	$\frac{\Delta u}{\bar{\sigma}_{1c}}$ (2)	A (3)	q	$\bar{p}$	$\bar{\sigma}_r$	$\bar{\sigma}_a$	$\frac{q_f}{\bar{\sigma}_{vc}}$	$\frac{\bar{p}}{\bar{\sigma}_{vc}}$	$\frac{q_f}{\bar{\sigma}_{vm}}$	$\frac{\bar{p}}{\bar{\sigma}_{vm}}$
0	0	0	0	1.200	1.200	1.000	0	0	—	0	1.200	↑	↑	0	1.00	0	0.500
55	0.48	0.439	0.366	0.968	1.406	1.453	0.232	0.193	0.529	0.219	1.188			0.183	0.989	0.092	0.495
115	1.17	0.666	0554	0.761	1.427	1.875	0.438	0.365	0.658	0.333	1.094			0.277	0.911	0.166	0.456
185	1.98	0.796	0.664	0.576	1.372	2.382	0.624	0.520	0.784	0.398	0.974			0.332	0.811	0.173	0.406
255	2.92	0.828	0.690	0.481	1.308	2.719	0.719	0.599	0.868	0.414	0.894			0.345	0.746	0.169	0.373
325	3.86	0.811	0.674	0.429	1.299	2.890	0.771	0.643	0.951	0.405	0.864			0.337	0.720	0.169	0.360
445	5.46	0.804	0.670	0.385	1.189	3.088	0.815	0.679	1.014	0.402	0.787			0.335	0.656	0.168	0.328
523	6.47	0.800	0.666	0.372	1.172	3.150	0.827	0.689	1.034	0.400	0.772	$\bar{\sigma}_r = \bar{\sigma}_3$		0.333	0.643	0.167	0.322
583	7.26	0.805	0.670	0.367	1.172	3.193	0.833	0.694	1.035	0.402	0.769		$\bar{\sigma}_a = \bar{\sigma}_1$	0.335	0.641	0.169	0.321
703	8.80	0.809	0.674	0.361	1.171	3.243	0.838	0.698	1.036	0.404	0.766			0.337	0.638	0.169	0.319
763	9.60	0.808	0.674	0.361	1.169	3.238	0.838	0.698	1.037	0.404	0.765			0.337	0.637	0.169	0.319
843	10.70	0.812	0.676	0.367	1.179	3.212	0.833	0.694	1.026	0.406	0.773			0.338	0.644	0.169	0.322
984	12.62	0.807	0.672	0.369	1.176	3.188	0.830	0.692	1.028	0.403	0.772			0.336	0.643	0.168	0.322
1104	14.26	0.771	0.642	0.375	1.146	3.056	0.825	0.688	1.070	0.385	0.760			0.321	0.634	0.161	0.317
1164	15.09	0.750	0.624	0.377	1.127	2.990	0.822	0.685	1.096	0.375	0.752			0.312	0.626	0.156	0.313
1183	15.33	0.732	0.610	0.383	1.114	2.909	0.818	0.690	1.117	0.366	0.748	↓	↓	0.305	0.624	0.153	0.312

(1) CORRECTED FOR friction.

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

REMARKS (4) Elapsed time in min.

CIUC 90° sample, OCR = 4.0. CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT. OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. ERIC-3-2
SOIL EAST WINDSOR VARVED CLAY
PROJECT 80142
TESTED BY SAM. DATE 30 Nov. 1975
ALL STRESSES IN kg/cm².

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL				79.4	8.08	9.83
PRESHEAR			100	76.9	7.82	9.83

G_s = TYPE CELL GEO

PRESHEAR: $\bar{\sigma}_{1c}$ = 0.60, t_c = 1 day; $\bar{\sigma}_{3c}$ = 0.60, P.P.R. = – %; $\bar{\sigma}_{oc}$ = 0.60, u_B = 2.0

DURING SHEAR: CONTROLLED STRAIN ✓ STRESS ____; RATE 0.35%/hr.; PATH Compression loading

PRESHEAR STRESS HISTORY Isotropic Consolidation to $\bar{\sigma}_{vc}$ = 0.60 kg/cm² to obtain OCR = 4.0.

ELAPSED TIME	AXIAL STRAIN, %	$(\bar{\sigma}_1 - \bar{\sigma}_3)$ (1)	$\frac{(\bar{\sigma}_1 - \bar{\sigma}_3)}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu (2)	$\frac{\Delta u}{\bar{\sigma}_{1c}}$ (2)	A (3)	q	$\bar{p}$	$\bar{\sigma}_r$	$\bar{\sigma}_a$	$\frac{q}{\bar{\sigma}_{vc}}$	$\frac{\bar{p}}{\bar{\sigma}_{vc}}$	$\frac{q}{\bar{\sigma}_{vm}}$	$\frac{\bar{p}}{\bar{\sigma}_{vm}}$
0	0	0	0	0.600	0.600	1.00	0	0	–	0	0.600	↑	↑	0	1.000	0	0.250
26	0.13	0.1063	0.105	0.574	0.637	1.100	0.1026	0.065	0.413	0.032	0.606			0.053	1.009	0.026	0.252
156	0.64	0.1461	0.768	0.378	0.839	2.200	0.1222	0.370	0.482	0.231	0.609			0.384	1.014	0.096	0.254
236	1.08	0.1513	0.855	0.319	0.832	2.608	0.1281	0.468	0.548	0.257	0.576			0.428	0.956	0.107	0.240
266	1.25	0.1516	0.860	0.319	0.835	2.618	0.1281	0.468	0.548	0.258	0.577			0.430	0.962	0.108	0.240
386	1.96	0.1526	0.877	0.275	0.801	2.913	0.1325	0.542	0.618	0.263	0.538			0.438	0.897	0.110	0.224
546	2.93	0.1539	0.898	0.267	0.806	3.019	0.1333	0.555	0.618	0.270	0.537			0.449	0.894	0.112	0.224
657	3.60	0.1541	0.902	0.272	0.813	2.989	0.1328	0.547	0.606	0.271	0.543			0.451	0.904	0.113	0.226
777	4.31	0.1547	0.912	0.270	0.817	3.026	0.1330	0.550	0.603	0.274	0.544	$\bar{\sigma}_r = \bar{\sigma}_3$		0.456	0.906	0.114	0.226
1017	5.73	0.1562	0.937	0.288	0.850	2.951	0.1312	0.520	0.555	0.281	0.569		$\bar{\sigma}_a = \bar{\sigma}_1$	0.468	0.948	0.117	0.237
1197	6.79	0.1568	0.947	0.288	0.856	2.972	0.1312	0.520	0.549	0.284	0.572			0.473	0.953	0.118	0.238
1347	7.67	0.1570	0.950	0.309	0.879	2.845	0.1291	0.485	0.511	0.285	0.594			0.475	0.990	0.119	0.248
1545	8.83	0.1576	0.960	0.301	0.877	2.914	0.1299	0.498	0.519	0.288	0.589			0.480	0.982	0.120	0.245
1729	9.92	0.1567	0.945	0.309	0.868	2.883	0.1299	0.498	0.527	0.284	0.585			0.472	0.974	0.118	0.243
2029	11.75	0.1549	0.915	0.296	0.845	2.855	0.1304	0.507	0.554	0.275	0.571			0.457	0.950	0.114	0.238
2569	14.93	0.1536	0.884	0.301	0.831	2.761	0.1299	0.498	0.564	0.265	0.566			0.442	0.943	0.110	0.236
2749	15.98	0.1529	0.882	0.296	0.825	2.787	0.1304	0.506	0.574	0.265	0.561	↓	↓	0.441	0.934	0.110	0.233

(1) CORRECTED FOR Friction.

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

REMARKS:

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT. OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. ERKE-1

SOIL EAST WINDSOR VARVED CLAY

PROJECT 80142

TESTED BY _____ DATE _____

ALL STRESSES IN kg/cm²

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL				77.20	7.98	9.93
PRESHEAR				74.98	7.81	9.60

G_s = _____ TYPE CELL _____

PRESHEAR: $\bar{\sigma}_{1c}$ = 1.206 t_c = 1 day; $\bar{\sigma}_{3c}$ = 1.010 P.P.R. = — %; $\bar{\sigma}_{oc}$ = 1.206 u_B = 2.0

DURING SHEAR: CONTROLLED STRAIN ✓ STRESS _____ RATE 0.3%/hr PATH EXTENSION UNLOADING

PRESHEAR STRESS HISTORY Anisotropic Consolidated to $\bar{\sigma}_{vc}$ = 1.20 kg/cm².

ELAPSED TIME	AXIAL STRAIN, %	$\sigma_v - \sigma_h$	$\frac{\sigma_v - \sigma_h}{\bar{\sigma}_{vc}}$	$\bar{\sigma}_v$	$\bar{\sigma}_h$	$\bar{\sigma}_h / \bar{\sigma}_v$	Δu (2)	$\frac{\Delta u - \Delta \sigma_3}{\bar{\sigma}_{vc}}$	A (3)	q	p̄	$\bar{\sigma}_r$	$\bar{\sigma}_a$	$\frac{q}{\bar{\sigma}_{vc}}$	$\frac{\bar{p}}{\bar{\sigma}_{vc}}$	$\frac{q}{\bar{\sigma}_{vm}}$	$\frac{\bar{p}}{\bar{\sigma}_{vm}}$
	0	0.196	0.163	1.206	1.010	0.837	0	0	—	0.098	1.108	1.010	1.206	0.082	0.923	0.041	0.461
	0.02	-0.044	-0.037	1.036	1.080	1.043	-0.070	0.142	0.708	-0.022	1.058	1.080	1.036	-0.018	0.882	-0.009	0.440
	0.07	-0.209	-0.175	0.933	1.429	1.224	-0.133	0.227	0.673	-0.105	1.038	1.429	0.933	-0.087	0.865	-0.0437	0.432
	0.32	-0.506	-0.422	0.709	1.215	1.714	-0.1206	0.414	0.707	-0.253	0.962	1.215	0.709	-0.210	0.802	-0.105	0.401
	0.76	-0.689	-0.574	0.525	1.214	2.312	-0.204	0.567	0.769	-0.344	0.869	1.214	0.525	-0.287	0.725	-0.144	0.362
	1.25	-0.773	-0.644	0.406	1.179	2.904	-0.169	0.466	0.826	-0.387	0.793	1.179	0.406	-0.322	0.660	-0.161	0.330
	2.48	-0.835	-0.696	0.271	1.106	4.080	-0.096	0.179	0.906	-0.417	0.688	1.106	0.271	-0.347	0.574	-0.173	0.287
	3.16	-0.851	-0.709	0.228	1.079	4.734	-0.069	0.815	0.934	-0.426	0.654	1.079	0.228	-0.355	0.545	-0.177	0.272
	3.86	-0.854	-0.711	0.203	1.048	5.170	-0.048	0.835	0.955	-0.427	0.625	1.048	0.203	-0.355	0.521	-0.177	0.261
	4.59	-0.801	-0.668	0.231	1.033	4.450	-0.033	0.804	0.967	-0.400	0.632	1.033	0.231	-0.334	0.527	-0.167	0.263
	5.32	-0.786	-0.655	0.231	1.018	4.395	-0.018	0.804	0.982	-0.393	0.624	1.018	0.231	-0.327	0.520	-0.163	0.260
	6.04	-0.793	-0.660	0.220	1.013	4.605	-0.003	0.823	0.997	-0.396	0.617	1.013	0.220	-0.330	0.512	-0.165	0.257

(1) CORRECTED FOR _____

(2) Δu FOR $\Delta \sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta \sigma_r}{\Delta \sigma_a - \Delta \sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta \sigma_a}{\Delta \sigma_r - \Delta \sigma_a}$ FOR EXTENSION TESTS

REMARKS:

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT. OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. ERIE-1-1
SOIL EAST WINDSOR VARVED CLAY
PROJECT 80142
TESTED BY SAM. DATE 2 Dec 1975
ALL STRESSES IN kg/cm²

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL				77.20	7.78	9.93
PRESHEAR				77.68	7.90	9.77

G_s = _____ TYPE CELL _____

PRESHEAR: $\bar{\sigma}_{1c}$ = 0.550 t_c = _____ $\bar{\sigma}_{3c}$ = 0.550 P.P.R. = _____ % $\bar{\sigma}_{oct}$ = _____ u_s = _____

PRESHEAR STRESS HISTORY _____

DURING SHEAR: CONTROLLED STRAIN _____ STRESS _____ RATE 0.3%/hr. PATH EXTENSION UNLOADING

ELAPSED TIME	AXIAL STRAIN, %	$\sigma_v - \sigma_h$	$\frac{\sigma_v - \sigma_h}{\bar{\sigma}_{vc}}$	$\bar{\sigma}_v$	$\bar{\sigma}_h$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu (2)	$\frac{\Delta u - \Delta\sigma_3}{\bar{\sigma}_{vc}}$	A (3)	q	p	$\bar{\sigma}_r$	$\bar{\sigma}_a$	$q/\bar{\sigma}_{vc}$	$\bar{p}/\bar{\sigma}_{vc}$	$q/\bar{\sigma}_{ym}$	$\bar{p}/\bar{\sigma}_{ym}$
0	0	0	0	0.550	0.550	1.000	0	0	–	0	0.550	↑	↑	0	1.000	0	0.229
12	0.011	-0.011	-0.020	0.567	0.571	1.017	-0.028	-0.030	-1.51	-0.006	0.572			-0.010	1.040	-0.002	0.238
30	0.040	-0.109	-0.198	0.512	0.622	1.213	-0.072	+0.068	0.343	-0.054	0.567			-0.099	1.030	-0.022	0.236
73	0.209	-0.272	-0.496	0.392	0.664	1.694	-0.114	0.287	0.580	-0.136	0.528			-0.248	0.960	-0.057	0.220
123	0.447	-0.395	-0.718	0.298	0.693	2.324	-0.143	0.458	0.638	-0.197	0.496			-0.354	0.900	-0.082	0.206
283	1.295	-0.546	-0.998	0.184	0.731	3.956	-0.181	0.663	0.668	-0.273	0.458	$\bar{\sigma}_r = \bar{\sigma}_h$	$\bar{\sigma}_a = \bar{\sigma}_v$	-0.496	0.832	-0.114	0.191
463	2.321	-0.552	-1.000	0.167	0.719	4.292	-0.169	0.696	0.693	-0.276	0.443			-0.502	0.805	-0.115	0.185
613	3.178	-0.567	-1.032	0.161	0.728	4.514	-0.178	0.707	0.686	-0.284	0.444			-0.516	0.808	-0.118	0.185
733	3.88	-0.578	-1.052	0.160	0.738	4.603	-0.188	0.709	0.674	-0.289	0.449			-0.526	0.812	-0.120	0.187
883	4.76	-0.586	-1.066	0.166	0.752	4.526	-0.202	0.698	0.655	-0.293	0.459			-0.533	0.834	-0.122	0.191
1060	5.78	-0.556	-1.011	0.191	0.747	3.909	-0.197	0.652	0.645	-0.278	0.469	↓	↓	-0.505	0.852	-0.115	0.195

(1) CORRECTED FOR _____

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

REMARKS:

SECTION D.2-4

TABULATION OF TEST DATA FROM $\overline{CK_OU}$DSS TESTS ON

VERTICAL EAST WINDSOR SAMPLES

DIRECT - SIMPLE SHEAR TEST

PROJECT 80142 TYPE OF TEST DSS NO. EDS-1 OCR 1.0

SOIL TYPE VARVED CLAY TESTED BY JNL DEVICE _____ DATE _____

LOCATION EAST WINDSOR

CONSOLIDATION (Stresses in kg/cm²)

$\bar{\sigma}_{vc}$ 4.013 τ_{hc} 0 $\bar{\sigma}_{vm}$ 4.013

t_c(Day) _____ ε_v(%) _____ γ_c(%) _____ t_c(Day) 2.0

	W,%	e	S,%	H (in)
Initial	54.8			0.972
Preshear				0.860
Final				

DURING SHEAR

Controlled Strain ✓ Stress _____

Rate (% / Hr.) 5

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\bar{\sigma}_{vc}}$	$\frac{\Delta u}{\bar{\sigma}_{vc}}$	$\frac{\bar{\sigma}_v}{\bar{\sigma}_{vc}}$	$\frac{\tau_h}{\bar{\sigma}_v}$	$\frac{\tau_h}{\bar{\sigma}_{vm}}$	$\frac{\bar{\sigma}_v}{\bar{\sigma}_{vm}}$	REMARKS
	0	0	0	1.00	0	0	1.00	
	0.014	0.0038	0	1.00	0.0038			
	0.023	0.0112	0.002	0.998	0.0112			
	0.069	0.0247	0.024	0.976	0.0253			
	0.092	0.0294	0.026	0.974	0.0302			
	0.137	0.0388	0.044	0.956	0.0406			
	0.229	0.0574	0.057	0.943	0.0609			
	0.366	0.0776	0.067	0.933	0.0832			
	0.595	0.0970	0.087	0.913	0.106			
	0.778	0.1082	0.108	0.892	0.121			
	1.144	0.1226	0.134	0.866	0.141			
	1.602	0.1340	0.169	0.831	0.161			
	2.152	0.1436	0.201	0.799	0.180			
	2.564	0.149	0.221	0.779	0.192			
	2.976	0.154	0.249	0.751	0.205			
	3.433	0.158	0.272	0.728	0.217			
	3.891	0.160	0.293	0.707	0.226			
	4.349	0.161	0.316	0.684	0.236			
	4.807	0.160	0.339	0.661	0.242			
	5.265	0.161	0.349	0.651	0.248			
	6.180	0.162	0.380	0.620	0.261			
	6.730	0.163	0.393	0.607	0.268			
	7.416	0.163	0.415	0.585	0.278			
	7.782	0.163	0.428	0.572	0.285			
	8.240	0.163	0.441	0.559	0.291			

SOIL MECHANICS LABORATORY
DEPT. OF CIVIL ENGINEERING
MASSACHUSETTS INSTITUTE OF TECHNOLOGY

REMARKS:

DIRECT - SIMPLE SHEAR (continued)

PROJECT 80142 SOIL EAST WINDSOR VC. TYPE OF TEST DSS NO. EDS-1

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\overline{\sigma}_{vc}}$	$\frac{\Delta u}{\overline{\sigma}_{vc}}$	$\frac{\overline{\sigma}_v}{\overline{\sigma}_{vc}}$	$\frac{\tau_h}{\overline{\sigma}_v}$	$\frac{\tau_h}{\overline{\sigma}_{vm}}$	$\frac{\overline{\sigma}_v}{\overline{\sigma}_{vm}}$	REMARKS
	9.385	0.162	0.462	0.538	0.300			
	10.300	0.161	0.479	0.521	0.309			
	10.987	0.160	0.498	0.502	0.319			
	11.674	0.161	0.504	0.496	0.324			
	12.589	0.160	0.519	0.481	0.332			
	13.596	0.158	0.536	0.464	0.341			
	14.741	0.155	0.557	0.443	0.350			
	17.167	0.150	0.585	0.415	0.361			

SOIL MECHANICS LABORATORY
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REMARKS:

DIRECT - SIMPLE SHEAR TEST

PROJECT 80142 TYPE OF TEST DSS NO. EDS-2 OCR 4.0

SOIL TYPE VARVED CLAY TESTED BY JNL DEVICE _____ DATE _____

LOCATION EAST WINDSOR

CONSOLIDATION (Stresses in kg/cm²)

$\bar{\sigma}_{vc}$ 1.031 τ_{hc} _____ $\bar{\sigma}_{vm}$ 4.017

t_c(Day) 2.0 ε_v(%) ____ γ_c(%) ____ t_c(Day) _____

	W,%	e	S,%	H ( )
Initial				
Preshear				
Final				

DURING SHEAR

Controlled Strain ✓ Stress ____

Rate (% / Hr.) 5

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\bar{\sigma}_{vc}}$	$\frac{\Delta u}{\bar{\sigma}_{vc}}$	$\frac{\bar{\sigma}_v}{\bar{\sigma}_{vc}}$	$\frac{\tau_h}{\bar{\sigma}_v}$	$\frac{\tau_h}{\bar{\sigma}_{vm}}$	$\frac{\bar{\sigma}_v}{\bar{\sigma}_{vm}}$	REMARKS
	0	0	0	1.000	0.000	0.0000	0.2567	
	0.022	0.0183	-0.044	1.044	0.0176	0.0047	0.2679	
	0.044	0.0462	-0.040	1.040	0.0444	0.0118	0.2669	
	0.066	0.0578	-0.040	1.040	0.0556	0.0148	0.2669	
	0.088	0.0727	-0.040	1.040	0.0654	0.0175	0.2669	
	0.177	0.0926	-0.056	1.056	0.0878	0.0238	0.2708	
	0.221	0.1083	-0.076	1.076	0.1007	0.0278	0.2761	
	0.274	0.1278	-0.076	1.076	0.1188	0.0328	0.2761	
	0.310	0.1400	-0.076	1.076	0.1301	0.0359	0.2761	
	0.354	0.1553	-0.080	1.080	0.1438	0.0399	0.2771	
	0.442	0.1863	-0.080	1.080	0.1726	0.0478	0.2771	
	0.531	0.1933	-0.080	1.080	0.1791	0.0496	0.2771	
	0.663	0.2183	-0.088	1.088	0.2008	0.0560	0.2791	
	0.752	0.2309	-0.096	1.096	0.2107	0.0593	0.2813	
	0.973	0.2566	-0.100	1.100	0.2327	0.0657	0.2823	
	1.06	0.2653	-0.116	1.116	0.2378	0.0681	0.2863	
	1.24	0.2833	-0.128	1.128	0.2512	0.0727	0.2895	
	1.55	0.3087	-0.136	1.136	0.2718	0.0792	0.2915	
	1.77	0.3236	-0.152	1.152	0.2810	0.0830	0.2955	
	1.99	0.3366	-0.152	1.152	0.2923	0.0864	0.2955	
	2.21	0.3477	-0.168	1.168	0.2978	0.0892	0.2997	
	2.65	0.3673	-0.180	1.180	0.3114	0.0943	0.3027	
	3.10	0.3845	-0.192	1.192	0.3228	0.0987	0.3057	
	3.54	0.3971	-0.200	1.200	0.3310	0.1019	0.3079	
	3.98	0.4078	-0.208	1.208	0.3377	0.1047	0.3099	

SOIL MECHANICS LABORATORY
DEPT. OF CIVIL ENGINEERING
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REMARKS:

DIRECT - SIMPLE SHEAR (continued)

PROJECT 80142 SOIL VARVED CLAY TYPE OF TEST DSS NO. EDS-2

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\sigma_{vc}}$	$\frac{\Delta u}{\sigma_{vc}}$	$\frac{\overline{\sigma_v}}{\sigma_{vc}}$	$\frac{\tau_h}{\overline{\sigma_v}}$	$\frac{\tau_h}{\sigma_{vm}}$	$\frac{\overline{\sigma_v}}{\sigma_{vm}}$	REMARKS
	4.42	0.4147	-0.219	1.219	0.3402	0.1064	0.3129	
	4.86	0.4208	-0.212	1.212	0.3473	0.1080	0.3169	
	5.31	0.4268	-0.215	1.215	0.3512	0.1095	0.3119	
	5.75	0.4324	-0.219	1.219	0.3547	0.1110	0.3129	
	6.19	0.4412	-0.219	1.219	0.3619	0.1132	0.3129	
	6.64	0.4464	-0.219	1.219	0.3661	0.1146	0.3129	
	7.07	0.4482	-0.219	1.219	0.3676	0.1150	0.3129	
	7.52	0.4496	-0.219	1.219	0.3687	0.1154	0.3129	
	7.96	0.4527	-0.219	1.219	0.3713	0.1162	0.3129	
	8.40	0.4557	-0.208	1.208	0.3773	0.1176	0.3099	
	8.84	0.4562	-0.208	1.208	0.3778	0.1171	0.3099	
	9.29	0.4549	-0.208	1.208	0.3767	0.1168	0.3099	
	10.21	0.4533	-0.188	1.188	0.3819	0.1164	0.3047	
	11.32	0.4529	-0.172	1.172	0.3865	0.1162	0.3007	
	11.72	0.4472	-0.152	1.152	0.3885	0.1148	0.2955	
	12.82	0.4398	-0.124	1.124	0.3915	0.1129	0.2883	
	13.31	0.4361	-0.116	1.116	0.3910	0.1119	0.2863	
	14.19	0.4211	-0.076	1.076	0.3915	0.1081	0.2761	
	15.52	0.3890	-0.006	1.006	0.3868	0.0999	0.2582	
	16.71	0.3630	+0.040	0.960	0.3781	0.0932	0.2465	
	17.47	0.3491	+0.082	0.918	0.3804	0.0896	0.2355	
	18.17	0.3350	+0.104	0.896	0.3738	0.0866	0.2300	
	19.27	0.3166	+0.152	0.848	0.3728	0.0811	0.2176	
	20.16	0.3020	+0.184	0.816	0.3703	0.0775	0.2094	
	21.52	0.2824	+0.224	0.776	0.3640	0.0725	0.1992	
	22.33	0.2716	+0.252	0.748	0.3632	0.0697	0.1919	
	23.33	0.2604	+0.288	0.712	0.3658	0.0668	0.1827	
	24.11	0.2534	+0.306	0.700	0.3619	0.0650	0.1797	
	84.98	0.2433	+0.324	0.676	0.3598	0.0624	0.1735	

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REMARKS:

DIRECT - SIMPLE SHEAR TEST

PROJECT 80142 TYPE OF TEST DSS-3 NO. EDS-3 OCR 1.38

SOIL TYPE VARVED CLAY TESTED BY JNL DEVICE GEONOR DATE

LOCATION EAST WINDSOR CONSOLIDATION (Stresses in ~~kg/cm²~~)

$\bar{\sigma}_{vc}$ 0.524 τ_{hc} ____ $\bar{\sigma}_{vm}$ 3.868

t_c(Day) ____ ε_v(%) ____ γ_c(%) ____ t_c(Day) ____

	W,%	e	S,%	H ( )
Initial				
Preshear				
Final				

DURING SHEAR
Controlled Strain ✓ Stress ____
Rate (% / Hr.) 5

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\bar{\sigma}_{vc}}$	$\frac{\Delta u}{\bar{\sigma}_{vc}}$	$\frac{\bar{\sigma}_v}{\bar{\sigma}_{vc}}$	$\frac{\tau_h}{\bar{\sigma}_v}$	$\frac{\tau_h}{\bar{\sigma}_{vm}}$	$\frac{\bar{\sigma}_v}{\bar{\sigma}_{vm}}$	REMARKS
0	0.00	0.00	0.00	1.00	0.00	0.00	0.135	
	0.246	0.099	-0.063	1.063	0.084	0.012	0.144	
	0.394	0.124	-0.071	1.071	0.116	0.017	0.145	
	0.517	0.167	-0.079	1.079	0.155	0.023	0.146	
	0.787	0.237	-0.141	1.141	0.208	0.032	0.155	
	1.142	0.289	-0.134	1.134	0.255	0.039	0.154	
	1.476	0.324	-0.165	1.165	0.279	0.044	0.158	
	1.969	0.366	-0.181	1.181	0.310	0.050	0.160	
	2.362	0.392	-0.220	1.220	0.321	0.053	0.165	
	2.584	0.409	-0.244	1.244	0.329	0.055	0.169	
	3.223	0.438	-0.259	1.259	0.348	0.059	0.171	
	3.740	0.457	-0.322	1.322	0.346	0.062	0.179	
	4.232	0.477	-0.330	1.330	0.359	0.065	0.180	
	4.724	0.498	-0.330	1.330	0.374	0.067	0.180	
	5.069	0.510	-0.353	1.353	0.377	0.069	0.183	
	5.315	0.516	-0.369	1.369	0.377	0.070	0.186	
	5.709	0.526	-0.381	1.381	0.381	0.071	0.187	
	6.701	0.538	-0.409	1.409	0.382	0.073	0.191	
	6.792	0.551	-0.416	1.416	0.391	0.075	0.192	
	7.185	0.567	-0.432	1.432	0.396	0.077	0.194	
	7.677	0.578	-0.448	1.448	0.399	0.078	0.196	
	8.169	0.586	-0.456	1.456	0.403	0.079	0.197	
	8.661	0.592	-0.464	1.464	0.404	0.080	0.198	
	9.154	0.598	-0.479	1.479	0.404	0.081	0.200	
	9.646	0.606	-0.479	1.479	0.410	0.082	0.200	

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REMARKS:

DIRECT - SIMPLE SHEAR (continued)

PROJECT 80142 SOIL EAST WINDSOR TYPE OF TEST DSS NO. EDS-3

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\bar{\sigma}_{vc}}$	$\frac{\Delta u}{\bar{\sigma}_{vc}}$	$\frac{\bar{\sigma}_v}{\bar{\sigma}_{vc}}$	$\frac{\tau_h}{\bar{\sigma}_v}$	$\frac{\tau_h}{\bar{\sigma}_{vm}}$	$\frac{\bar{\sigma}_v}{\bar{\sigma}_{vm}}$	REMARKS
	10.138	0.612	-0.495	1.495	0.409	0.083	0.203	
	10.630	0.619	-0.503	1.503	0.412	0.084	0.204	
	11.220	0.626	-0.503	1.503	0.416	0.085	0.204	
	11.614	0.627	-0.503	1.503	0.417	0.085	0.204	
	11.860	0.629	-0.519	1.519	0.414	0.085	0.206	
	12.500	0.633	-0.519	1.519	0.417	0.086	0.206	
	12.992	0.634	-0.534	1.534	0.414	0.086	0.208	
	14.222	0.642	-0.511	1.511	0.425	0.087	0.205	
	15.059	0.646	-0.534	1.534	0.417	0.087	0.208	
	15.650	0.643	-0.519	1.519	0.423	0.087	0.206	
	16.929	0.642	-0.519	1.519	0.423	0.087	0.206	
	17.815	0.622	-0.479	1.479	0.420	0.084	0.200	
	18.996	0.584	-0.401	1.401	0.417	0.079	0.190	
	19.931	0.565	-0.369	1.369	0.412	0.077	0.186	
	20.965	0.547	-0.338	1.338	0.409	0.074	0.181	
	21.703	0.536	-0.291	1.291	0.415	0.073	0.175	
	22.441	0.526	-0.271	1.271	0.414	0.071	0.172	
	23.868	0.512	-0.212	1.212	0.423	0.069	0.164	
	24.902	0.494	-0.169	1.169	0.423	0.067	0.158	
	25.89	0.481	-0.141	1.141	0.422	0.065	0.155	

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REMARKS:

SECTION D.2-5

TABULATION OF TEST DATA FROM $\overline{CK_oU}$DSS TESTS ON INCLINED EAST WINDSOR SAMPLES

10 REFERENCES

Note: ASCE = American Society of Civil Engineers
ASTM = American Society of Testing and Materials
JGED = Journal of Geotechnical Engineering Division
JSMFD = Journal of the Soil Mechanics and Foundation Division
ICSMFE = International Conference on Soil Mechanics and Foundation Engineering

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BIOGRAPHY

Surachat Sambhandharaksa was born to Mr. Chanai and Mrs. Sabha Sambhandharaksa on Feb. 24, 1944 in Bangkok, Thailand. He attended high school in Bangkok, Thailand. In 1962, he got a Colombo plan scholarship to study at the University of New South Wales in Sydney, Australia and in 1967 was awarded the degree of Bachelor of Engineering. While an undergradaute, he was employed for two 4-month periods by the Department of Main Roads in New South Wales, Australia.

The writer went to work as an instructor at Chulalongkorn University for one year, before continuing his study under a scholarship from the Asian Institute of Technology where in 1970 he was awarded the Master of Engineering. He again lectured at Chulalongkorn University.

In 1971, he and Dr. Za-Chieh Moh, his thesis advisor at the Asian Institute of Technology, wrote a paper entitled, "Effects of Salt Content on Thixotropic Behavior of a Compacted Clay," published in the Proceedings of the First Australia-New Zealand Conference on Geomechanics, Melbourne, 1971.

On leave of absence from Chulalongkorn University since 1971, he went to study at Northwestern University for a year before transferring to MIT to study for his Doctor of Science.

APPENDIX A

Prefix Δ indicates a change

Suffix f indicates a final or failure condition

A bar over a stress indicates an effective stress

A bar over a property indicates value in terms of effective stress.

INDEX AND CLASSIFICATION PROPERTIES

e_o - Initial void ratio

G_s - Specific gravity

LI - Liquidity index

PI - Plasticity index

w - Water content

W_L - Liquid limit

W_N - Natural water content

W_P - Plastic limit

γ_b - Bouyant unit weight

γ_t - Total unit weight

γ_w - Unit weight of water

STRESSES, STRAINS, MODULUS, AND STRENGTH PATRAMTERS

a - Pore pressure parameter (Eq. 2.9)

$\bar{a}$ - Intercept of q_f vs. $\bar{p}_f$ failure envelope

$\bar{a}/\bar{\sigma}_{vm}$ - Normalized intercept of $q_f/\bar{\sigma}_{vm}$ vs. $\bar{p}_f/\bar{\sigma}_{vm}$ failure envelope

A	-	Skempton's pore pressure parameter A
$A_f(V)$	-	Skempton's pore pressure parameter A at failure for vertical loading
$A_f(PSC)$	-	Skempton's pore pressure parameter A at failure from K_o consolidated plane strain compression test
$A_f(H)$	-	Skempton's pore pressure parameter A at failure for horizontal loading
$A_f(PSE)$	-	Skempton's pore pressure parameter A at failure from K_o consolidated plane strain extension test
$A_f(45°)$	-	Skempton's pore pressure parameter A at failure for inclined (45°) loading
$A_f(DSS)$	-	Skempton's pore pressure parameter A at failure from K_o consolidated direct simple shear test
B	-	Skempton's pore pressure parameter $B = \Delta u/\bar{\sigma}_c$
b/a	-	Axes of Davis-Christian (1971) elliptical strength relationship
c, $\bar{c}$	-	Intercept of Mohr-Coulomb failure envelope
$c/\bar{\sigma}_{vm}$, $\bar{c}/\bar{\sigma}_{vm}$		Normalized intercept of normalized Mohr-Coulomb failure envelope
E, $\bar{E}$	-	Young's modulus
E_u	-	Undrained secant E
$E_u(H)$	-	Undrained secant E from horizontal loading
$E_u(V)$	-	Undrained secant E from vertical loading
$E_u(45°)$	-	Undrained secant E from inclined (45°) loading
f	-	Initial shear stress ratio = $(1-K_o)/[2S_u(V)/\bar{\sigma}_{vo})$
G	-	Shear Modulus
G_i	-	Initial G for hyperbolic stress-strain model
G_y	-	Yielded G for hyperbolic stress-strain model
K	-	Bulk modulus
K_c	-	$\bar{\sigma}_{vc}/\bar{\sigma}_{hc}$

K_o	-	Coefficient of earth pressure at rest
K_s	-	Anisotropic strength ratio = $q_f(H)/q_f(V)$
NESE	-	Normalized effective stress envelope, the plot in terms of $q_f/\bar{\sigma}_{vm}$ versus $\bar{p}_f/\bar{\sigma}_{vm}$
$\overline{\text{NESE}}$	-	Normalized effective stress envelope, the plot in terms of $\bar{\sigma}_{ff}/\bar{\sigma}_{vm}$ versus $\tau_{ff}/\bar{\sigma}_{vm}$
OCR	-	Overconsolidation ratio = $\bar{\sigma}_{vm}/\bar{\sigma}_{vo}$, $\bar{\sigma}_{vm}(L)/\bar{\sigma}_{vc}$
p, $\bar{p}$	-	$0.5(\sigma_v + \sigma_h)$, $1/2(\bar{\sigma}_v + \bar{\sigma}_h)$
q	-	$0.5(\sigma_v - \sigma_h)$ or $1/2\ (\sigma_1 - \sigma_3)$
q_f	-	q at failure
$q_f(V)$	-	q_f from vertical loading
$q_f(H)$	-	q_f from horizontal loading
$q_f(45°)$	-	q_f from inclined (45°) loading
R_f	-	Parameter for hyperbolic stress strain model
S_t	-	Sensitivity = S_u(undisturbed)/S_u(remolded)
S_u	-	Undrained shear strength = q_f or $\tau_{ff} = q_f\cos\bar{\phi}$
$S_u(H)$	-	S_u from horizontal loading
$S_u(V)$	-	S_u from vertical loading
$S_u(45°)$	-	S_u from inclined (45° loading
S_u(DSS)	-	S_u from direct simple shear test
S_u(PSC)	-	S_u from plane strain compression test
S_u(PSE)	-	S_u from plane strain extension test
SCF	-	Strain compatibility factor (see Table 8.4)
SR	-	Strength ratio (see Fig. 5.24)
t_f	-	Time at failure
u	-	Pore water pressure
u_d	-	u for undrained condition
u_m	-	u from field measurement

u_u	-	u at undrained failure
$\bar{\alpha}$	-	Slope of q_f vs. $\bar{p}_f$ failure envelope
α	-	Angle between the failure plane and the horizontal plane
δ	-	Angle between the normal to the failure plane and the major principal stress axis
β	-	Angle between σ_{1f} and vertical direction
θ	-	Varve inclination
γ, γ_f	-	Shear strain
ε	-	Linear strain
ε_a	-	Axial strain
ε_v	-	Vertical strain
ε_{vol}	-	Volumetric strain
μ	-	Field vane correction factor
ν, $\bar{\nu}$	-	Poisson's ratio
σ, $\bar{\sigma}$	-	Normal total stress, normal effective stress
σ_1, σ_2, σ_3		Principal stress
$(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$		Maximum obliquity
σ_c	-	Confining pressure (isotropic)
$\bar{\sigma}_c$	-	Consolidation pressure (isotropic)
σ_{ff}, $\bar{\sigma}_{ff}$	-	Normal stress on failure plane at failure
σ_h	-	Horizontal normal stress
$\bar{\sigma}_{hc}$	-	Norizontal consolidation stress
σ_{oct}	-	Octahedral stress
σ_{ps}	-	Isotropic stress of perfect sample
σ_s	-	Pre-shear effective stress of UUC sample
σ_v	-	Vertical consolidation stress
$\bar{\sigma}_{vf}$	-	Final vertical effective stress

$\bar{\sigma}_{vo}$ - Initial vertical effective stress

$\bar{\sigma}_{vm}$ - Maximum past pressure

$\bar{\sigma}_{vm}(L)$ - Laboratory maximum past pressure

$\bar{\sigma}_{\alpha}$ - Normal stress on a plane with inclination α to the horizontal plane

$\bar{\sigma}_{\alpha o}$ - Initial $\bar{\sigma}_{\alpha}$

τ - Shear stress

τ_{ff} - τ on failure plane at failure

τ_h - τ on horizontal plane (direct-simple shear test)

$\tau_{h_{max}}$ - Maximum τ_h

τ_{oct} - Octahedral shear stress

τ_{α} - τ on a plane with inclination α to the horizontal plane

$\tau_{\alpha o}$ - Initial τ_{α}

$\tau_{\alpha f}$ - τ_{α} at failure

ϕ, $\bar{\phi}$ - Slope of Mohr-Coulomb or normalized Mohr-Coulomb failure envelope

CONSOLIDATION, STABILITY AND SETTLEMENT

CONSOLIDATION, STABILITY AND SETTLEMENT

B - Width of loaded area

C_v - Coefficient of consolidation for vertical flow

CR - Virgin compression ratio

FS - Factor of Safety

H - Depth of foundation clay or height of clay layer

I_ρ - Influence factor for elastic settlement (Eq. 8.1)

P_c - Quasi-consolidation pressure

RR - Recompression ratio

S_R - Settlement ratio = $\frac{\rho_e}{\rho_i}$

t	-	time
t_p	-	t required for primary consolidation
U	-	Degree of consolidation
$\bar{U}$	-	Average degree of consolidation
θ_A	-	Angle between failure surface and horizontal direction for plane strain compression failure surface
θ_p	-	Angle between failure surface and horizontal direction for plane strain extension failure surface
ρ_e	-	"Elastic" settlement
ρ_i	-	Initial settlement

CONSOLIDATION AND STRENGTH TESTS

CD	-	Consolidated-drained shear test
CDDS	-	CD direct shear test
CRSC	-	Constant rate of strain consolidation test
CU	-	Consolidated-undrained shear test
$\overline{\text{CIU}}$	-	Isotropically consolidated-undrained shear test
$\overline{\text{CIU}}$C	-	$\overline{\text{CIU}}$ triaxial compression test
$\overline{\text{CIU}}$E	-	$\overline{\text{CIU}}$ triaxial extension test
$\overline{\text{CK}_o\text{U}}$	-	K_o consolidated-undrained shear test
$\overline{\text{CK}_o\text{U}}$C	-	$\overline{\text{CK}_o\text{U}}$ triaxial compression test
$\overline{\text{CK}_o\text{U}}$DSS		$\overline{\text{CK}_o\text{U}}$ direct simple shear test
$\overline{\text{CK}_o\text{U}}$DSS(C)		$\overline{\text{CK}_o\text{U}}$ direct simple shear test run on inclined sample and sheared by increasing shear stress
$\overline{\text{CK}_o\text{U}}$DSS(E)		$\overline{\text{CK}_o\text{U}}$ direct simple shear test run on inclined sample and sheared by decreasing shear stress
$\overline{\text{CK}_o\text{U}}$E	-	$\overline{\text{CK}_o\text{U}}$ triaxial extension test

$\overline{CK_oU}$PSC		$\overline{CK_oU}$ plane strain compression test
$\overline{CK_oU}$PSE		$\overline{CK_oU}$ plane strain extension test
DSS	-	Direct-simple shear
FV	-	Field vane test
PSC	-	Plane strain compression ($\sigma_{1f} = \sigma_{vf}$)
PSE	-	Plane strain extension ($\sigma_{1f} = \sigma_{hf}$)
TC	-	Triaxial compression
TE	-	Triaxial extension
UC	-	Unconfined compression
UU	-	Unconsolidated-undrained shear test
UUC	-	UU triaxial compression test.

MISCELLANEOUS

CVVC	-	Connecticut Valley varved clays
FV	-	Field vane
M-C	-	Mohr-Coulomb
M-P	-	Morgenstern and Price
MPPR	-	Maximum Past Pressure Ratio = $\frac{\bar{\sigma}_{vm}(L)}{\bar{\sigma}_{vm}}$
MOC	-	Method of Consolidation
NC	-	Normally consolidated
OC	-	Over consolidated
PS	-	Plane strain

APPENDIX B

BEST ESTIMATES* OF STRESS-STRAIN-STRENGTH DATA OF CONNECTICUT VALLEY VARVED CLAYS

Summary of best estimates of test data from SHANSEP $\overline{CK_oUC}$, $\overline{CK_oUE}$ and $\overline{CK_oUDSS}$ tests are presented in Tables B-1 through B-3. Figures B-1 through B-6 show best estimates of normalized stress-strain curves and normalized stress paths from $\overline{CK_oUC}$, $\overline{CK_oUE}$ and $\overline{CK_oUDSS}$ tests on NC and OC samples. The normalized stress-strain curves from $\overline{CK_oUPSC}$ and $\overline{CK_oUPSE}$ tests on NC samples are shown in Figs. 5.20 amd 5.21. The following are the lists of tables and figures

LIST OF TABLES

B-1 Best Estimates of Test Data from $\overline{CK_oUC}$ Tests on NC and OC Samples from the Connecticut Valley

B-2 Best Estimates of Test Data from $\overline{CK_oUDSS}$ Tests on NC and OC Samples from the Connecticut Valley

B-3 Best Estimates of Test Data from $\overline{CK_oUE}$ Tests on NC and OC Samples from the Connecticut Valley

LIST OF FIGURES

B-1 Normalized Stress-Strain Curves from $\overline{CK_oU}$ Triaxial Compression and Extension Tests on Normally Consolidated Connecticut Valley Varved Clays

B-2 Normalized Stress-Strain Curves from $\overline{CK_oU}$ Triaxial Compression and Extension Tests on Overconsolidated Connecticut Valley Varved Clays: OCR=2

*Determined from average data of samples from Amherst, Northampton and East Windsor.

LIST OF FIGURES
(Continued)

B-3 Normalized Stress-Strain Curves from $\overline{CK_oU}$ Triaxial Compression and Extension Tests on Overconsolidated Connecticut Valley Varved Clays: OCR=4

B-4 Normalized Stress Paths from $\overline{CK_oU}$ Triaxial Compression and Extension Tests on Connecticut Valley Varved Clays

B-5 Normalized Stress-Strain Curves from $\overline{CK_oU}$DSS Tests on Normally Consolidated and Overconsolidated Connecticut Valley Varved Clays

B-6 Normalized Stress Paths from $\overline{CK_oU}$DSS Tests on Connecticut Valley Varved Clays

	DATA AT $(\sigma_1 - \sigma_3)_{max}$					DATA AT $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$		
OCR	$q_f/\bar{\sigma}_{vc}$	A_f	ε_f %	$\bar{\phi}°$	$\bar{c}/\bar{\sigma}_{vm}$	ε %	$\bar{\phi}$	$\bar{c}/\bar{\sigma}_{vm}$
1	0.24±0.01	1.43±0.20	1.06±0.2	20.9±0.05	—	13.8±2.0	32.3±0.5	—
2	0.40±0.00	0.50±0.05	2.80±0.05	14.5	0.088	12.6±0.	30.0	0.017
4	0.66±0.02	0.28±.05	7.5±0.8	14.5	0.088	7.5±0.8	30.0	0.017

BEST ESTIMATES OF TEST DATA FROM $\overline{CK_oUC}$ TESTS ON NC AND OC SAMPLES FROM THE CONNECTICUT VALLEY

Table B-1

	At $\tau_{h\,max}$			
OCR	$\tau_h/\bar{\sigma}_{vc}$	$\bar{\sigma}_v/\bar{\sigma}_{vc}$	γ %	$\tau_h/\bar{\sigma}_v$
1	0.16±.01	0.59±.03	7.3±1.5	0.28±.02
2	0.28±.01	0.91±.03	7.7±.02	0.32±.01
4	0.46±.01	1.18±.02	10.0±2.1	0.41±.01

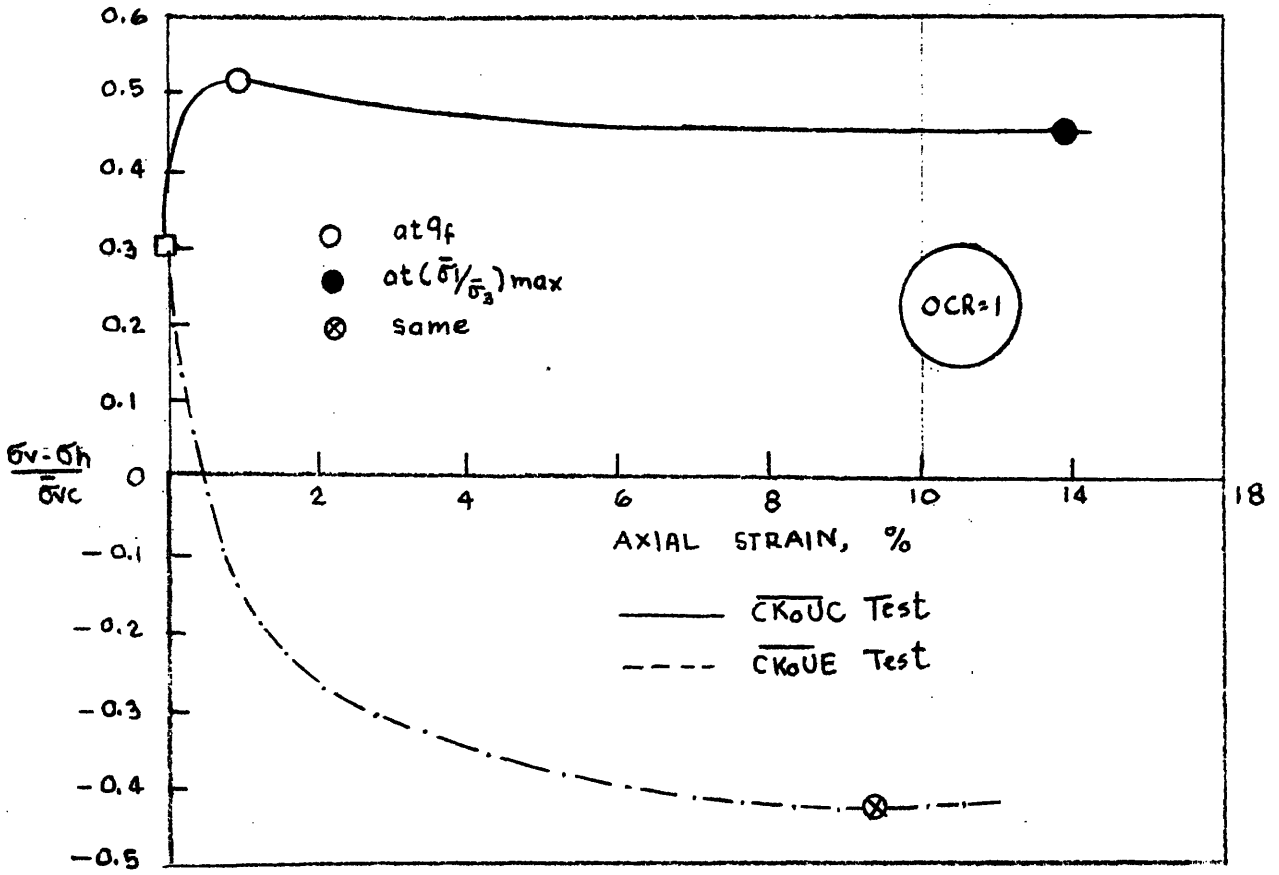
BEST ESTIMATES OF TEST DATA FROM $\overline{CK_oU}$DSS TESTS ON NC AND OC SAMPLES FROM THE CONNECTICUT VALLEY

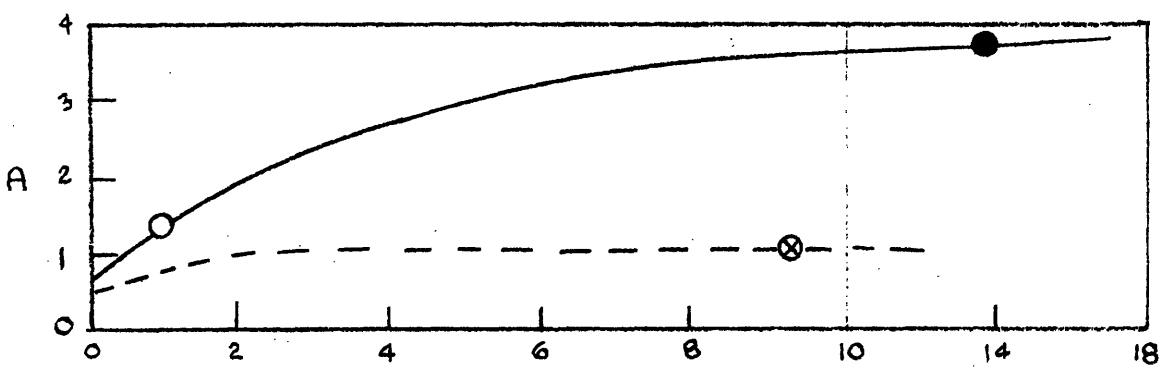
Table B-2

	DATA AT $(\bar{\sigma}_1 - \bar{\sigma}_3)_{max}$					DATA AT $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$		
OCR	$q_f/\bar{\sigma}_{vc}$	A_f	ε_f %	$\bar{\phi}°$	$\bar{c}/\bar{\sigma}_{vm}$	ε %	$\bar{\phi}°$	$\bar{c}/\bar{\sigma}_{vm}$
1	0.205±0.005	1.07±0.01	9.35±.005	31.2±1.2	-	9.35±.005	31.2±1.2	—
2	0.376	0.709	12.5	30.0	0.017	12.5	30.0	0.017
4	0.640±.005	0.36±.03	11.8±0.3	30.0	0.017	11.8±0.3	30.0	0.017

BEST ESTIMATES OF TEST DATA FROM $\overline{CK_oUE}$ TESTS ON NC AND OC SAMPLES FROM THE CONNECTICUT VALLEY

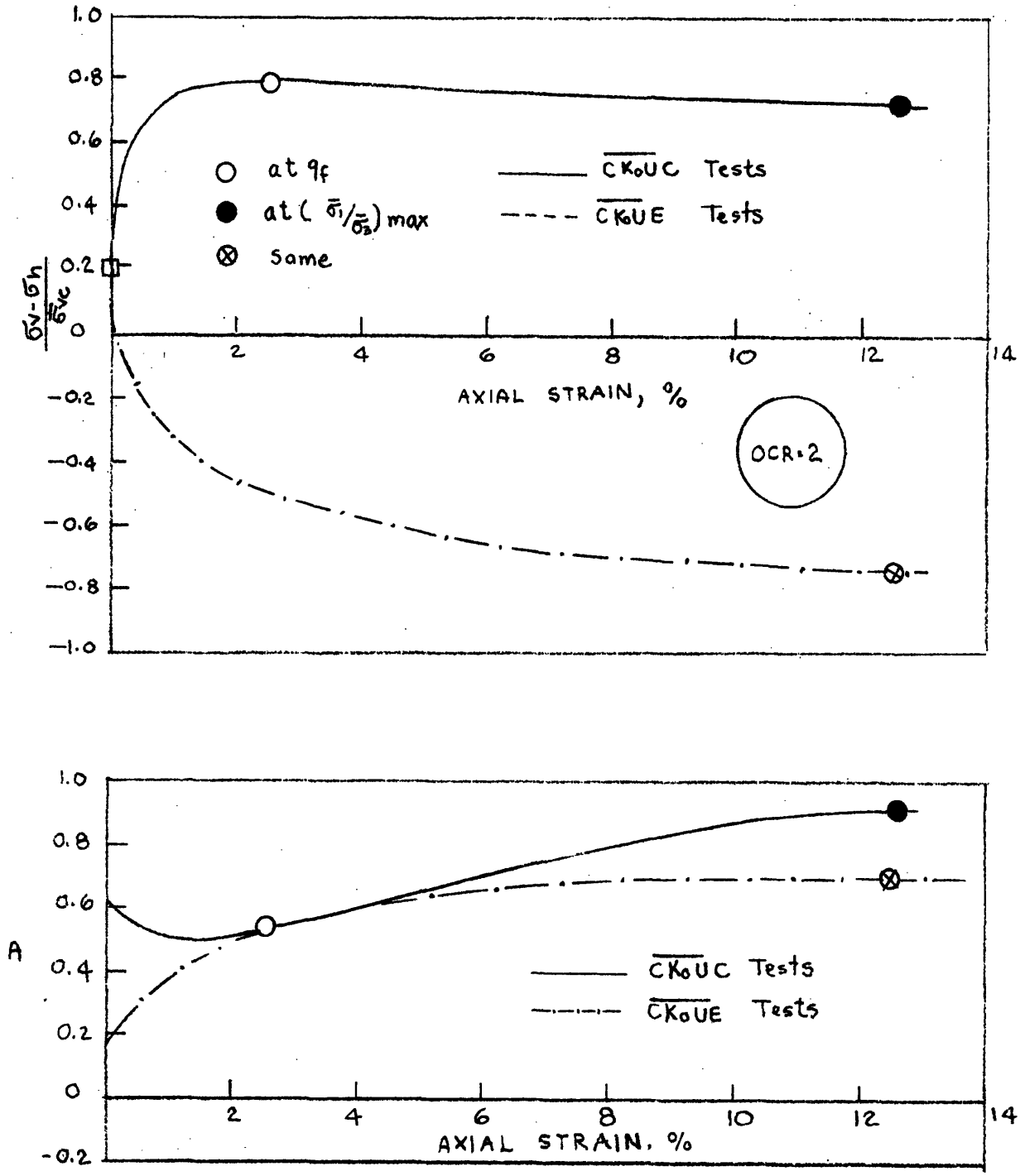
Table B-3





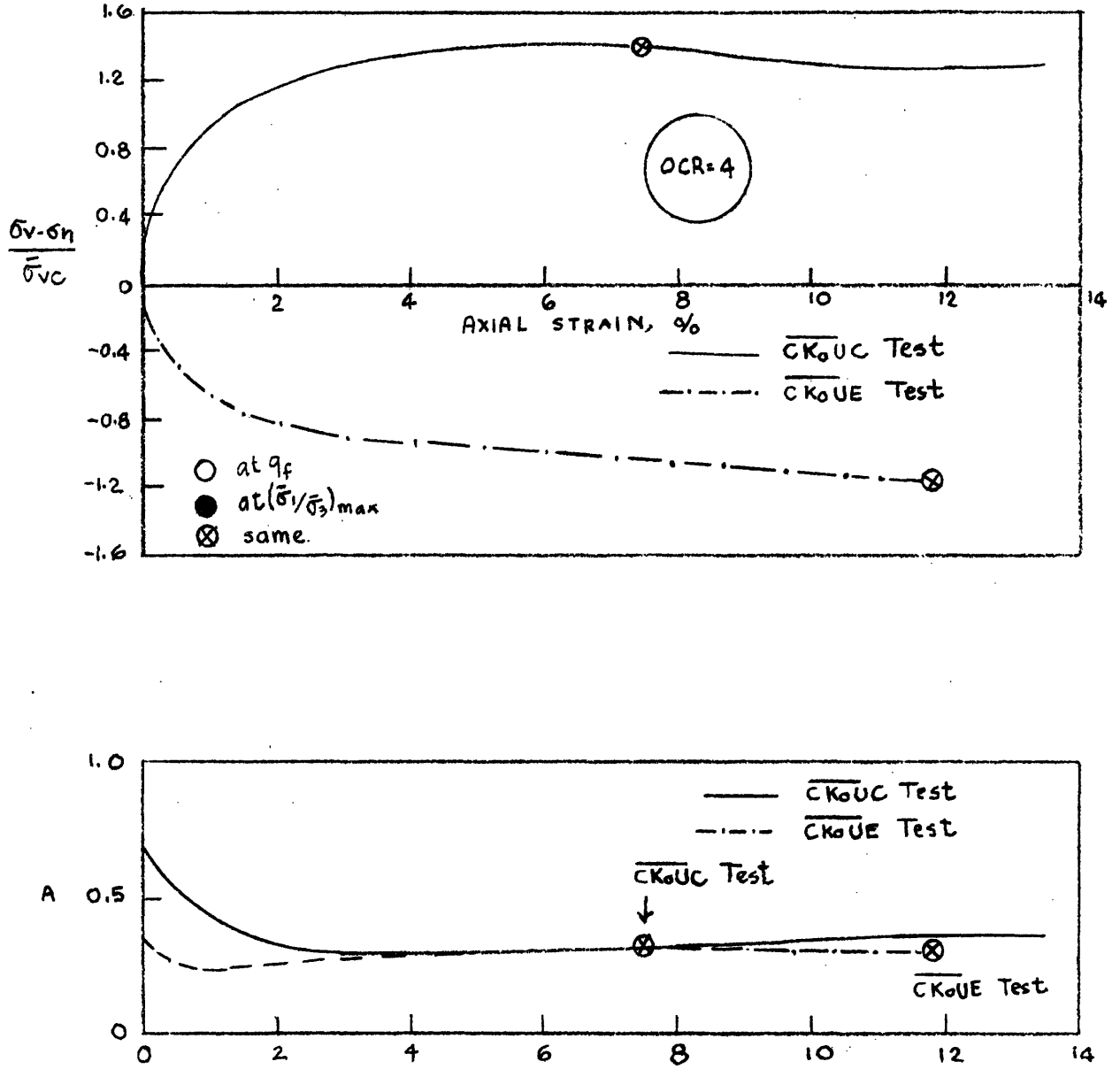
NORMALIZED STRESS-STRAIN CURVES FROM $\overline{CK_oU}$ TRIAXIAL COMPRESSION AND EXTENSION TESTS ON NORMALLY CONSOLIDATED CONNECTICUT VALLEY VARVED CLAYS

Fig. B-1



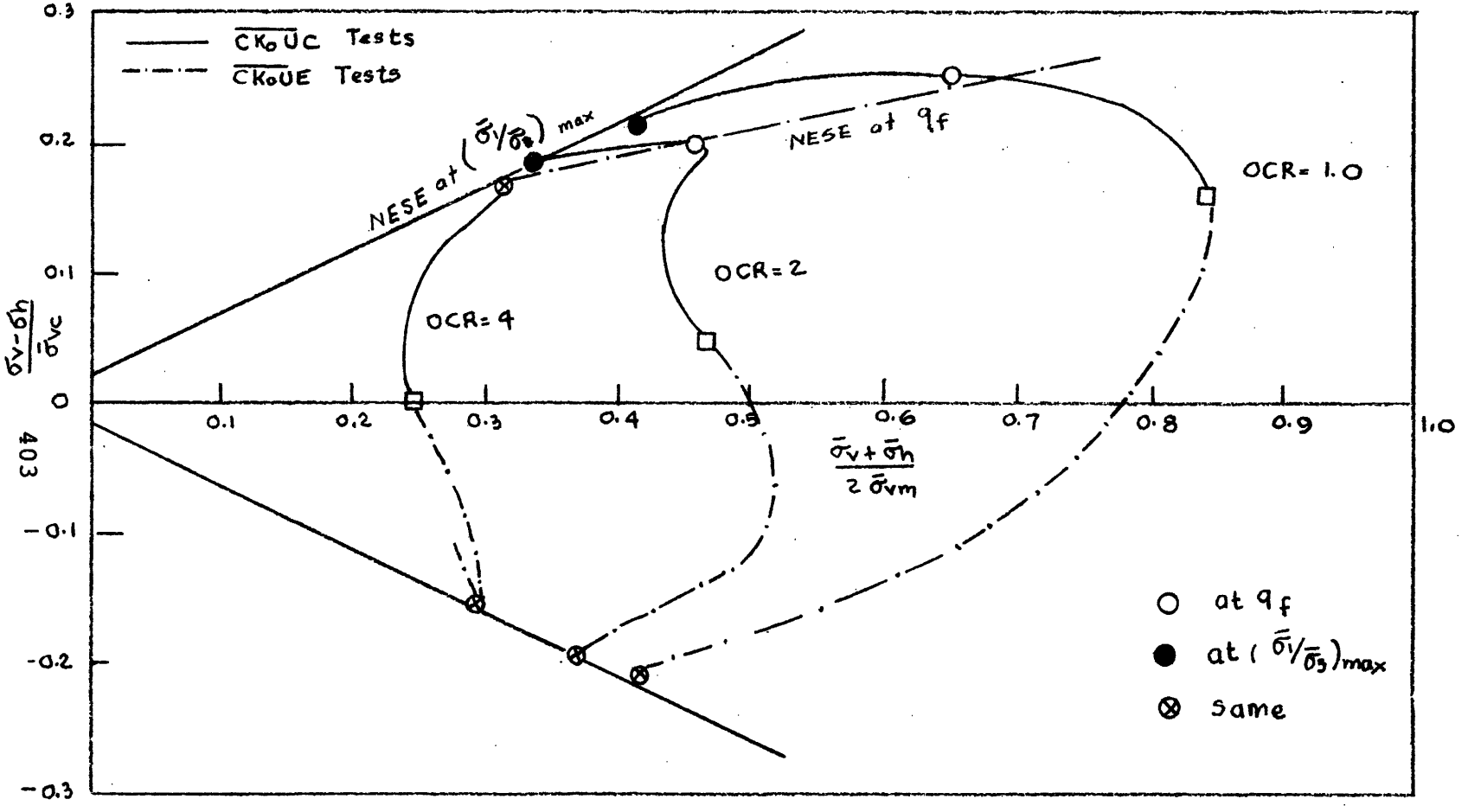
NORMALIZED STRESS-STRAIN CURVES FROM $\overline{CK_oU}$ TRIAXIAL COMPRESSION AND EXTENSION TESTS ON OVERCONSOLIDATED CONNECTICUT VALLEY VARVED CLAYS: OCR=2

Fig. B-2



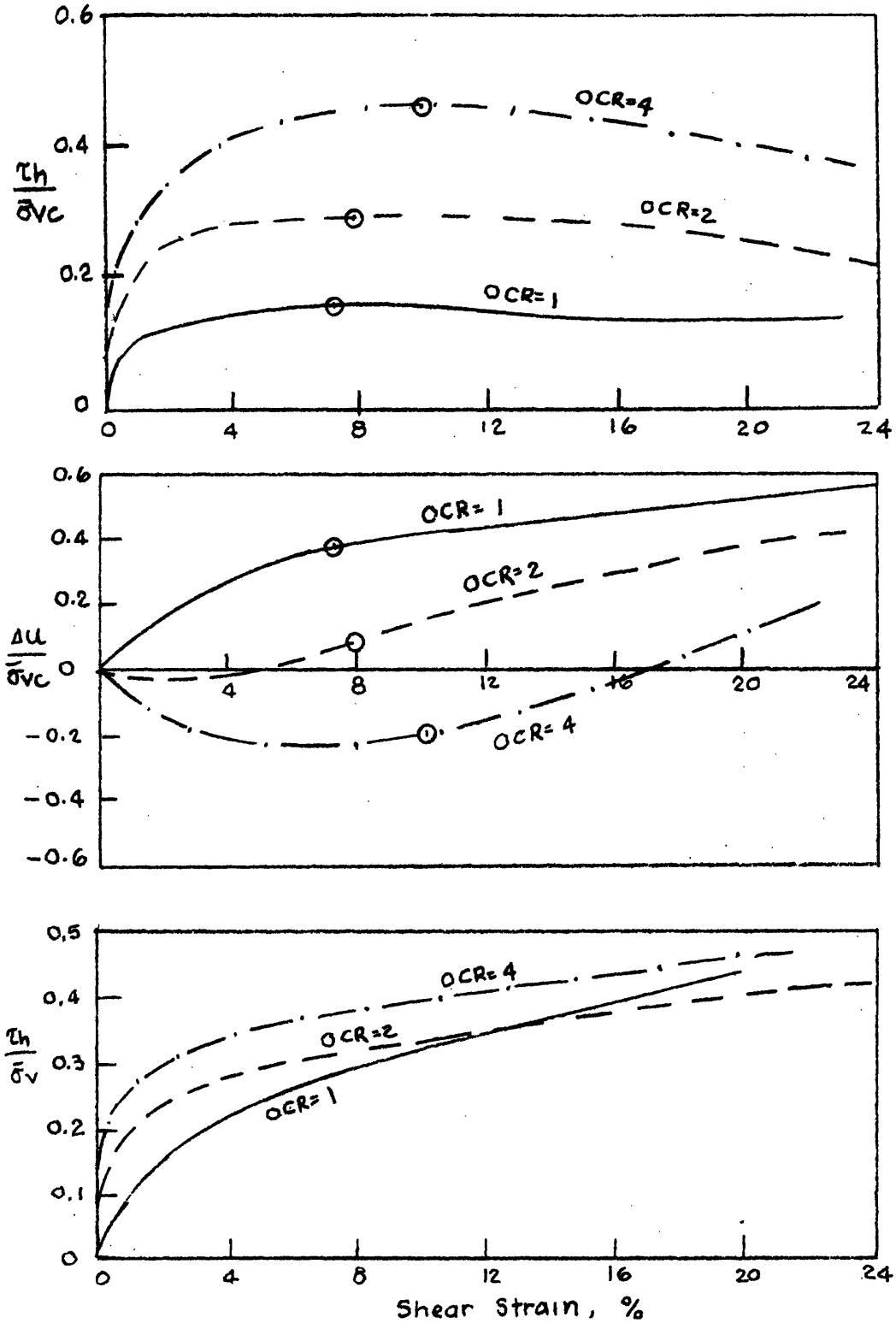
NORMALIZED STRESS-STRAIN CURVES FROM $\overline{CK_oU}$ TRIAXIAL COMPRESSION AND EXTENSION TESTS ON OVERCONSOLIDATED CONNECTICUT VALLEY VARVED CLAYS: OCR=4

Fig. B-3



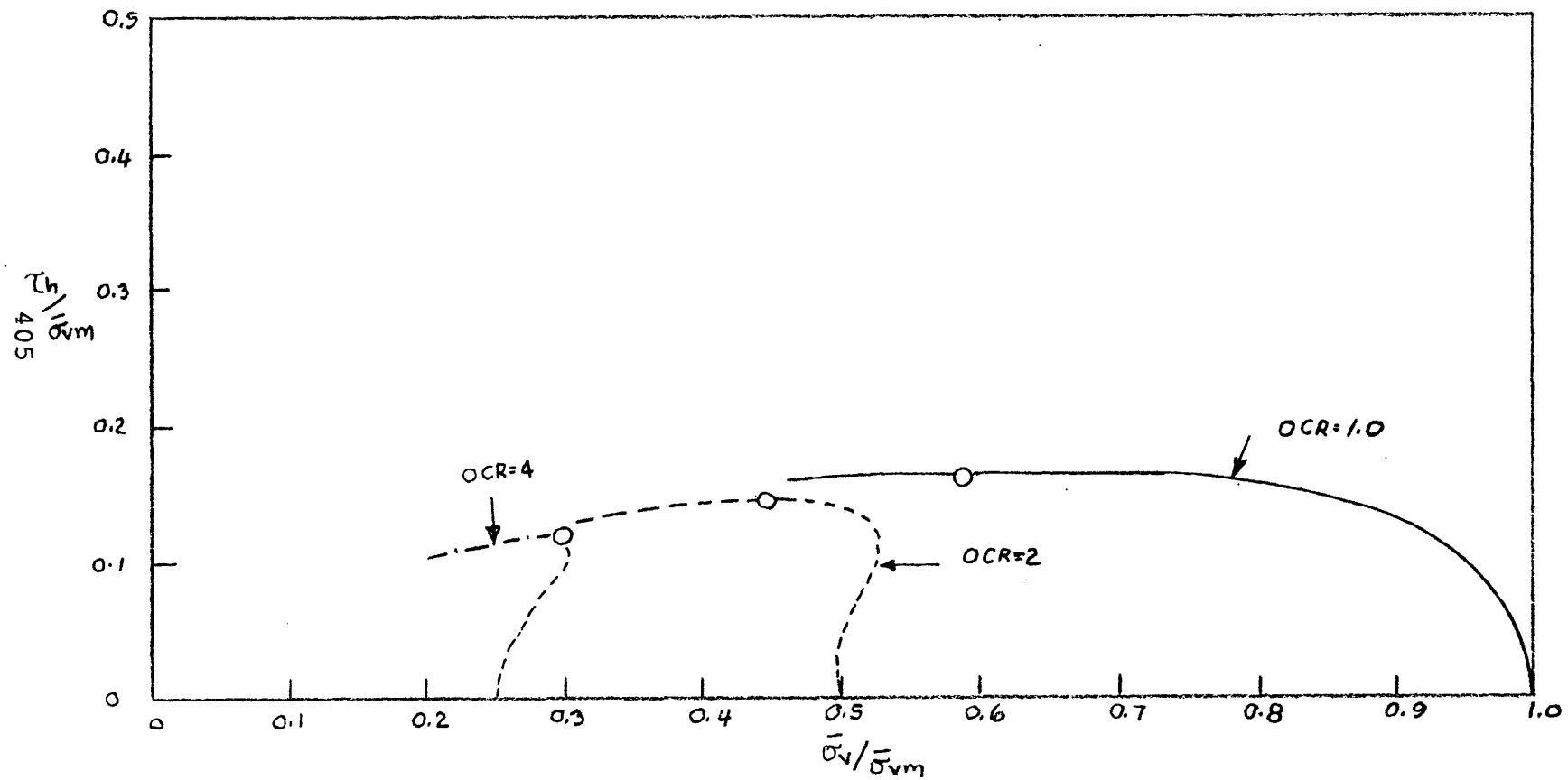
NORMALIZED STRESS PATHS FROM $\overline{CK_oU}$ TRIAXIAL COMPRESSION AND EXTENSION TESTS ON CONNECTICUT VALLEY VARVED CLAYS

Fig. B-4



NORMALIZED STRESS-STRAIN CURVES FROM $\overline{CK_oU}$DSS TESTS ON NORMALLY CONSOLIDATED AND OVERCONSOLIDATED CONNECTICUT VALLEY VARVED CLAYS

Fig. B-5



NORMALIZED STRESS PATHS FROM $\overline{CK_oUDSS}$ TESTS ON CONNECTICUT VALLEY VARVED CLAYS

Fig. B-6

APPENDIX C

SUMMARY OF TEST RESULTS FROM CONNECTICUT VALLEY AND HACKENSACK VALLEY VARVED CLAYS

Appendix C contains a summary of test results from UUC tests, $\overline{CIU}$ and $\overline{CK_oU}$ tests on NC and OC samples from Amherst, East Windsor, Northampton and Hackensack Valley with details of the testing conditions. The following are the lists of tables which are presented in this appendix.

C.1 SUMMARY OF TEST RESULTS FROM AMHERST VARVED CLAY

LIST OF TABLES

C.1-1 Locations of Amherst Samples Used for Undrained Strength Tests

C.1-2 Summary of UUC Tests on Amherst Samples

C.1-3 Summary of $\overline{CIUC}$ Tests on NC Amherst Samples at Different Inclinations (θ=0, 45, and 90) and $\overline{CIUE}$ Tests on NC Vertical and Horizontal Samples

C.1-4 Summary of $\overline{CK_oUC}$ Tests on NC and OC Amherst Samples

C.1-5 Summary of $\overline{CK_oUE}$ Tests on OC Amherst Samples

C.1-6 Summary of $\overline{CK_oUDSS}$ Tests on NC and OC Amherst Samples

C.2 SUMMARY OF TESTS RESULTS FROM EAST WINDSOR VARVED CLAY

LIST OF TABLES

C.2-1 Summary of UUC Tests on East Windsor Samples at Different Inclinations

C.2-2a Summary of $\overline{CIUC}$ Tests on NC and OC SHANSEP East Windsor Samples at Different Inclinations

C.2.2b Summary of $\overline{CIUC}$ Tests on OC SHANSEP East Windsor Samples at Different Inclinations

C.2-3 Summary of $\overline{CIUE}$ Tests on NC and OC SHANSEP East Windsor (θ=0 and 90) Samples

C.2-4 Summary of $\overline{CK_oUC}$ Tests on NC and OC SHANSEP East Windsor Vertical Samples

C.2-5 Summary of $\overline{CK_oUE}$ Tests on NC and OC SHANSEP East Windsor Vertical Samples

C.2-6 Summary of $\overline{CK_oUC}$, $\overline{CK_oUE}$, and $\overline{CIUC}$ (θ=0, 45, 90) Tests on OC RECOMPRESSION East Windsor Samples

C.2-7 Summary of $\overline{CK_oUDSS}$ Tests on NC and OC East Windsor SHANSEP Samples

C.2-8 Summary of $\overline{CK_oUDSS}$ Tests on NC SHANSEP Inclined East Windsor Samples

C.3 SUMMARY OF TEST RESULTS FROM NORTHAMPTON VARVED CLAY

LIST OF TABLES

C.3-1 Summary of $\overline{CK_oUC}$, $\overline{CK_oUPSC}$, and $\overline{CK_oUPSE}$ Tests on NC Northampton Varved Clay

C.3-2 Summary of $\overline{CK_oUDSS}$ Tests on NC and OC SHANSEP Northampton Varved Clay

C.3-3 Summary of CDDS Tests on NC and OC Northampton Varved Clay

C.4 SUMMARY OF TEST RESULTS FROM HACKENSACK VALLEY VARVED CLAY

NOTE: Additional data from Hackensack Valley varved clay can be obtained from Saxena et al. (1974).

LIST OF TABLES

C.4-1 Summary of $\overline{CIUC}$, $\overline{CK_oUC}$ Tests on NC and of $\overline{CK_oUDSS}$ Tests on NC and OC SHANSEP Hackensack Valley Varved Clay Samples

SUMMARY OF TEST DATA FROM $\overline{CIU}$ AND $\overline{CK_oU}$ TESTS ON SHANSEP AND RECOMPRESSION AMHERST SAMPLES

LOCATIONS OF AMHERST SAMPLES USED FOR UNDRAINED STRENGTH TESTS

Stress in kg/cm^2

TEST NO:-	TYPE OF TEST	EL, ft.	In Situ $\bar{\sigma}_{vo}$ $\bar{\sigma}_{vm}$	OCR	$\dot{\varepsilon}$ or $\dot{\gamma}$ %/hr	$\theta°$
AIC-1-1 AIC-2-1 AIC-3-1	$\overline{CIUC}$	115.5	1.5 2.1	1	0.7	0 45 90
AKC-1	$\overline{CK_oUC}$	112.0	0.95 1.70	1	0.5	0
AKC-2 & AKC-4b		83	1.5 2.1	1, 2, 4	0.7	0
AIE-1-1 AIE-3-1	$\overline{CIUE}$	115.5	0.8 1.6	1	0.7	0 90
AKE-1 & AKE-2	$\overline{CK_oUE}$	83	1.5 2.1	2, 4	0.7	0
ADS-1 to ADS-4	$\overline{CK_oUDSS}$	101	1.10 1.70	1, 1.5, 3	5.5	0
ADS-5		112.9	0.80 1.65	6	5.5	0

Table C.1-1

SUMMARY OF UUC TESTS ON AMHERST SAMPLES

Depth ft	$\bar{\sigma}_{vo}$ (TSF)	Su (TSF) (σ_c)			$S_u/\bar{\sigma}_{vo}$			In Situ. $\bar{\sigma}_{vm}$ (TSF)	OCR	REMARK
15.5	0.48	0.36	0.26	0.35 (2.5)	0.76	0.54	0.73	2.3	4.79	-
28	0.74	0.21	0.28 (4.0)	0.30 (6.0)	0.28	0.37	0.41	1.8	2.43	-
38.5	0.95	0.20 (2.5)	0.20	-	0.21	0.21	-	1.6	1.68	ONLY 2 TESTS
48.5	1.15	0.21	0.18	-	0.18	0.16	-	1.8	1.57	ONLY 2 TESTS
58	1.37	0.20	0.20	-	0.15	0.15	-	2.1	1.53	ONLY 2 TESTS
66.5	1.54	0.18	0.21	-	0.12	0.14	-	2.1	1.36	ONLY 2 TESTS

Notes: (1) At least 2 Tests were performed at one depth

(2) σ_c = 2.0 kg/cm^2 in every Test except those are indicated.

Table C.1-2

SUMMARY OF $\overline{CIUC}$ TESTS ON NC AMHERST SAMPLES AT DIFFERENT INCLINATIONS (θ=0, 45, 90) AND $\overline{CIUE}$ TESTS ON NC VERTICAL AND HORIZONTAL SAMPLES

					At $(\sigma_1 - \sigma_3)_{max}$							At $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$					
Test No Type of Test	$\bar{\sigma}_{vc}$ OCR	ω_N % ω_f	θ°	t_c Days ε_{vc} %	$q_f/\bar{\sigma}_{vm}$ $q_f/\bar{\sigma}_{vc}$	$\bar{p}_f/\bar{\sigma}_{vm}$ $\bar{p}_f/\bar{\sigma}_{vc}$	A_f	ε_f	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\phi}^\circ$	$\bar{c}/\bar{\sigma}_{vm}$	$q/\bar{\sigma}_{vm}$ $q/\bar{\sigma}_{vc}$	$\bar{p}/\bar{\sigma}_{vm}$ $\bar{p}/\bar{\sigma}_{vc}$	ε %	$\bar{\sigma}_1/\bar{\sigma}_3$	ϕ°	$\bar{c}/\bar{\sigma}_{vm}$
AIC-1-1	4.0	58.4	0	2.0	0.253	0.513	1.46	5.0	2.94	28.6	–	0.258	0.473	16.8	3.435	33.5	–
$\overline{CIUC}$	1.0	45.0		7.46	0.253	0.513	1.46					0.253	0.473				
AIC-2-1	4.0	60.3	45	2.0	0.214	0.517	1.626	5.88	2.416	24.5	–	0.211	0.465	9.85	2.659	27	–
$\overline{CIUC}$	1.0	46.0		6.60	0.214	0.517			2.416			0.211	0.465				
AIC-3-1	4.0	61.2	90	2.0	0.284	0.605	1.191	9.7	2.765	28	–	0.283	0.601	9.7	2.780	28	–
$\overline{CIUC}$	1.0	48.1		3.81	0.284	0.605											
AIE-1-1	4.0	65.5	0	2.0	0.256	0.526	1.427	10.05	2.893	29.1	–	0.255	0.519	10.9	2.94	29.5	–
$\overline{CIUE}$	1.0	50.2		15.05	0.256	0.526						0.255	0.519				
AIE-3-1	4.0	60.8	90	2.0	0.271	0.434	1.543	4.56	4.30	38.5	–	0.268	0.406	7.32	2.94	41.3	–
$\overline{CIUE}$	1.0	48.2		3.30	0.271	0.434						0.268	0.406				

Table C.1-3

SUMMARY OF $\overline{CK_oUC}$ TESTS ON NC AND OC AMHERST SAMPLES

					At $(\sigma_1 - \sigma_3)_{max}$							At $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$					
Test No.	$\bar{\sigma}_{vc}$ OCR	w_N % w_f	K_c	t_c Days ε_{vc} %	$q_f/\bar{\sigma}_{vm}$ $q_f/\bar{\sigma}_{vc}$	$\bar{p}/\bar{\sigma}_{vm}$ $\bar{p}/\bar{\sigma}_{vc}$	A_f	ε_f	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\phi}^\circ$	$\bar{c}/\bar{\sigma}_{vm}$	$q/\bar{\sigma}_{vm}$ $q/\bar{\sigma}_{vc}$	$\bar{p}/\bar{\sigma}_{vm}$ $q/\bar{\sigma}_{vc}$	ε	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\phi}^\circ$	$\bar{c}/\bar{\sigma}_{vm}$
AKC-1	3.00	–	0.67	2.3	0.248	0.699	1.32	1.00	2.101	21	–	0.230	0.437	15.5	3.216	31.7°	–
	1.00	–		9.5	0.248	0.699						0.230	0.437				
AKC-2	1.8	67.9	0.84	1.0	0.198	0.443	0.541	2.75	2.610	14.5	.088	0.182	0.336	12.6	3.360	30.0	0.017
	2.0	54.0		12,99	0.396	0.885						0.364	0.671				
AKC-3	0.9	66.0	1.0	1.0	0.162	0.305	0.330	8.37	3.258	14.5	.088	0.162	0.305	8.37	3.258	30.0	0.017
	4.0	52.8		11.10	0.646	1.219						0.646	1.219				
AKC-4a	3.6	–	0.68	2	0.25	0.71	1.27	0.65	2.08	20.5	–	—	—	—	—	—	-
	1.0	–		–													
AKC-4b	1.0	62.8	1.0	1.0	0.167	0.321	0.289	8.06	3.186	14.5	.088	0.167	0.321	8.06	3.186	30.0	.017
	4.0	–		12.77	0.669	1.282						0.669	1.282				
* ARKC-1	1.0	61.1	0.75	1.0	0.238	0.422	0.863	4.0	3.45	30.0	0.017	0.238	0.422	4.0	3.576	30.0	.017
	1.6	–		–	0.380	0.680						0.380	0.680				

* RECOMPRESSION SAMPLE

Table C.1-4

SUMMARY OF $\overline{CK_oUE}$ TESTS ON OC AMHERST SAMPLES

					At $(\sigma_1 - \sigma_3)_{max}$							At $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$					
Test No	$\bar{\sigma}_{vc}$ OCR	w_N % w_f	K_c	t_c Days ε_{vc} %	$q_f/\bar{\sigma}_{vm}$ $q_f/\bar{\sigma}_{vc}$	$\bar{p}_f/\bar{\sigma}_{vm}$ $\bar{p}_f/\bar{\sigma}_{vc}$	A_f	ε_f	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\phi}°$	$\bar{c}/\bar{\sigma}_{vm}$	$q_f/\bar{\sigma}_{vm}$ $q_f/\bar{\sigma}_{vc}$	$\bar{p}/\bar{\sigma}_{vm}$ $\bar{p}/\bar{\sigma}_{vc}$	ε	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\phi}°$	$\bar{c}/\bar{\sigma}_{vm}$
AKE-1	1.9	67.8	0.84	1.0	0.188	0.362	0.709	12.5	3.44	30.0	0.017	0.188	0.362	0.709	3.44	30.0	0.017
	2.0	53.5		12.82	0.376	0.724						0.376	0.724				
AKE-2	0.9	65.3	1.0	1.0	0.158	0.309	0.311	12.0	3.00	30.0	0.017	0.158	0.309	12.0	3.00	30.0	0.017
	4.0	51.2		11.03	0.632	1.234						0.632	1.234				

Table C.1-5

SUMMARY OF $\overline{CK_oU}$DSS TESTS ON NC AND OC AMHERST SAMPLES

stress in kg/cm²

Test No. Depth (ft)	In Situ $\bar{\sigma}_{vo}$ $\bar{\sigma}_{vm}$	w_N % w_f	Test OCR	Test $\bar{\sigma}_{vc}$ $\bar{\sigma}_{vm}$	t_c Days ε_{vc} %	At τ_h max γ %	At τ_h max $\tau_h/\bar{\sigma}_{vc}$ $\tau_h/\bar{\sigma}_{vm}$	At τ_h max $\bar{\sigma}_v/\bar{\sigma}_{vc}$ $\bar{\sigma}_v/\bar{\sigma}_{vm}$	At τ_h max $\frac{\tau_h}{\bar{\sigma}_v}$ $\bar{\Phi}^\circ$	At End of Test γ %	At End of Test $\tau_h/\bar{\sigma}_{vc}$ $\tau_h/\bar{\sigma}_{vm}$	At End of Test $\bar{\sigma}_v/\bar{\sigma}_{vc}$ $\bar{\sigma}_v/\bar{\sigma}_{vm}$	At End of Test $\frac{\tau_h}{\bar{\sigma}_v}$ $\bar{\Phi}^\circ$
ADS-1	1.10	61.9	1.0	4.0	2.0	8.5	0.171	0.557	0.307	29.3	0.114	0.275	0.413
43.4	1.70	48.4		4.0	15.2		0.171	0.557	17°		0.114	0.275	22°
ADS-2	1.10	60.0	1.0	3.0	3.0	9.0	0.159	0.519	0.306	30.7	0.111	0.268	0.415
43.6	1.70	53.0		3.0	15.2		0.159	0.519	17°		0.111	0.268	22°
ADS-3[1]	1.10	67.7	1.5	1.1	3.0	5.0	0.200	0.591	0.338	30.1	0.117	0.210	0.547
43.8	1.70	63.4		1.7	6.1		0.133	0.394	–		0.078	0.140	–
ADS-4	1.10	67.9	3.0	1.0	1.0	12.3	0.396	1.093	0.362	37.1	0.279	0.590	0.474
44.0	1.70	54.3		3.0	15.3		0.132	0.364	–		0.093	0.197	–
ADS-5	0.80	59.2	6.0	0.5	1.0	18.7	0.622	1.464	0.425	37.1	0.404	0.781	0.518
32.1	1.65	51.6		3.0	12.1		0.104	0.244	–		0.067	0.130	–

Table C.1-6

SUMMARY OF TEST DATA FROM $\overline{CIU}$ AND $\overline{CK_oU}$ TESTS ON SHANSEP AND RECOMPRESSION EAST WINDSOR SAMPLES

SUMMARY OF UUC TESTS ON EAST WINDSOR SAMPLES AT DIFFERENT INCLINATIONS

Stress in kg/cm^2

Test No	$\bar{\sigma}_{vo}$	In situ $\bar{\sigma}_{vm}$	OCR	Su	$\frac{Su}{\bar{\sigma}_{vo}}$	$\frac{Su}{\bar{\sigma}_{vm}}$	REMARK
EUC-1	0.6	2.4	4	0.384	0.640	0.160	Block Sample
EUC-2				0.204	0.340	0.085	
EUC-3				0.228	0.380	0.095	

Table C.2-1

SUMMARY OF $\overline{CIUC}$ TESTS ON NC AND OC SHANSEP EAST WINDSOR SAMPLES AT DIFFERENT INCLINATIONS

stress in kg/cm^2

					At $(\sigma_1 - \sigma_3)_{max}$							At $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$					
Test No.	$\bar{\sigma}_{vc}$ OCR	ω_N % ω_f	θ°	t_c Days ε_{vc} %	$q_f/\bar{\sigma}_{vm}$ $q_f/\bar{\sigma}_{vc}$	$\bar{P}_f/\bar{\sigma}_{vm}$ $\bar{P}_f/\bar{\sigma}_{vc}$	A_f	ε_f	$(\bar{\sigma}_1/\bar{\sigma}_3)$	$\bar{\phi}^\circ$	$\bar{c}/\bar{\sigma}_{vm}$	$q_f/\bar{\sigma}_{vm}$ $q_f/\bar{\sigma}_{vc}$	$\bar{P}/\bar{\sigma}_{vm}$ $\bar{P}/\bar{\sigma}_{vc}$	ε %	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\phi}^\circ$	$\bar{c}/\bar{\sigma}_{vm}$
EIC-1-1	4.0	56.0	0	2.0	0.240	0.565	1.390	4.45	2.485	25.0	—	—	—	—	—	—	—
	1.0	—		10.14	0.240	0.565											
EIC-2-1	4.0	58.7	45	2.0	0.205	0.540	1.64	8.8	2.24	22.5	—	0.187	0.434	10.31	2.507	25.5	—
	1.0	48.2		2.86	0.205	0.540						0.187	0.434				
EIC-3-1	4.0	53.7	90	2.0	0.290	0.605	1.185	9.29	2.830	28.5	—	0.288	0.605	9.29	2.830	28.5	—
	1.0	45.8		6.34	0.290	0.605						0.288	0.605				
EIC-1-2	2.0	58.2	0	1.0	0.210	0.430	0.675	7.39	2.938	22.5	0.060	0.206	0.402	12.2	3.117	30.0	0.017
	2.0	47.4		5.84	0.420	0.855						0.412	0.803				
EIC-3-2	2.0	57.6	90	1.0	0.255	0.550	0.400	6.54	2.717	30.0	0.017	0.255	0.550	6.54	2.717	30.0	0.017
	2.0	45.4		4.23	0.510	1.110						0.510	1.110				
EIC-1-3	1.0	56.0	0	1.0	0.170	0.338	0.240	11.3	3.040	22.5	0.060	0.170	0.333	13.59	3.070	30.0	0.017
	4.0	—		4.14	0.685	1.350						0.679	1.333				

Table C.2-2a

SUMMARY OF $\overline{CIUC}$ TESTS ON OC SHANSEP EAST WINDSOR SAMPLES DIFFERENT INCLINATIONS

					At $(\sigma_1 - \sigma_3)_{max}$							At $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$				
Test No	$\bar{\sigma}_{vc}$ OCR	W_N % W_f	θ°	t_c Days ε_{vc} %	$q_f/\bar{\sigma}_{vm}$ $q_f/\bar{\sigma}_{vc}$	$\bar{p}_f/\bar{\sigma}_{vm}$ $\bar{p}_f/\bar{\sigma}_{vc}$	A_f	ε_f	$\left(\frac{\bar{\sigma}_1}{\bar{\sigma}_3}\right)$	ϕ°	$\frac{\bar{c}}{\bar{\sigma}_{vm}}$	$q_f/\bar{\sigma}_{vm}$ $q_f/\bar{\sigma}_{vc}$	$\bar{p}_f/\bar{\sigma}_{vm}$ $\bar{p}_f/\bar{\sigma}_{vc}$	ε %	$\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	ϕ°	$\frac{\bar{c}}{\bar{\sigma}_{vm}}$
EIC-2-3	1.0 4.0	57.1 49.5	45	1.0 3.21	0.144 0.574	0.337 1.346	0.199	4.87	2.487	17.0	0.045	0.138 0.550	0.314 1.255	8.72	2.564	22	.027
EIC-3-3	1.0 4.0	60.9 51.7	90	1.0 3.17	0.196 0.784	0.396 1.583	0.128	7.43	2.961	30.0	0.017	0.195 0.779	0.392 1.568	9.04	2.977	30.0	0.017
EIC-1-4	0.20 20	56.3 51.4	0	1.0 2.0	0.080 1.601	0.138 2.757	-0.048	6.31	3.774	30.0	0.017	0.052 1.044	0.086 1.722	6.31	4.080	30.0	0.017
EIC-2-4	0.20 20	57.1 52.4	45	1.0 2.12	0.067 1.330	0.128 2.551	-0.083	4.80	3.179	22.0	.027	0.067 1.330	0.128 2.551	4.80	3.179	22	.027
EIC-3-4	0.20 20	61.0 54.7	90	1.0 2.8	0.098 1.957	0.174 3.484	-0.134	7.13	3.565	30.0	0.017	0.088 1.758	0.150 3.002	3.35	3.830	30.0	0.017

Table C.2-2b

SUMMARY OF $\overline{CIUE}$ TESTS ON NC AND OC SHANSEP EAST WINDSOR (θ=0 and 90) SAMPLES

					At $(\sigma_1-\sigma_3)_{max}$							At $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$					
Test NO	$\bar{\sigma}_{vc}$ OCR	w_N % w_f	θ°	t_c Days ε_{vc},%	$q_f/\bar{\sigma}_{vm}$ $q_f/\bar{\sigma}_{vc}$	$\bar{P}_f/\bar{\sigma}_{vm}$ $\bar{P}_f/\bar{\sigma}_{vc}$	A_f	ε_f	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\phi}^\circ$	$\bar{c}/\bar{\sigma}_{vm}$	$q/\bar{\sigma}_{vm}$ $q/\bar{\sigma}_{vc}$	$\bar{P}/\bar{\sigma}_{vm}$ $\bar{P}/\bar{\sigma}_{vc}$	ε %	$(\bar{\sigma}_1/\bar{\sigma}_3)$	$\bar{\phi}^\circ$	$\bar{c}/\bar{\sigma}_{vm}$
EIE-1-1	4.0 1.0	57.1 47.0	0	2.0 6.06	0.252 0.252	0.497 0.497	1.494	9.86	3.05	30.4	–	0.251 0.251	0.495 0.495	10.06	3.06	30.5	–
EIE-3-1	4.0 1.0	56.6 45.6	90	2.0 3.18	0.268 0.268	0.473 0.473	1.486	6.24	3.61	34.5	–	0.268 0.268	0.473 0.473	6.24	3.610	34.5	–
EIE-1-2	2.0 2	57.4 46.6	0	1.0 5.34	0.231 0.462	0.433 0.866	0.645	9.79	3.295	30.0	.017	0.231 0.462	0.433 0.866	9.79	3.295	30.0	.017
EIE-3-2	2.0 2	59.0 49.2	90	1.0 3.03	0.226 0.452	0.393 0.786	0.748	6.05	3.69	30.0	.017	0.226 0.452	0.393 0.786	6.05	3.694	30.0	.017
EIE-1-3	1.0 4	56.3 47.8	0	1.0 4.12	0.187 0.747	0.326 1.303	0.297	10.70	3.719	30.0	.017	0.186 0.745	0.323 1.293	9.51	3.719	30.0	.017
EIE-3-3	1.0 4	56.2 47.0	90	1.0 3.18	0.175 0.701	0.282 1.128	0.409	4.40	4.283	30.0	.017	0.175 0.701	0.282 1.128	4.40	4.283	30.0	.017

Tables C.2-3

SUMMARY OF $\overline{CK_oUC}$ TESTS ON NC AND OC SHANSEP EAST WINDSOR VERTICAL SAMPLES

					At $(\sigma_1-\sigma_3)_{max}$							At $\left(\bar{\sigma}_1/\bar{\sigma}_3\right)_{max}$					
Test No	$\bar{\sigma}_{vc}$ OCR	w_N % w_f	K_c	t_c Days ε_{vc} %	$q_f/\bar{\sigma}_{vm}$ $q_f/\bar{\sigma}_{vc}$	$\bar{p}_f/\bar{\sigma}_{vm}$ $\bar{p}_f/\bar{\sigma}_{vc}$	A_f	ε_f %	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\phi}^\circ$	$\bar{c}/\bar{\sigma}_{vm}$	$q/\bar{\sigma}_{vm}$ $q/\bar{\sigma}_{vc}$	$\bar{p}/\bar{\sigma}_{vm}$ $\bar{p}/\bar{\sigma}_{vc}$	ε %	$\left(\bar{\sigma}_1/\bar{\sigma}_3\right)$	$\bar{\phi}^\circ$	$\bar{c}/\bar{\sigma}_{vm}$
EKC-1	3.6 1.0	56.3 –	0.68	2.0 7.53	0.250 0.250	0.695 0.695	1.297	0.91	2.133	21°	–	–	–	–	–	–	–
EKC-2	3.0 1.0	57.6 48.9	0.68	2.0 –	0.232 0.232	0.698 0.698	1.49	1.09	1.999	21.1	–	–	–	–	–	–	–
EKC-3	 1.0	– –	0.68	2.0 5.11	0.242	0.646	1.69	1.06	2.193	21.9	–	–	–	–	–	–	–
EKC-4	1.8 2.0	58.5 48.9	0.84	1.0 8.31	0.203 0.405	0.470 0.940	0.452	2.84	2.515	14.5	.088	–	–	–	–	–	–
EKC-5	1.0 4.0	56.3 –	1.0	1.0 8.23	0.169 0.677	0.347 1.388	0.213	6.22	2.908	14.5	.088	0.169 0.675	0.345 1.378	6.22	2.908	30.0	.017

Table C.2-4

SUMMARY OF $\overline{CK_oUE}$ TESTS ON NC AND OC SHANSEP EAST WINDSOR VERTICAL SAMPLES

					At $(\sigma_1 - \sigma_3)_{max}$							At $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$					
Test No.	$\bar{\sigma}_{vc}$ OCR	w_N % w_f	Kc	t_c Days ε_{vc} %	$q_f/\bar{\sigma}_{vm}$ $q_f/\bar{\sigma}_{vc}$	$\bar{p}_f/\bar{\sigma}_{vm}$ $\bar{p}_f/\bar{\sigma}_{vc}$	A_f	ε_f	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\phi}°$	$\bar{c}/\bar{\sigma}_{vm}$	$q_f/\bar{\sigma}_{vm}$ $q_f/\bar{\sigma}_{vc}$	$\bar{p}/\bar{\sigma}_{vm}$ $\bar{p}/\bar{\sigma}_{vc}$	ε %	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\phi}°$	$\bar{c}/\bar{\sigma}_{vm}$
EKE-1	3.6 1.0	56.6 47.1	0.68	2.0 9.91	0.210 0.210	0.412 0.412	1.068	9.41	3.144	32.4	–	0.210 0.210	0.412 0.412	9.41	3.144	32.4	–
EKE-2	3.6 1.0	56.4 46.3	0.68	2.0 8.00	0.205 0.205	0.411 0.411	1.083	9.34	2.994	30	–	0.205 0.205	0.411 0.411	9.34	2.994	30	–
EKE-3	0.9 4.0	58.7 50.1	1.0	1.0 6.51	0.162 0.646	0.286 1.143	0.398	11.5	3.604	30.0	.017	0.162 0.646	0.286 1.143	11.5	3.604	30.0	.017

Table C.2-5

SUMMARY OF $\overline{CK_oUC}$, $\overline{CK_oUE}$, AND $\overline{CIUC}$ (θ=0, 45, 90) TESTS ON OC RECOMPRESSION EAST WINDSOR SAMPLES

						At $(\sigma_1-\sigma_3)_{max}$						At $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$					
Test No. Type of Test	$\bar{\sigma}_{vc}$ OCR	ω_N % ω_f	θ°	K_c	t_c Days ε_{vc} %	$q_f/\bar{\sigma}_{vm}$ $q_f/\bar{\sigma}_{vm}$	$\bar{p}_f/\bar{\sigma}_{vm}$ $\bar{p}_f/\bar{\sigma}_{vm}$	A_f	ε_f	$\bar{\phi}^\circ$	$\bar{c}/\bar{\sigma}_{vm}$	$q/\bar{\sigma}_{vm}$ $q/\bar{\sigma}_{vm}$	$\bar{p}/\bar{\sigma}_{vm}$ $\bar{p}/\bar{\sigma}_{vm}$	ε %	$(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$	$\bar{\phi}^\circ$	$\bar{c}/\bar{\sigma}_{vm}$
ERKC-1	1.2	57.4	0	0.84	1.0	0.215	0.386	0.556	1.60	30.0	.017	0.215	0.386	1.60	3.526	30.0	.017
$\overline{CK_oUC}$	2.0	54.9			1.46	0.430	0.711					0.430	0.711				
ERIC-1-1	0.6	57.9	0	1.0	1.0	0.160	0.251	0.497	2.24	30.0	.017	0.144	0.216	1.13	4.93	30.0	.017
$\overline{CIUC}$	4.0	56.5			0.94	0.639	1.003					0.574	0.866				
ERIC-2-1	0.6	57.8	45	1.0	1.0	0.118	0.240	0.542	3.92	30.0	.017	0.118	0.240	3.92	2.937	30.0	.017
$\overline{CIUC}$	4.0	56.0			1.34	0.473	0.960					0.473	0.960				
ERIC-3-1	1.2	57.8	90	1.0	1.0	0.169	0.373	0.868	2.92	30.0	.017	0.169	0.319	2.92	3.243	30.0	.017
$\overline{CIUC}$	2.0	53.3			2.4	0.345	0.746					0.337	0.638				
ERIC-3-2	0.6	57.8	90	1.0	1.0	0.120	0.245	0.519	8.83	30.0	.017	0.120	0.245	8.83	2.914	30.0	.017
$\overline{CIUC}$	4.0	55.0			1.69	0.480	0.982					0.480	0.982				
ERKE-1	1.2	58.9	0	0.84	1.0	0.177	0.272	0.934	3.16	30.0	.017	0.177	0.272	3.16	4.73	30.0	.017
$\overline{CK_oUE}$	2.0	55.1				0.355	0.545					0.355	0.545				
ERIE-1-1	0.6	57.8	0	1.0	1.0	0.122	0.191	0.655	4.76	30.0	.017	0.120	0.187	4.76	4.526	30.0	.017
$\overline{CIUE}$	4.0	56.6			0.89	0.533	0.834					0.526	0.816				

Table C.2-6

SUMMARY OF $\overline{CK_oU}$DSS TESTS ON NC AND OC EAST WINDSOR SHANSEP SAMPLES

Test No	In situ $\bar{\sigma}_{vo}$ $\bar{\sigma}_{vm}$	w_N % w_f	Testing Condition $\bar{\sigma}_{vc}$ $\bar{\sigma}_{vm}$	OCR	tc pays Evc%	At τ_h max γ %	$\tau_h/\bar{\sigma}_{vm}$ $\tau_h/\bar{\sigma}_{vc}$	$\bar{\sigma}_v/\bar{\sigma}_{vm}$ $\bar{\sigma}_v/\bar{\sigma}_{vc}$	$\tau_h/\bar{\sigma}_v$ ϕ°	At End of Tests γ %	$\tau_h/\bar{\sigma}_{vm}$ $\tau_h/\bar{\sigma}_{vc}$	$\bar{\sigma}_v/\bar{\sigma}_{vm}$ $\bar{\sigma}_v/\bar{\sigma}_{vc}$	$\tau_h/\bar{\sigma}_v$
EDS-1	0.6	54.8	4.01	1.0	2.0	7.4	0.163	0.585	0.278	17.2	0.150	0.415	0.361
	2.4	–	4.01		11.5		0.163	0.585	15.5		0.150	0.415	
EDS-2	0.6	54.1	1.02	≈4.0	1.0	8.8	0.117	0.310	0.378	24.9	0.061	0.169	0.359
	2.4	50.9	4.02		8.2		0.456	1.208	–		0.243	0.676	
EDS-3	0.6	55.7	0.524	7.38	1.0	15.6	0.087	0.206	0.423	25.9	0.065	0.155	0.422
	2.4	–	3.868		10.4		0.643	1.519	–		0.481	1.141	

Table C.2-7

SUMMARY OF $\overline{CK_oU}$DSS TESTS ON NC SHANSEP INCLINED EAST WINDSOR SAMPLES

Test No.	Test Condition: $\bar{\sigma}_{ac}$ (1) / τ_{ac}	Test Condition: τ_c (Days) / ε_a (%)	θ°	At $\tau_{a\,max}$: γ %	At $\tau_{a\,max}$: $\frac{\tau_a}{\bar{\sigma}_{vc}}$	At $\tau_{a\,max}$: $\frac{\tau_a}{\bar{\sigma}_{vc}}$	At $\tau_{a\,max}$: $\frac{\bar{\sigma}_a}{\bar{\sigma}_{vc}}$	At End of Test: γ %	At End of Test: $\frac{\tau_d}{\bar{\sigma}_{vc}}$	At End of Test: $\frac{\tau_d}{\bar{\sigma}_a}$	At End of Test: $\frac{\bar{\sigma}_a}{\bar{\sigma}_{vc}}$	w_N % / w_f	In Situ: $\bar{\sigma}_{vo}$ / $\bar{\sigma}_{vm}$
ESDS-1	3.378 / 0.639	2.0 / 12.8	45	3.6	0.237	0.368	0.644	15.1	0.158	0.423	0.374	54.0 / —	0.6 / 2.4
ESDS-2	3.395 / 0.639	2.0 / 13.1	45	22.0	-0.196	-0.401	0.490	26.6	-0.190	-0.409	0.465	56.4 / —	0.6 / 2.4

Table c.2-8

SUMMARY OF TEST DATA FROM $\overline{CK_oU}$ TESTS ON SHANSEP NORTHAMPTON SAMPLES

SUMMARY OF $\overline{CK_oUC}$, $\overline{CK_oUPSC}$, AND $\overline{CK_oUPSE}$ TESTS ON NC NORTHAMPTON VARVED CLAY

					At $(\sigma_1 - \sigma_3)_{max}$							At $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$					
Test No.	$\bar{\sigma}_{vc}$ OCR	ω_N % ω_f	K_c	t_c Days ε_{vc} %	$q_f/\bar{\sigma}_{vm}$ $q_f/\bar{\sigma}_{vc}$	$\bar{p}_f/\bar{\sigma}_{vm}$ $\bar{p}_f/\bar{\sigma}_{vc}$	A_f	ε_f	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\phi}°$	$\bar{c}/\bar{\sigma}_{vm}$	$q/\bar{\sigma}_{vm}$ $q/\bar{\sigma}_{vc}$	$\bar{p}/\bar{\sigma}_{vm}$ $\bar{p}/\bar{\sigma}_{vc}$	ε	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\phi}°$	$\bar{c}/\bar{\sigma}_{vm}$
NPC-1	4.05 1	61.5 50.8	0.66	2 -	0.279 0.279	0.700 0.706	1.09	0.4	3.33	23.5	-	0.232 0.232	0.425 0.423	3.9	2.34	33.0	-
NKC-1	3.44 1	- -	0.67	3 -	0.250 0.250	0.68 0.68	1.40	1.25	2.17	20.1	-	0.22 0.22	3.00	12.00	3.00	30.0	-
NKC-2	4.00 1	- -	0.68	5 -	0.240 0.240	0.67 0.67	1.68	1.07	2.08	20.6	-	-	-	-	-	-	-
NPE-2	4.00 1	60.1 50.1	0.67	3 -	0.265 0.265	0.598 0.598	0.78	4.4	2.604	26.4	-	0.257 0.257	0.578 0.578	5.4	2.59	26.4	-
NPE-1	4.00 1	60.2 50.3	0.68	2 -	0.252 0.252	0.548 0.548	0.85	4.0	2.707	27.3	-	-	-	-	-	-	-

Table C.3-1

SUMMARY OF $\overline{CK_oU}$DSS TESTS ON NC AND OC SHANSEP NORTHAMPTON VARVED CLAY

stress in kg/cm²

Test NO	Sample No & El.(ft) $\bar{\sigma}_{vo}$	w_N % w_f	Stress History OCR	$\bar{\sigma}_{vc}$ $\bar{\sigma}_{vm}$	t_c	At $\tau_{h\,max}$: γ %	$\frac{\tau_h}{\bar{\sigma}_{vm}}$ $\tau_h/\bar{\sigma}_{vc}$	$\frac{\bar{\sigma}_v}{\bar{\sigma}_{vm}}$ $\bar{\sigma}_v/\bar{\sigma}_{vc}$	$\tau_h/\bar{\sigma}_v$ $\phi°$	At $(\tau_h/\bar{\sigma}_v)_{max}$: γ %	$\tau_h/\bar{\sigma}_{vm}$ $\tau_h/\bar{\sigma}_{vc}$	$\bar{\sigma}_v/\bar{\sigma}_{vm}$ $\bar{\sigma}_v/\bar{\sigma}_{vc}$	$\tau_h/\bar{\sigma}_v$ $\phi°$
NDS-1	B_3, U_6 44.3 2.25	61.4 49.1	1	6.0 6.0	1	8.0	0.152 0.152	0.613 0.613	0.248 14.0	19.8	0.138 0.138	0.413 0.413	0.333 18.3
NDS-2	B_3, U_1 81.9 1.40	57.1 49.0	1	5.0 5.0	2	5.9	0.172 0.172	0.612 0.612	0.280 15.7	14.8	0.160 0.160	0.420 0.420	0.381 20.8
NDS-3	B_3, U_6 43.6 2.28	56.3 50.3	1	4.0 4.0	1	5.1	0.162 0.162	0.662 0.662	0.244 13.7	17.7	0.142 0.142	0.404 0.404	0.352 19.4
NDS-4	B_3, U_2 73.0 1.55	62.5 55.4	2	2.50 5.00	3	7.9	0.147 0.294	0.470 0.940	0.313 17.4	24.7	0.113 0.225	0.260 0.520	0.430 23.3
NDS-5	B_4, U_3 72.6 3.68	59.4 48.7	2	2.50 5.00	1	7.4	0.139 0.278	0.439 0.878	0.317 17.6	23.2	0.119 0.238	0.278 0.555	0.429 23.3
NDS-6	B_4, U_3 72.2 3.20	59.9 49.1	1	6.0 6.0	1	8.9	0.172 0.172	0.617 0.617	0.280 15.7	37.2	0.098 0.098	0.193 0.193	0.503 27.0
NDS-7	B_4, U_3 72.0 3.20	63.9 54.5	4	1.25 5.00	1	11.4	0.118 0.470	0.291 1.162	0.404 22.0	25.4	0.097 0.386	0.192 0.766	0.504 26.8
NDS-8	B_4, U_3 72.0 3.20	57.5 50.0	8	0.625 5.00	1	24.2	0.080 0.636	0.187 1.493	0.187 1.493	24.2	0.064 0.510	0.131 1.048	0.487 26.0

Table C.3-2

SUMMARY OF CDDS TESTS ON NC AND OC NORTHAMPTON VARVED CLAY

Stress in TSF

Test No Boring Sample	EI ft	w_N % w_f	$\bar{\sigma}_{vm}$ TSF	$\bar{\sigma}_{vc}$ TSF	$\frac{\tau_h}{\bar{\sigma}_{vc}}$	$\phi°$	$\frac{q_f}{\bar{\sigma}_{vc}}$ (1)	$\frac{q_f}{\bar{\sigma}_{vm}}$	$\frac{\bar{P}_f}{\bar{\sigma}_{vm}}$ (1)	Location of failure Plane
NCD-1 B_3, U_3	64.5	61.6 53.5	4.00	4.00	0.385	21.0	0.41	0.41	1.15	Clay layer
NCD-2 B_4, U_2	78.4	50 45	5.00	5.00	0.390	21.3	0.42	0.42	1.15	Clay layer
NCD-3 B_3, U_6	43.3	53 43	5.14	5.14	0.448	24.1	0.49	0.49	1.20	Clay layer
NCD-4 B_3, U_3	64.2	60.4 –	6.00	6.00	0.322	18.8	0.36	0.36	1.12	Clay layer
NCD-5 B_3, U_3	64.7	59.1 52.2	3.65	2.00	0.38	19.5	0.41	0.22	0.62	Clay layer
NCD-6 B_4-U_2	77.3	55 53	3.4	1.01	0.495	19.5	0.53	0.16	0.35	Clay layer

Note: (1) assuming horizontal failure plane.

Table C.3-3

SUMMARY OF TEST DATA FROM HACKENSACK VALLEY VARVED CLAY

SUMMARY OF $\overline{CIUC}$, $\overline{CK_oUC}$ TESTS ON NC AND OF $\overline{CK_oUDSS}$ TESTS ON NC AND OC SHANSEP HACKENSACK VALLEY VARVED CLAY SAMPLES

stress in kg/cm^2

Test No	$\bar{\sigma}_{vc}$ OCR	w_N % w_f	θ°	t_c Days ε_{vc} %	K_c	At $(\sigma_1-\sigma_3)_{max}$: $q_f/\bar{\sigma}_{vc}$	A_f	ε_f %	$\bar{\phi}^\circ$	At $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$: $q_f/\bar{\sigma}_{vc}$	A	ε %	$\bar{\phi}^\circ$
HKC-1	3.0 1.0	49.4 43.9	0	2 –	0.65	0.23	1.36	0.6	18.6	0.192	–	15.8	30.0
HIC-1-1	4.0 1.0	– –	0	2 –	1.0	0.25	1.27	5.3	23.6	0.24	1.44	9.5	25.3

stress in kg/cm^2

Test No.	$\bar{\sigma}_{vo}$ $\bar{\sigma}_{vm}$	w_N %	OCR	$\bar{\sigma}_{vc}$ $\bar{\sigma}_{vm(L)}$	At τ_h max: γ %	$\tau_h/\bar{\sigma}_{vm}$ $\tau_h/\bar{\sigma}_{vc}$	$\bar{\sigma}_v/\bar{\sigma}_{vm}$ $\bar{\sigma}_v/\bar{\sigma}_{vc}$	$\tau_h/\bar{\sigma}_v$	At End of test: γ %	$\tau_h/\bar{\sigma}_{vm}$ $\tau_h/\bar{\sigma}_{vc}$	$\bar{\sigma}_v/\bar{\sigma}_{vm}$ $\bar{\sigma}_v/\bar{\sigma}_{vc}$	$\tau_h/\bar{\sigma}_v$
HDS-1	1.3	47	1	4.0 4.0	11.7	0.159	0.614	0.259	24.62	0.136 0.136	0.315 0.315	0.431 0.431
HDS-2		46.9	1	4.0 4.0	10.7	0.157	0.570	0.275	28.50	0.120 0.120	0.303 0.303	0.392 0.392
HDS-3	2.5	50.8	1.98	2.02 4.0	9.5	0.140 0.278	0.448 0.887	0.313	20.36	0.118 0.233	0.340 0.673	0.175 0.347
HDS-4		46	4.12	0.97 4.0	14.8	0.121 0.497	0.329 1.357	0.336	26.93	0.085 0.352	0.211 0.869	0.021 0.085

Table C.4-1

APPENDIX D

TABULATION OF TEST DATA

Consolidated undrained strength test data from Amherst and East Windsor varved clays are tabulated in Sections D.1 and D.2. Strength data from Northampton varved clay were presented in Lacasse et al. (1972). Saxena et al. (1974) presented the test data from Hackensack Valley varved clay. The organization of the tabulated test data is as follows:

Section D.1-1: Tabulation of Test Data from $\overline{CIUC}$ and $\overline{CIUE}$ Tests on NC SHANSEP Amherst Samples

Section D.1-2: Tabulation of Test Data from $\overline{CK_oUC}$ and $\overline{CK_oUE}$ Tests on NC and OC SHANSEP Amherst Samples

Section D.1-3: Tabulation of Test Data from $\overline{CK_oUDSS}$ Tests on NC and OC SHANSEP AND RECOMPRESSION Amherst Samples

Section D.2-1: Tabulation of Test Data from $\overline{CIUC}$ and $\overline{CIUE}$ Tests on NC and OC SHANSEP East Windsor Samples

Section D.2-2: Tabulation of Test Data from $\overline{CK_oUC}$ and $\overline{CK_oUE}$ Tests on NC and OC SHANSEP East Windsor Samples

Section D.2-3: Tabulation of Test Data from $\overline{CK_oUC}$, $\overline{CIUC}$, and $\overline{CK_oUE}$ Tests on RECOMPRESSION East Windsor Samples

Section D.2-4: Tabulation of Test Data from $\overline{CK_oUDSS}$ Tests on Vertical East Windsor Samples

Section D.2-5: Tabulation of Test Data from $\overline{CK_oUDSS}$ Tests on Inclined East Windsor Samples

SECTION D.1-1

TABULATION OF TEST DATA FROM $\overline{\text{CIUC}}$ AND $\overline{\text{CIUE}}$ TESTS ON NC SHANSEP AMHERST SAMPLES

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT. OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. AIC-1-1 (TC)
SOIL EAST WINDSOR VARVED CLAY
PROJECT 80142
TESTED BY JNL. DATE 16 Jan 1975
ALL STRESSES IN kg/cm²

	W, %	e	S, %	V, cc.	L, cm	A, cm²
INITIAL	60.3			78.69	7.93	9.93
PRESHEAR	46			67.61	7.33	9.22

G_s = ____ TYPE CELL GBO.

PRESHEAR: $\bar{\sigma}_{1c}$ = 4.0 t_c = 2 days; $\bar{\sigma}_{3c}$ = 4.0 P.P.R. = — %; $\bar{\sigma}_{oc}$ = 4.0 u_s = 2.00

PRESHEAR STRESS HISTORY Isotropic Consolidated to $\bar{\sigma}_c$ = 4.0 kg/cm²

DURING SHEAR: CONTROLLED STRAIN ✓ STRESS ____ RATE 0.7%/hr PATH compression loading

ELAPSED TIME	AXIAL STRAIN, %	$(\bar{\sigma}_1 - \bar{\sigma}_3)$ (1)	$\frac{(\bar{\sigma}_1 - \bar{\sigma}_3)}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu (2)	$\frac{\Delta u}{\bar{\sigma}_{1c}}$ (2)	A (3)	q	p̄	$\bar{\sigma}_r$	$\bar{\sigma}_a$	$\frac{q}{\bar{\sigma}_{vc}}$	$\frac{p}{\bar{\sigma}_{vc}}$	$\frac{q}{\bar{\sigma}_{vm}}$	$\frac{p}{\bar{\sigma}_{vm}}$
0	0	0	0	4.000	4.000	1.000	0	0	—	0	4.000	↑	↑	0	1.000	0	1.000
99	0.314	1.612	0.403	2.417	4.029	1.667	1.583	0.396	0.982	0.806	3.223			0.202	0.806	0.202	0.806
140	0.764	1.824	0.456	2.026	3.851	1.900	1.974	0.493	1.082	0.912	2.938			0.228	0.735	0.228	0.735
180	1.265	1.927	0.482	1.759	3.687	2.095	2.240	0.560	1.163	0.963	2.723			0.241	0.681	0.241	0.681
220	1.797	1.972	0.493	1.574	3.546	2.253	2.426	0.606	1.230	0.986	2.559			0.247	0.640	0.247	0.640
260	2.346	1.992	0.498	1.434	3.426	2.388	2.566	0.641	1.288	0.996	2.430			0.249	0.608	0.249	0.608
296	2.828	2.002	0.500	1.343	3.345	2.489	2.656	0.664	1.327	1.001	2.344			0.250	0.586	0.250	0.586
326	3.232	2.013	0.503	1.276	3.289	2.578	2.724	0.681	1.353	1.007	2.283			0.252	0.571	0.252	0.571
356	3.644	2.016	0.504	1.221	3.237	2.651	2.779	0.695	1.378	1.008	2.229			0.252	0.557	0.252	0.557
386	4.052	2.022	0.505	1.175	3.196	2.721	2.825	0.706	1.398	1.011	2.185			0.253	0.546	0.253	0.546
416	4.464	2.024	0.506	1.137	3.161	2.781	2.864	0.716	1.415	1.012	2.149			0.253	0.537	0.253	0.537
446	4.881	2.023	0.506	1.098	3.122	2.841	2.901	0.725	1.434	1.011	2.110	$\bar{\sigma}_r = \bar{\sigma}_3$	$\bar{\sigma}_a = \bar{\sigma}_1$	0.253	0.527	0.253	0.527
506	5.736	2.021	0.505	1.042	3.063	2.941	2.958	0.739	1.463	1.011	2.052			0.253	0.513	0.253	0.513
553	6.400	2.019	0.505	1.009	3.029	2.999	2.990	0.748	1.481	1.009	2.019			0.252	0.505	0.252	0.505
615	7.276	2.018	0.504	0.976	2.994	3.067	3.024	0.756	1.498	1.009	1.985			0.252	0.496	0.252	0.496
705	8.511	2.041	0.510	0.936	2.977	3.181	3.064	0.766	1.501	1.021	1.957			0.255	0.489	0.255	0.489
825	10.193	2.035	0.509	0.902	2.937	3.256	3.098	0.774	1.522	1.018	1.919			0.254	0.479	0.254	0.479
945	11.859	2.055	0.514	0.879	2.933	3.338	3.121	0.780	1.519	1.027	1.906			0.257	0.477	0.257	0.477
1065	13.585	2.064	0.516	0.864	2.928	3.388	3.136	0.784	1.519	1.032	1.896			0.258	0.474	0.258	0.474
1185	15.178	2.059	0.515	0.855	2.915	3.408	3.145	0.786	1.527	1.030	1.885			0.258	0.471	0.258	0.471
1305	16.809	2.078	0.519	0.853	2.931	3.435	3.146	0.786	1.514	1.039	1.892			0.259	0.473	0.259	0.473
1425	18.444	2.061	0.515	0.859	2.921	3.398	3.140	0.785	1.523	1.031	1.890	↓	↓	0.258	0.473	0.258	0.473

(1) CORRECTED FOR ____

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

REMARKS:

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT. OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. AIC-2-1 (TC)
SOIL EAST WINDSOR VARVED CLAY
PROJECT 80142
TESTED BY JNL DATE
ALL STRESSES IN kg/cm²

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL	60.3					
PRESHEAR	46.0			66.91	7.37	9.08

G_s = TYPE CELL GEO.

PRESHEAR: $\bar{\sigma}_{1c}$ = 4.0, $\bar{\sigma}_{3c}$ = 4.0, $\bar{\sigma}_{ac}$ = , t_c = 2 days, P.P.R. = — %, u_B = 2.0

DURING SHEAR: CONTROLLED STRAIN ✓ STRESS
RATE 0.7%/hr
PATH compression loading

PRESHEAR STRESS HISTORY

ELAPSED TIME	AXIAL STRAIN, %	$(\bar{\sigma}_1 - \bar{\sigma}_3)$ [1]	$\frac{(\bar{\sigma}_1 - \bar{\sigma}_3)}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu [2]	$\frac{\Delta u}{\bar{\sigma}_{1c}}$ [2]	A [3]	q	$\bar{p}$	$\bar{\sigma}_r$	$\bar{\sigma}_c$	$q/\bar{\sigma}_{vc}$	$\bar{p}/\bar{\sigma}_{vc}$	$q/\bar{\sigma}_{vm}$	$\bar{p}/\bar{\sigma}_{vm}$
0	0	0	0	4.000	4.000	1.000	0	0	—	0	4.000	↑	↑	0	1.000	0	1.000
79	0.175	0.988	0.247	3.085	4.073	1.320	0.915	0.229	0.926	0.494	3.579			0.123	0.895	0.123	0.895
109	0.399	1.258	0.314	2.784	4.042	1.452	1.216	0.304	0.967	0.629	3.413			0.157	0.853	0.157	0.853
139	0.699	1.397	0.349	2.518	3.915	1.555	1.482	0.370	1.060	0.699	3.127			0.175	0.804	0.175	0.804
169	0.986	1.495	0.374	2.302	3.798	1.649	1.697	0.424	1.135	0.648	3.050			0.187	0.763	0.187	0.763
199	1.341	1.577	0.394	2.108	3.684	1.748	1.892	0.473	1.200	0.788	2.896			0.197	0.724	0.197	0.724
229	1.691	1.614	0.404	1.957	3.571	1.825	2.043	0.511	1.266	0.807	2.764			0.202	0.691	0.202	0.691
259	2.021	1.631	0.408	1.839	3.471	1.886	2.160	0.540	1.324	0.816	2.655			0.204	0.664	0.204	0.664
293	2.424	1.664	0.416	1.715	3.379	1.970	2.285	0.571	1.373	0.832	2.547			0.208	0.637	0.208	0.637
338	2.922	1.681	0.420	1.588	3.269	2.059	2.412	0.603	1.435	0.841	2.428			0.210	0.607	0.210	0.607
398	3.630	1.686	0.422	1.466	3.151	2.150	2.534	0.633	1.502	0.843	2.309			0.211	0.577	0.211	0.577
458	4.364	1.708	0.427	1.372	3.079	2.245	2.628	0.657	1.539	0.854	2.226			0.213	0.557	0.213	0.557
518	5.143	1.704	0.426	1.284	2.987	2.327	2.716	0.679	1.595	0.852	2.135			0.213	0.534	0.213	0.534
578	5.879	1.715	0.429	1.211	2.926	2.417	2.789	0.697	1.626	0.858	2.068	$\bar{\sigma}_r = \bar{\sigma}_3$		0.214	0.517	0.214	0.517
638	6.601	1.703	0.426	1.159	2.862	2.469	2.841	0.710	1.669	0.851	2.010			0.213	0.503	0.213	0.503
698	7.330	1.693	0.423	1.117	2.811	2.515	2.883	0.721	1.703	0.847	1.964		$\bar{\sigma}_c = \bar{\sigma}_1$	0.211	0.491	0.211	0.491
758	8.047	1.686	0.421	1.086	2.772	2.552	2.914	0.729	1.728	0.843	1.929			0.211	0.482	0.211	0.482
818	8.781	1.689	0.422	1.050	2.739	2.609	2.950	0.737	1.746	0.845	1.895			0.211	0.474	0.211	0.474
908	9.854	1.686	0.422	1.016	2.702	2.659	2.984	0.746	1.769	0.843	1.859			0.211	0.465	0.211	0.465
1028	11.317	1.605	0.401	0.977	2.582	2.643	3.023	0.756	1.883	0.803	1.780			0.2006	0.445	0.201	0.445
1148	12.870	1.538	0.384	0.956	2.489	2.617	3.049	0.762	1.983	0.769	1.720			0.192	0.430	0.192	0.430
1268	14.411	1.489	0.372	0.933	2.422	2.597	3.067	0.767	2.059	0.745	1.678			0.186	0.419	0.186	0.419
1374	15.799	1.434	0.358	0.929	2.367	2.537	3.067	0.767	2.139	0.717	1.650	↓	↓	0.179	0.413	0.179	0.413

(1) CORRECTED FOR

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

REMARKS:

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT. OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. AIC-3-1 (TC)
SOIL EAST WINDSOR VARVED CLAY
PROJECT 80142
TESTED BY JNL DATE
ALL STRESSES IN

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL	61.2					
PRESHEAR	48.1			76.86	7.70	9.98

G_s = TYPE CELL

PRESHEAR
$\bar{\sigma}_{1c}$ = 4.0 t_c = 2 days
$\bar{\sigma}_{3c}$ = 4.0 P.P.R. = %
$\bar{\sigma}_{oct}$ = u_s = 2.0

DURING SHEAR
CONTROLLED STRAIN ✓ STRESS
RATE 0.7%/hr
PATH Compression loading

PRESHEAR STRESS HISTORY

ELAPSED TIME	AXIAL STRAIN, %	$(\bar{\sigma}_1 - \bar{\sigma}_3)$ (1)	$\frac{(\bar{\sigma}_1 - \bar{\sigma}_3)}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu (2)	$\frac{\Delta u}{\bar{\sigma}_{1c}}$ (2)	A (3)	q	p̄	$\bar{\sigma}_r$	$\bar{\sigma}_a$	$q/\bar{\sigma}_{vc}$	$\bar{p}/\bar{\sigma}_{1c}$	$q/\bar{\sigma}_{vm}$	$\bar{p}/\bar{\sigma}_{1m}$
0	0	0	0	4.000	4.000	1.000	0	0	—	0	4.000	↑	↑	0	1.000	0	1.000
46	0.023	0.619	0.155	3.543	4.162	1.175	0.457	0.114	0.738	0.309	3.853			0.077	0.963	0.077	0.963
56	0.045	0.816	0.204	3.489	4.305	1.234	0.511	0.128	0.626	0.408	3.897			0.102	0.974	0.102	0.974
71	0.097	1.049	0.262	3.395	4.445	1.309	0.605	0.151	0.576	0.525	3.920			0.131	0.980	0.131	0.980
102	0.278	1.366	0.341	3.146	4.512	1.434	0.854	0.214	0.626	0.683	3.829			0.171	0.957	0.171	0.957
162	0.827	1.752	0.438	2.637	4.389	1.665	1.363	0.341	0.778	0.876	3.513			0.219	0.878	0.219	0.878
222	1.416	1.984	0.496	2.279	4.263	1.871	1.721	0.430	0.867	0.992	3.271			0.248	0.818	0.248	0.818
282	2.063	2.095	0.524	2.012	4.107	2.042	1.989	0.497	0.949	1.048	3.059			0.262	0.765	0.262	0.765
342	2.755	2.163	0.541	1.809	3.972	2.196	2.191	0.548	1.013	1.082	2.891			0.270	0.723	0.270	0.723
402	3.412	2.202	0.551	1.667	3.869	2.322	2.334	0.584	1.059	1.101	2.768			0.275	0.692	0.275	0.692
427	3.719	2.219	0.554	1.617	3.836	2.372	2.383	0.596	1.074	1.109	2.727			0.277	0.682	0.277	0.682
488	4.409	2.237	0.559	1.519	3.756	2.473	2.482	0.620	1.109	1.118	2.637			0.279	0.659	0.279	0.659
548	5.118	2.249	0.562	1.443	3.692	2.558	2.557	0.639	1.137	1.124	2.568			0.281	0.642	0.281	0.642
608	5.782	2.251	0.563	1.389	3.640	2.621	2.611	0.653	1.160	1.126	2.514	$\bar{\sigma}_r = \bar{\sigma}_3$	$\bar{\sigma}_a = \bar{\sigma}_1$	0.281	0.629	0.281	0.629
668	6.509	2.251	0.563	1.355	3.606	2.661	2.645	0.661	1.175	1.125	2.480			0.281	0.620	0.281	0.626
728	7.191	2.260	0.565	1.324	3.584	2.708	2.676	0.669	1.184	1.130	2.454			0.283	0.613	0.283	0.613
788	7.918	2.252	0.563	1.300	3.553	2.732	2.699	0.675	1.199	1.126	2.426			0.282	0.607	0.282	0.607
878	8.983	2.268	0.567	1.285	3.553	2.765	2.715	0.679	1.197	1.134	2.419			0.284	0.605	0.284	0.605
998	10.419	2.251	0.563	1.269	3.519	2.773	2.731	0.683	1.213	1.125	2.395			0.281	0.598	0.281	0.598
1118	11.945	2.231	0.558	1.267	3.498	2.761	2.733	0.683	1.225	1.115	2.382			0.279	0.596	0.279	0.596
1238	13.245	2.231	0.558	1.266	3.498	2.762	2.733	0.683	1.225	1.116	2.382			0.279	0.596	0.279	0.596
1358	14.662	2.220	0.555	1.267	3.487	2.753	2.733	0.683	1.231	1.110	2.377	↓	↓	0.278	0.594	0.278	0.594

(1) CORRECTED FOR

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

REMARKS:

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT. OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. AIE-1-1 (TE)
SOIL Amherst. Varved Clay
PROJECT 80142
TESTED BY JNL DATE
ALL STRESSES IN kg/cm²

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL	65.5			78.99	7.90	9.93
PRESHEAR	30.2			66.68	7.19	9.27

σ_3 = TYPE CELL CLG

PRESHEAR: σ_{1c} = 4.00, t_c = 2days.; σ_{3c} = 4.00, P.P.R. = -; σ_{oct} = 4.00, u_s = 2.00

PRESHEAR STRESS HISTORY ISOTROPIC CONSOLIDATED TO 4.0 kg/cm².

DURING SHEAR: CONTROLLED STRAIN ✓ STRESS; RATE 0.7%/hr.; PATH EXTENSION UNLOADING

ELAPSED TIME	AXIAL STRAIN, %	$\sigma_v - \sigma_h$	$\frac{\sigma_v - \sigma_h}{\bar{\sigma}_{vc}}$	$\bar{\sigma}_v$	$\bar{\sigma}_h$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu (2)	$\frac{\Delta u - \Delta\sigma_3}{\bar{\sigma}_{vc}}$	A (3)	q	p	σ_r	σ_a	$\frac{q}{\bar{\sigma}_{vc}}$	$\frac{\bar{p}}{\bar{\sigma}_{vc}}$	$\frac{q}{\bar{\sigma}_{vm}}$	$\frac{\bar{p}}{\bar{\sigma}_{vm}}$
0	0	0	0	4.000	4.000	1.000	0	0	-	0	4.000	4.000	4.000	0	1.000	0	1.000
3:00	0.0002	-0.042	-0.042	3.971	4.013	1.011	-0.013	0.007	0.701	-0.021	3.992	4.013	3.971	-0.005	0.998	-0.005	0.998
7:00	0.002	-0.125	-0.125	3.924	4.049	1.032	-0.049	0.019	0.610	-0.063	3.986	4.049	3.924	-0.016	0.997	-0.016	0.997
13:00	0.009	-0.232	-0.232	3.869	4.102	1.060	-0.102	0.033	0.563	-0.116	3.986	4.102	3.869	-0.029	0.996	-0.029	0.996
23.00	0.033	-0.395	-0.395	3.786	4.181	1.104	-0.181	0.054	0.542	-0.197	3.983	4.181	3.786	-0.049	0.996	-0.049	0.996
41:00	0.111	-0.670	-0.670	3.612	4.282	1.185	-0.282	0.097	0.579	-0.335	3.947	4.282	3.612	-0.084	0.987	-0.084	0.987
86:00	0.444	-1.146	-1.146	3.149	4.295	1.364	-0.295	0.213	0.743	-0.573	3.722	4.295	3.149	-0.143	0.930	-0.143	0.930
116.00	0.740	-1.338	-1.338	2.861	4.198	1.467	-0.198	0.285	0.852	-0.669	3.529	4.198	2.861	-0.167	0.882	-0.167	0.882
143:00	1.041	-1.462	-1.462	2.636	4.098	1.555	-0.098	0.341	0.933	-0.731	3.367	4.098	2.636	-0.183	0.842	-0.183	0.842
173:00	1.397	-1.568	-1.568	2.417	3.985	1.649	0.015	0.396	1.010	-0.784	3.201	3.985	2.417	-0.196	0.800	-0.196	0.800
200:00	1.738	-1.636	-1.636	2.254	3.889	1.725	0.110	0.437	1.068	-0.818	3.072	3.889	2.254	-0.204	0.768	-0.204	0.768
230:00	2.126	-1.696	-1.696	2.097	3.793	1.809	0.207	0.476	1.122	-0.848	2.945	3.793	2.097	-0.212	0.736	-0.212	0.736
260:00	2.522	-1.746	-1.746	1.963	3.709	1.889	0.291	0.509	1.167	-0.873	2.836	3.709	1.963	-0.218	0.709	-0.218	0.709
320:00	3.356	-1.820	-1.820	1.742	3.562	2.045	0.438	0.564	1.241	-0.909	2.652	3.562	1.742	-0.228	0.663	-0.228	0.663
380:00	4.170	-1.877	-1.877	1.572	3.449	2.194	0.551	0.607	1.293	-0.939	2.511	3.449	1.572	-0.235	0.628	-0.235	0.628
440:00	4.976	-1.911	-1.911	1.452	3.363	2.316	0.637	0.637	1.333	-0.956	2.407	3.363	1.452	-0.239	0.602	-0.239	0.602
500:00	5.803	-1.935	-1.935	1.359	3.294	2.424	0.706	0.666	1.365	-0.968	2.326	3.294	1.359	-0.242	0.582	-0.242	0.582
561.00	6.668	-1.958	-1.958	1.284	3.243	2.526	0.758	0.679	1.387	-0.979	2.264	3.243	1.284	-0.245	0.566	-0.245	0.566
602:00	8.389	-2.014	-2.014	1.159	3.173	2.666	0.827	0.710	1.410	-1.007	2.166	3.173	1.159	-0.252	0.542	-0.252	0.542
802:00	10.053	-2.046	-2.046	1.081	3.127	2.893	0.873	0.730	1.427	-1.023	2.104	3.127	1.081	-0.256	0.526	-0.256	0.526
922	11.844	-1.955	-1.955	1.066	3.020	2.833	0.979	0.734	1.501	-0.977	2.093	3.020	1.066	-0.244	0.511	-0.244	0.511

(1) CORRECTED FOR _____

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

REMARKS:

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT. OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. AIE-3-1, 90° Sample (TE)
SOIL Amherst Varved Clay
PROJECT 80142
TESTED BY JNL DATE 27 March 1975
ALL STRESSES IN kg/cm²

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL	60.8			79.45	8.00	9.93
PRESHEAR	48.2			67.67	7.59	8.91

G_s = ____ TYPE CELL CLO

PRESHEAR: $\bar{\sigma}_{1c}$ = 4.0 kg/cm² t_c = 2 days; $\bar{\sigma}_{3c}$ = 4.0 P.P.R. = — %; $\bar{\sigma}_{oc}$ = 4.0 u_B = 2.02

DURING SHEAR: CONTROLLED STRAIN ✓ STRESS ____ RATE 0.7% / hr.

PATH EXTENSION UNLOADING

PRESHEAR STRESS HISTORY Isotropic Consolidated to $\bar{\sigma}_{VC}$ = 4.0 kg/cm²

ELAPSED TIME	AXIAL STRAIN, %	$\sigma_v - \sigma_H$	$\frac{\sigma_v - \sigma_H}{\bar{\sigma}_{VC}}$	$\bar{\sigma}_v$	$\bar{\sigma}_H$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu (2)	$\frac{\Delta u - \Delta \sigma_H}{\bar{\sigma}_{VC}}$	A (3)	q	p̄	$\bar{\sigma}_f$	$\bar{\sigma}_a$	$\frac{q}{\bar{\sigma}_{VC}}$	$\frac{\bar{p}}{\bar{\sigma}_{VC}}$	$\frac{q}{\bar{\sigma}_{VM}}$	$\frac{\bar{p}}{\bar{\sigma}_{VM}}$
0	0	0	0	4.000	4.000	1.000	0	0	—	0	4.000	4.000	4.000	0	1.000	0	1.000
14	0.002	+0.262	0.066	4.023	3.761	1.069	-0.023	0.059	0.912	0.132	3.890	4.023	3.761	0.033	0.972	0.033	0.972
33	0.027	+0.623	0.156	4.034	3.411	1.183	-0.034	0.147	0.945	0.312	3.727	4.034	3.411	0.078	0.931	0.078	0.931
65	0.167	+1.191	0.298	3.990	2.799	1.426	0.010	0.300	1.008	0.596	3.395	3.990	2.799	0.149	0.849	0.149	0.849
125	0.607	+1.832	0.458	3.697	1.865	1.982	0.303	0.533	1.165	0.916	2.781	3.697	1.865	0.229	0.695	0.229	0.695
196	1.45	+2.043	0.511	3.327	1.284	2.591	0.673	0.679	1.329	1.020	2.306	3.327	1.284	0.255	0.576	0.255	0.576
256	2.24	+2.104	0.520	3.126	1.022	3.058	0.874	0.745	1.415	1.052	2.074	3.126	1.022	0.263	0.519	0.263	0.519
376	4.08	+2.163	0.540	2.858	0.694	4.110	1.142	0.826	1.527	1.080	1.776	2.858	0.694	0.270	0.444	0.270	0.444
411	4.56	+2.167	0.541	2.822	0.655	4.30	1.178	0.836	1.543	1.084	1.739	2.822	0.655	0.271	0.434	0.271	0.434
491	5.64	+2.169	0.542	2.763	0.594	4.65	1.237	0.852	1.570	1.084	1.679	2.763	0.594	0.271	0.419	0.271	0.419
551	6.47	+2.161	0.540	2.732	0.570	4.79	1.268	0.857	1.587	1.080	1.651	2.732	0.570	0.270	0.412	0.270	0.412
611	7.32	+2.148	0.537	2.701	0.553	4.88	1.298	0.861	1.604	1.072	1.627	2.701	0.553	0.268	0.406	0.268	0.406
671	8.21	+2.038	0.509	2.642	0.604	4.37	1.358	0.849	1.667	1.020	1.623	2.642	0.604	0.255	0.406	0.255	0.406

(1) CORRECTED FOR ____

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

REMARKS:

SECTION D.1-2

TABULATION OF TEST DATA FROM $\overline{CK_oUC}$ AND $\overline{CK_oUE}$ TESTS

ON NC AND OC SHANSEP AMHERST SAMPLES

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. AKC-1 (TC)
SOIL Amherst Varved Clay
PROJECT 80142
TESTED BY F.D.L. DATE
ALL STRESSES IN kg/cm².

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL				82.88	8.015	10.34
PRESHEAR				74.37	7.256	10.25

G_s = TYPE CELL GEO.

PRESHEAR STRESS HISTORY CIDOUC consolidated to $\bar{\sigma}_{VC}$ = 2.99 kg/cm².

PRESHEAR
$\bar{\sigma}_{1c}$ = 2.99 t_c = 2.3 days
$\bar{\sigma}_{3c}$ = 1.98 P.P.R. = %
$\bar{\sigma}_{ac}$ = 2.99 u_B = 2.01

DURING SHEAR
CONTROLLED STRAIN ✓ STRESS
RATE 0.6%/hr.
PATH Compression loading.

ELAPSED TIME	AXIAL STRAIN, %	$(\sigma_1 - \sigma_3)$ (1)	$\frac{(\bar{\sigma}_1 - \bar{\sigma}_3)}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu (2)	$\frac{\Delta u}{\bar{\sigma}_{1c}}$ (2)	A (3)	q	$\bar{p}$	$\bar{\sigma}_r$	$\bar{\sigma}_c$	$\frac{q}{\bar{\sigma}_{VC}}$	$\frac{\bar{p}}{\bar{\sigma}_{VC}}$	$\frac{q}{\bar{\sigma}_{VM}}$	$\frac{\bar{p}}{\bar{\sigma}_{VM}}$
0	0	1.010	0.337	1.980	2.990	1.510	0	0	–	0.505	2.495	1.980	2.990	0.169	0.832	0.169	0.832
23	0.10	1.302	0.434	1.835	3.137	1.171	0.145	0.048	0.497	0.651	2.486	1.835	3.137	0.217	0.829	0.217	0.828
53	0.27	1.444	0.481	1.653	3.097	1.870	0.327	0.109	0.753	0.722	2.375	1.653	3.097	0.241	0.792	0.241	0.792
83	0.52	1.482	0.494	1.518	3.000	1.976	0.462	0.154	0.939	0.741	2.259	1.518	3.000	0.247	0.753	0.247	0.753
133	1.00	1.486	0.495	1.348	2.834	2.102	0.632	0.211	1.330	0.743	2.091	1.348	2.834	0.248	0.697	0.248	0.697
173	1.39	1.480	0.493	1.241	2.721	2.192	0.739	0.246	1.576	0.740	1.981	1.241	2.721	0.247	0.660	0.247	0.660
223	1.88	1.468	0.489	1.139	2.607	2.288	0.841	0.280	1.840	0.734	1.873	1.139	2.607	0.245	0.624	0.245	0.624
294	2.60	1.446	0.482	1.034	2.480	2.397	0.946	0.315	2.175	0.723	1.757	1.034	2.480	0.241	0.586	0.241	0.586
394	3.59	1.422	0.474	0.927	2.349	2.534	1.053	0.351	2.556	0.711	1.638	0.927	2.349	0.237	0.546	0.237	0.546
493	4.56	1.400	0.467	0.853	2.253	2.640	1.127	0.376	2.897	0.700	1.553	0.853	2.253	0.233	0.518	0.233	0.518
613	5.71	1.382	0.461	0.799	2.179	2.734	1.183	0.394	3.186	0.691	1.488	0.797	2.179	0.230	0.496	0.230	0.496
733	6.85	1.368	0.456	0.715	2.083	2.913	1.265	0.422	3.533	0.684	1.399	0.715	2.083	0.228	0.466	0.228	0.466
913	8.57	1.364	0.455	0.700	2.064	2.949	1.280	0.427	3.616	0.682	1.382	0.700	2.064	0.227	0.461	0.227	0.461
1093	10.29	1.366	0.455	0.608	1.974	3.245	1.372	0.457	3.865	0.683	1.391	0.608	1.974	0.228	0.430	0.228	0.430
1273	12.04	1.372	0.457	0.650	2.022	3.109	1.330	0.443	3.654	0.686	1.336	0.650	2.022	0.229	0.445	0.229	0.445
1453	13.77	1.376	0.459	0.628	2.004	3.189	1.352	0.451	3.704	0.688	1.316	0.628	2.064	0.229	0.439	0.229	0.439
1633	15.45	1.374	0.458	0.610	1.994	3.246	1.360	0.453	3.736	0.687	1.307	0.620	1.994	0.229	0.436	0.229	0.436
1813	17.15	1.348	0.449	0.621	1.969	3.169	1.359	0.453	4.033	0.674	1.295	0.621	1.969	0.225	0.432	0.225	0.432
1991	18.97	1.206	0.402	0.635	1.841	2.898	1.345	0.448	6.897	0.603	1.238	0.635	1.841	0.201	0.413	0.101	0.413
2171	20.80	1.106	0.369	0.640	1.746	2.726	1.340	0.447	14.105	0.553	1.193	0.640	1.746	0.184	0.398	0.184	0.398
2411	23.16	1.078	0.359	0.648	1.723	2.661	1.331	0.445	19.534	0.539	1.188	0.648	1.723	0.180	0.396	0.180	0.396

(1) CORRECTED FOR

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

REMARKS:

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO AKC - 2 (TC)
SOIL EAST WINDSOR VARVED CLAY
PROJECT 80142
TESTED BY SAM DATE
ALL STRESSES IN kg/cm²

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL	67.9			79.38	7.99	9.93
PRESHEAR	54.0			68.78	6.95	9.89

G_s = TYPE CELL GEO

PRESHEAR: $\bar{\sigma}_{1c}$ = 1.800, t_c = 1 day; $\bar{\sigma}_{3c}$ = 1.476, P.P.R. = — %; $\bar{\sigma}_{ac}$ = 1.800, u_B = 2.0

DURING SHEAR: CONTROLLED STRAIN ____ STRESS ____; RATE 0.7%/hr; PATH compression, loading

PRESHEAR STRESS HISTORY Ko consolidation to $\bar{\sigma}_{vc}$ = 3.6 kg/cm² and rebound to 1.80 kg/cm²

ELAPSED TIME	AXIAL STRAIN, %	$(\bar{\sigma}_1 - \bar{\sigma}_3)$ (1)	$\frac{(\bar{\sigma}_1 - \bar{\sigma}_3)}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu (2)	$\frac{\Delta u}{\bar{\sigma}_{1c}}$ (2)	A (3)	q	p	$\bar{\sigma}_r$	$\bar{\sigma}_a$	$\frac{q}{\bar{\sigma}_{vc}}$	$\frac{\bar{F}}{\bar{\sigma}_{vc}}$	$\frac{\tau}{\bar{\sigma}_m}$	$\frac{F}{\sigma_{um}}$
	0	0.324	0.180	1.476	1.800	1.22	0	0	—	0.162	1.638			0.090	0.910	0.045	0.455
	0.040	0.649	0.361	1.245	1.894	1.52	0.231	0.128	0.710	0.325	1.570			0.181	0.872	0.091	0.436
	0.091	0.816	0.453	1.148	1.964	1.71	0.328	0.182	0.667	0.408	1.555			0.227	0.864	0.114	0.432
	0.197	1.080	0.556	1.068	2.068	1.94	0.408	0.227	0.603	0.500	1.568			0.278	0.871	0.139	0.436
	0.270	1.070	0.594	1.046	2.116	2.022	0.429	0.238	0.575	0.535	1.580			0.297	0.878	0.149	0.439
	0.343	1.12	0.622	1.033	2.149	2.08	0.443	0.246	0.555	0.560	1.591			0.311	0.884	0.156	0.442
	0.453	1.184	0.658	1.023	2.207	2.157	0.453	0.252	0.527	0.592	1.615			0.329	0.897	0.165	0.448
	0.526	1.218	0.677	1.015	2.233	2.20	0.461	0.256	0.516	0.610	1.624			0.339	0.902	0.169	0.451
	0.690	1.274	0.708	1.007	2.280	2.26	0.469	0.261	0.494	0.637	1.643			0.354	0.913	0.177	0.457
	0.972	1.337	0.743	0.993	2.33	2.35	0.483	0.268	0.477	0.670	1.661			0.372	0.923	0.186	0.462
	1.260	1.377	0.765	0.974	2.35	2.41	0.502	0.279	0.478	0.689	1.661			0.383	0.923	0.192	0.467
	1.47	1.391	0.773	0.963	2.354	2.44	0.512	0.284	0.483	0.697	1.658	$\bar{\sigma}_r = \bar{\sigma}_3$	$\bar{\sigma}_a = \bar{\sigma}_1$	0.387	0.921	0.194	0.460
	1.84	1.410	0.783	0.939	2.349	2.50	0.536	0.298	0.494	0.706	1.643			0.392	0.913	0.196	0.457
	2.39	1.424	0.789	0.907	2.328	2.56	0.568	0.316	0.518	0.711	1.618			0.395	0.899	0.198	0.449
	2.75	1.425	0.792	0.881	2.305	2.61	0.596	0.331	0.541	0.713	1.593			0.396	0.885	0.198	0.443
	3.12	1.423	0.791	0.851	2.274	2.67	0.625	0.347	0.569	0.713	1.562			0.396	0.868	0.198	0.434
	5.24	1.410	0.783	0.740	2.150	2.90	0.735	0.408	0.677	0.706	1.445			0.392	0.803	0.196	0.402
	5.84	1.398	0.777	0.725	2.12	2.92	0.750	0.417	0.698	0.700	1.422			0.389	0.790	0.195	0.395
	9.89	1.338	0.743	0.596	1.934	3.24	0.879	0.488	0.867	0.670	1.265			0.372	0.703	0.186	0.351
	11.45	1.324	0.732	0.569	1.893	3.32	0.906	0.503	0.906	0.662	1.231			0.368	0.684	0.184	0.342
	12.61	1.310	0.728	0.553	1.863	3.36	0.923	0.513	0.936	0.655	1.208			0.364	0.671	0.182	0.335
	12.98	1.306	0.726	0.555	1.855	3.34	0.926	0.514	0.937	0.653	1.204			0.363	0.669	0.182	0.335

(1) CORRECTED FOR ____

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

REMARKS:

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT. OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. AKC - 3 (TC)
SOIL Amherst. Varved Clay
PROJECT 80142
TESTED BY JNL DATE 31 Jan 1975
ALL STRESSES IN kg/cm².

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL	66.0			79.65	8.02	9.93
PRESHEAR	52.8			69.85	7.13	9.79

G_s = ____ TYPE CE ____

PRESHEAR: $\bar{\sigma}_{1c}$ = 0.9 kg/cm²; $\bar{\sigma}_{3c}$ = 0.9; $\bar{\sigma}_{ac}$ = 0.9; t_c = 1 day; P.P.R. = — %; u_b = 2.00

DURING SHEAR: CONTROLLED STRAIN ____ STRESS ____; RATE 0.7%/hr; PATH compression loading

PRESHEAR STRESS HISTORY Ko consolidation, to $\bar{\sigma}_{vc}$ = 3.6 kg/cm² and rebound to 0.9 kg/cm²

ELAPSED TIME	AXIAL STRAIN, %	$(\bar{\sigma}_1 - \bar{\sigma}_3)$ (1)	$\frac{(\bar{\sigma}_1 - \bar{\sigma}_3)}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu (2)	$\frac{\Delta u}{\bar{\sigma}_{1c}}$ (2)	A (3)	q	$\bar{p}$	$\bar{\sigma}_r$	$\bar{\sigma}_a$	$\frac{q}{\bar{\sigma}_{vc}}$	$\frac{\bar{p}}{\bar{\sigma}_{vc}}$	$\frac{q}{\bar{\sigma}_{vm}}$	$\frac{\bar{p}}{\bar{\sigma}_{vm}}$
0	0	0	0	0.900	0.900	1.000	0	0	—	0	0.900	↑	↑	0	1.000	0	0.250
4	0.005	0.049	0.054	0.834	0.912	1.056	0.036	0.041	0.749	0.024	0.888			0.027	0.987	0.007	0.247
12	0.025	0.162	0.180	0.775	0.938	1.209	0.125	0.139	0.768	0.081	0.857			0.090	0.952	0.023	0.238
20	0.053	0.264	0.293	0.708	0.972	1.373	0.192	0.214	0.728	0.132	0.840			0.147	0.933	0.037	0.233
40	0.139	0.459	0.511	0.607	1.066	1.758	0.293	0.326	0.638	0.229	0.836			0.255	0.929	0.064	0.232
62	0.313	0.599	0.654	0.565	1.154	2.042	0.335	0.372	0.589	0.294	0.859			0.327	0.955	0.082	0.239
86	0.553	0.717	0.796	0.542	1.258	2.323	0.358	0.398	0.500	0.358	0.899			0.398	0.999	0.100	0.250
116	0.834	0.810	0.900	0.541	1.352	2.496	0.358	0.398	0.442	0.405	0.947			0.450	1.052	0.113	0.263
146	1.165	0.902	1.002	0.536	1.438	2.682	0.363	0.404	0.403	0.451	0.987			0.501	1.097	0.125	0.274
191	1.667	0.990	1.100	0.552	1.542	2.794	0.348	0.387	0.351	0.495	1.047			0.550	1.163	0.138	0.291
251	2.410	1.077	1.197	0.552	1.629	2.952	0.348	0.387	0.323	0.539	1.091			0.599	1.212	0.150	0.303
311	3.138	1.111	1.235	0.565	1.676	2.967	0.335	0.372	0.302	0.555	1.120	$\bar{\sigma}_r = \bar{\sigma}_3$	$\bar{\sigma}_a = \bar{\sigma}_1$	0.617	1.245	0.154	0.311
356	3.693	1.132	1.258	0.565	1.697	3.004	0.335	0.372	0.296	0.566	1.131			0.629	1.257	0.157	0.314
386	4.046	1.138	1.264	0.557	1.695	3.042	0.343	0.381	0.301	0.569	1.126			0.632	1.251	0.158	0.313
431	4.896	1.148	1.275	0.555	1.702	3.069	0.346	0.384	0.301	0.574	1.128			0.638	1.254	0.160	0.314
491	5.386	1.136	1.284	0.544	1.700	3.124	0.358	0.395	0.308	0.578	1.122			0.642	1.247	0.161	0.312
551	6.144	1.157	1.286	0.536	1.694	3.158	0.364	0.404	0.314	0.579	1.115			0.643	1.239	0.161	0.310
611	6.876	1.162	1.291	0.534	1.696	3.178	0.366	0.407	0.315	0.581	1.115			0.646	1.239	0.162	0.310
671	7.599	1.159	1.288	0.518	1.677	3.236	0.382	0.424	0.329	0.579	1.098			0.644	1.220	0.161	0.305
731	8.372	1.164	1.293	0.516	1.679	3.258	0.384	0.427	0.330	0.582	1.098			0.647	1.219	0.162	0.305
791	9.078	1.163	1.292	0.505	1.668	3.303	0.395	0.439	0.339	0.582	1.087			0.646	1.207	0.162	0.302
851	9.826	1.163	1.293	0.500	1.663	3.327	0.400	0.444	0.344	0.582	1.082			0.646	1.202	0.162	0.301
911	10.584	1.160	1.289	0.495	1.655	3.345	0.405	0.450	0.349	0.580	1.075			0.644	1.194	0.161	0.299
971	11.367	1.152	1.280	0.498	1.649	3.316	0.403	0.447	0.349	0.576	1.073			0.640	1.193	0.160	0.298
1031	12.127	1.140	1.267	0.495	1.635	3.304	0.405	0.450	0.355	0.570	1.065			0.633	1.183	0.158	0.296
1151	13.699	1.098	1.221	0.492	1.591	3.232	0.408	0.453	0.371	0.549	1.041	↓	↓	0.616	1.157	0.153	0.289

(1) CORRECTED FOR ____

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_3}{\Delta\sigma_r - \Delta\sigma_3}$ FOR EXTENSION TESTS

REMARKS:

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT. OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. AKC-4a, (TC)
SOIL Amherst
PROJECT
TESTED BY DATE
ALL STRESSES IN kg/cm²

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL	62.8					
PRESHEAR						

G_s = TYPE CELL

PRESHEAR STRESS HISTORY OCR = 1.0

PRESHEAR
$\bar{\sigma}_{1c}$ = 3.6 t_c = 2
$\bar{\sigma}_{3c}$ = 2.45 P.P.R. = %
$\bar{\sigma}_{ac}$ = u_s = 2.0

DURING SHEAR
CONTROLLED STRAIN ✓ STRESS
RATE 0.7%/hr
PATH

ELAPSED TIME	AXIAL STRAIN, %	$(\bar{\sigma}_1 - \bar{\sigma}_3)$ (1)	$\frac{(\bar{\sigma}_1 - \bar{\sigma}_3)}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu (2)	$\frac{\Delta u}{\bar{\sigma}_{1c}}$ (2)	A (3)	q	$\bar{p}$	γ_d/γ_{dc}	$\bar{\gamma}_{ave}$				
0	0	1.151	0.319	2.448	3.599	1.47	0	0	–	0.58	3.05	0.160	0.840				
4	0.004	1.197	0.332	2.409	3.606	1.49	0.039	0.011	0.859	0.598	3.007	0.166	0.835				
8	0.008	1.259	0.349	2.360	3.619	1.53	0.088	0.024	0.815	0.629	2.99	0.175	0.830				
30	0.08	1.582	0.439	2.08	3.64	1.76	0.362	0.100	0.840	0.791	2.87	0.220	0.799				
46	0.19	1.699	0.472	1.94	3.64	1.88	0.509	0.141	0.929	0.849	2.788	0.236	0.779				
62	0.363	1.787	0.488	1.842	3.569	1.969	0.636	0.177	1.05	0.873	2.690	0.244	0.767				
74	0.513	1.773	0.493	1.727	3.500	2.027	0.721	0.200	1.16	0.886	2.613	0.246	0.756				
86	0.688	1.781	0.495	1.647	3.488	2.082	0.861	0.239	1.27	0.891	2.557	0.247	0.705				
98	0.846	1.764	0.490	1.587	3.351	2.115	0.861	0.239	1.41	0.882	2.469	0.245	0.686				
104	0.926	1.774	0.493	1.553	3.328	2.142	0.894	0.249	1.43	0.887	2.440	0.246	0.678				

(1) CORRECTED FOR

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

REMARKS:

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT. OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. AKC-4b. (TC)
SOIL Amherst Varved Clay.
PROJECT 80142
TESTED BY JUL DATE 4 Feb 1975
ALL STRESSES IN kg/cm².

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL	62.8			78.25	7.88	9.93
PRESHEAR				67.76	6.87	9.85

G_s = _____ TYPE CELL GEO

PRESHEAR: $\bar{\sigma}_{1c}$ = 1.00 kg/cm², t_c = 1 day; $\bar{\sigma}_{3c}$ = 1.00, P.P.R. = 210 %; $\bar{\sigma}_{oc}$ = 1.00, u_s = 2.00

DURING SHEAR: CONTROLLED STRAIN ✓ STRESS _____; RATE 0.7 %/hr; PATH compression loading

PRESHEAR STRESS HISTORY CKoUC to $\bar{\sigma}_{vc}$ = 3.6 kg/cm², and shear to max q. the increase $\bar{\sigma}_{vc}$ to 4.0 kg/cm² and rebound to 1.0 kg/cm².

ELAPSED TIME	AXIAL STRAIN, %	$(\sigma_1 - \sigma_3)$ (1)	$\frac{(\bar{\sigma}_1 - \bar{\sigma}_3)}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu (2)	$\frac{\Delta u}{\bar{\sigma}_{1c}}$ (2)	A (3)	q	$\bar{p}$	$\bar{\sigma}_r$	$\bar{\sigma}_c$	$\frac{q}{\bar{\sigma}_{vc}}$	$\frac{\bar{p}}{\bar{\sigma}_{vc}}$	$\frac{q}{\bar{\sigma}_{vm}}$	$\frac{\bar{p}}{\bar{\sigma}_{vm}}$
0	0	0	0	1.000	1.000	1.000	0	0	–	0	1.000	↑		0	1.000	0	0.250
2:00	0.007	0.023	0.023	0.982	1.004	1.023	0.018	0.018	0.800	0.011	0.993			0.011	0.993	0.003	0.248
6:00	0.008	0.098	0.098	0.923	1.021	1.106	0.078	0.078	0.788	0.049	0.972			0.049	0.972	0.012	0.243
14:00	0.028	0.212	0.212	0.840	1.052	1.253	0.160	0.160	0.756	0.106	0.946			0.106	0.946	0.027	0.237
22:00	0.056	0.315	0.315	0.783	1.098	1.402	0.217	0.217	0.690	0.157	0.940			0.157	0.940	0.039	0.235
38:00	0.148	0.501	0.501	0.682	1.184	1.735	0.318	0.318	0.634	0.251	0.933			0.251	0.933	0.063	0.233
58:00	0.298	0.653	0.653	0.628	1.281	2.041	0.372	0.372	0.570	0.327	0.954			0.327	0.954	0.082	0.239
78:00	0.478	0.768	0.768	0.602	1.369	2.276	0.398	0.398	0.518	0.384	0.986			0.384	0.986	0.096	0.247
99:00	0.698	0.864	0.865	0.594	1.459	2.456	0.406	0.406	0.469	0.432	1.027			0.432	1.027	0.108	0.257
119:00	0.902	0.917	0.917	0.607	1.524	2.511	0.393	0.393	0.428	0.459	1.066			0.459	1.066	0.115	0.267
139:00	1.119	0.998	0.998	0.597	1.595	2.673	0.403	0.403	0.404	0.499	1.096			0.499	1.096	0.125	0.274
169:00	1.457	1.074	1.074	0.599	1.674	2.793	0.400	0.400	0.373	0.537	1.137			0.537	1.137	0.134	0.284
207:00	1.907	1.141	1.141	0.620	1.761	2.839	0.380	0.380	0.333	0.570	1.190			0.570	1.190	0.143	0.298
267:00	2.672	1.212	1.212	0.641	1.853	2.892	0.359	0.354	0.296	0.606	1.247	$\bar{\sigma}_r = \bar{\sigma}_3$		0.606	1.247	0.152	0.312
327:00	3.417	1.266	1.266	0.646	1.912	2.960	0.354	0.354	0.279	0.633	1.279			0.633	1.279	0.158	0.320
387:00	4.152	1.290	1.290	0.648	1.939	2.989	0.352	0.352	0.272	0.645	1.294		$\bar{\sigma}_a = \bar{\sigma}_1$	0.645	1.294	0.161	0.324
447:00	4.903	1.309	1.309	0.643	1.953	3.035	0.357	0.357	0.272	0.655	1.298			0.655	1.298	0.164	0.325
507:00	5.652	1.305	1.305	0.643	1.948	3.028	0.357	0.357	0.273	0.652	1.296			0.652	1.296	0.163	0.324
567:00	6.416	1.332	1.332	0.630	1.962	3.112	0.395	0.395	0.277	0.666	1.296			0.666	1.296	0.167	0.324
627:00	7.215	1.334	1.334	0.625	1.959	3.134	0.375	0.375	0.281	0.667	1.293			0.667	1.293	0.167	0.323
687:00	8.000	1.339	1.339	0.612	1.952	3.187	0.387	0.387	0.289	0.669	1.282			0.669	1.282	0.167	0.321
747:00	8.838	1.329	1.329	0.607	1.936	3.189	0.393	0.393	0.296	0.665	1.272			0.665	1.272	0.165	0.318
867:00	10.429	1.318	1.319	0.589	1.908	3.238	0.411	0.411	0.312	0.659	1.249			0.659	1.249	0.165	0.312
987:00	11.987	1.304	1.304	0.579	1.883	3.253	0.421	0.421	0.323	0.652	(23)			0.652	1.231	0.163	0.308
1107:00	13.564	1.296	1.296	0.574	1.869	3.259	0.426	0.426	0.329	0.648	1.221			0.648	1.221	0.162	0.305
1227:00	15.118	1.266	1.266	0.571	1.837	3.217	0.429	0.429	0.339	0.633	1.204	↓		0.633	1.204	0.158	0.301

(1) CORRECTED FOR _____

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS; $A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

REMARKS:

RECOMPRESSION SAMPLE.

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT. OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. ARKC-1 (TC)
SOIL Amherst. Vc.
PROJECT 80142.
TESTED BY _____ DATE _____
ALL STRESSES IN kg/cm².

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL						
PRESHEAR						

G_s = _____ TYPE CELL _____

PRESHEAR: $\bar{\sigma}_{1c}$ = 1.00 t_c = 1 day
$\bar{\sigma}_{3c}$ = 0.75 P.P.R. = – %
$\bar{\sigma}_{ac}$ = 1.00 u_B = 2.00
PRESHEAR STRESS HISTORY consolidated Anisotropically below $\bar{\sigma}_{vm}$. ($\bar{\sigma}_{vm}$ = 1.60 kg/cm²).

DURING SHEAR: CONTROLLED STRAIN ✓ STRESS _____
RATE 1.0 %/hr
PATH compression loading.

ELAPSED TIME	AXIAL STRAIN, %	$(\bar{\sigma}_1 - \bar{\sigma}_3)$ (1)	$\frac{(\bar{\sigma}_1 - \bar{\sigma}_3)}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu (2)	$\frac{\Delta u}{\bar{\sigma}_{1c}}$ (2)	A (3)	q	p̄	$\bar{\sigma}_r$	$\bar{\sigma}_c$	$q/\bar{\sigma}_{vc}$	$\bar{p}/\bar{\sigma}_{vc}$	$q/\bar{\sigma}_{vm}$	$\bar{p}/\bar{\sigma}_{vm}$
	0	0.250	0.250	0.750	1.000	1.333	0	0	–	0.125	0.875	0.750	1.000	0.125	0.875	0.078	0.547
	0.3	0.500	0.500	0.550	1.050	1.909	0.20	0.200	0.800	0.250	0.800	0.550	1.050	0.250	0.800	0.156	0.500
	0.6	0.560	0.560	0.470	1.030	2.191	0.28	0.280	0.903	0.280	0.750	0.470	1.030	0.280	0.750	0.175	0.469
	1.0	0.680	0.680	0.395	1.075	2.722	0.355	0.355	0.876	0.340	0.735	0.395	1.075	0.340	0.735	0.213	0.459
	2.0	0.720	0.720	0.350	1.070	3.057	0.40	0.400	0.851	0.360	0.710	0.350	1.070	0.360	0.710	0.225	0.444
	3.0	0.750	0.750	0.300	1.050	3.500	0.450	0.450	0.900	0.375	0.675	0.300	1.050	0.375	0.675	0.234	0.422
	4.0	0.760	0.760	0.310	1.070	3.452	0.440	0.440	0.863	0.380	0.690	0.310	1.070	0.380	0.690	0.238	0.431
	5.0	0.740	0.740	0.300	1.040	3.467	0.450	0.450	0.918	0.370	0.670	0.300	1.040	0.370	0.670	0.231	0.419
	5.9	0.740	0.740	0.300	1.040	3.467	0.450	0.450	0.918	0.370	0.670	0.300	1.040	0.370	0.670	0.231	0.419
	7.5	0.730	0.730	0.310	1.040	3.355	0.440	0.440	0.917	0.365	0.675	0.310	1.040	0.365	0.675	0.228	0.422
	8.5	0.710	0.710	0.320	1.030	3.219	0.430	0.430	0.935	0.355	0.675	0.320	1.030	0.355	0.675	0.222	0.422

(1) CORRECTED FOR _____

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

REMARKS:

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT. OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. AKE-1 (TE)
SOIL Amherst Varved Clay
PROJECT 80142
TESTED BY SAM. DATE 22 April 1975
ALL STRESSES IN kg/cm².

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL	67.8			79.2	7.78	9.93
PRESHEAR	53.5			68.7	7.06	9.73

G_s = TYPE CELL CLO

PRESHEAR: σ_{ic} = 1.900, t_c = 1 day; σ_{3c} = 1.477, P.P.R. = - %; σ_{oc} = 1.900, u_b = 2.00

DURING SHEAR: CONTROLLED STRAIN ✓ STRESS ____; RATE 0.7%/hr; PATH EXTENSION UNLOADING

PRESHEAR STRESS HISTORY Ko Consolidation to $\bar{\sigma}_{vc}$ = 3.6 kg/cm² to 1.900 kg/cm².

ELAPSED TIME	AXIAL STRAIN, %	$\sigma_v - \sigma_h$	$\frac{\sigma_v - \sigma_h}{\bar{\sigma}_{vc}}$	$\bar{\sigma}_v$	$\bar{\sigma}_h$	$\bar{\sigma}_h/\bar{\sigma}_v$	Δu (2)	$\frac{\Delta u - \Delta\sigma_3}{\bar{\sigma}_{vc}}$	A (3)	q	β	σ_f	σ_3	$q/\bar{\sigma}_{vc}$	$\bar{p}/\bar{\sigma}_{vc}$	$q/\bar{\sigma}_{vm}$	$\bar{p}/\bar{\sigma}_{vm}$
	0	-0.423	0.223	1.900	1.477	0.777	0	0	-	0.211	1.689	1.477	1.900	+0.113	0.889	+0.059	0.445
	0.032	0.283	0.149	1.902	1.619	0.851	-0.142	-0.001	-.014	0.241	1.761	1.619	1.902	+0.074	0.927	0.037	0.464
	0.083	0.140	0.074	1.860	1.720	0.925	-0.243	0.021	0.141	0.070	1.790	1.720	1.860	+0.037	0.942	0.019	0.471
	0.363	-0.206	-0.108	1.749	1.955	1.118	-0.478	0.079	0.240	-0.103	1.852	1.955	1.749	-0.054	0.975	-0.027	0.488
	0.489	-0.305	-0.161	1.709	2.014	1.178	-0.537	0.100	0.262	-0.152	1.862	2.014	1.709	-0.080	0.979	-0.040	0.488
	0.593	-0.416	-0.219	1.665	2.081	1.250	-0.604	0.124	0.280	-0.208	1.873	2.081	1.665	-0.109	0.956	-0.055	0.478
	0.774	-0.511	-0.269	1.616	2.127	1.316	-0.650	0.150	0.304	-0.256	1.872	2.127	1.616	-0.135	0.985	-0.068	0.493
	0.993	-0.603	-0.317	1.558	2.161	1.387	-0.684	0.180	0.333	-0.302	1.860	2.161	1.558	-0.159	0.979	-0.080	0.490
	1.13	-0.653	-0.344	1.517	2.170	1.430	-0.693	0.176	0.356	-0.327	1.844	2.170	1.517	-0.172	0.970	-0.086	0.485
	1.29	-0.692	-0.364	1.488	2.180	1.465	-0.703	0.217	0.370	-0.346	1.834	2.180	1.488	-0.182	0.965	-0.091	0.483
	1.69	-0.801	-0.422	1.380	2.181	1.580	-0.704	0.273	0.425	-0.400	1.781	2.181	1.380	-0.211	0.937	-0.106	0.469
	2.79	-0.998	-0.525	1.158	2.156	1.862	-0.679	0.391	0.522	-0.499	1.657	2.156	1.158	-0.261	0.872	-0.131	0.436
	6.24	-1.245	-0.655	0.807	2.052	2.543	-0.575	0.575	0.655	-0.623	1.429	2.052	0.807	-0.328	0.752	-0.164	0.376
	8.21	-1.307	-0.688	0.720	2.072	2.815	-0.550	0.622	0.682	-0.654	1.374	2.072	0.720	-0.344	0.723	-0.172	0.362
	9.52	-1.350	-0.711	0.664	2.014	3.033	-0.537	0.650	0.697	-0.675	1.339	2.014	0.664	-0.355	0.705	-0.178	0.353
	10.37	-1.367	-0.720	0.646	2.013	3.116	-0.536	0.660	0.701	-0.684	1.329	2.013	0.646	-0.359	0.699	-0.180	0.350
	11.23	-1.379	-0.726	0.633	2.012	3.179	-0.535	0.666	0.703	-0.689	1.323	2.012	0.633	-0.363	0.696	-0.182	0.348
	11.67	-1.390	-0.732	0.610	2.000	3.279	-0.536	0.672	0.704	-0.695	1.305	2.000	0.610	-0.366	0.697	-0.183	0.349
	12.53	1.430	-0.753	0.586	2.017	3.442	-0.540	0.694	0.709	-0.715	1.302	2.017	0.586	-0.377	0.685	-0.189	0.343

(1) CORRECTED FOR ____

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_3}{\Delta\sigma_r - \Delta\sigma_3}$ FOR EXTENSION TESTS

REMARKS:

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT. OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. AKE-2 (TE)
SOIL Amherst VARVED CLAY
PROJECT 80142
TESTED BY JJL DATE 17 Feb 1975
ALL STRESSES IN kg/cm².

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL	65.3			79.2	7.78	9.93
PRESHEAR	51.2			68.80	7.03	9.77

G_s = _____ TYPE CELL CLO

PRESHEAR: σ_{1c} = 0.900 t_c = 1 day; σ_{3c} = 0.900 P.P.R. = _____ %; σ_{oct} = _____ u_B = 2.00

DURING SHEAR: CONTROLLED STRAIN ✓ STRESS _____ RATE 0.7%/hr PATH EXTENSION UNLOADING

PRESHEAR STRESS HISTORY Ko consolidated to 4.10 kg/cm² and rebound to 0.900 kg/cm².

ELAPSED TIME	AXIAL STRAIN, %	$\sigma_v - \sigma_H$	$\frac{\sigma_v - \sigma_H}{\sigma_{vc}}$	$\bar{\sigma}_v$	$\bar{\sigma}_H$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu (2)	$\frac{\Delta u - \Delta\sigma_3}{\sigma_{vc}}$	A (3)	q	p	σ_f	σ_3	q/σ_{vc}	$\bar{p}/\sigma_{vc}$	q/σ_{ym}	$\bar{p}/\sigma_{ym}$
0	0	0	0	0.900	0.900	1.000	0	0	—	0	0.900	0.900	0.900	0	1.000	0	0.250
2.0	0.005	-0.031	-0.034	0.888	0.919	1.034	-0.019	0.013	0.387	-0.015	0.904	0.919	0.888	-0.017	1.040	-0.004	0.260
8.0	0.033	-0.112	-0.124	0.862	0.974	1.130	-0.074	0.042	0.339	-0.056	0.918	0.974	0.862	-0.062	1.026	-0.016	0.255
18.0	0.117	-0.233	-0.258	0.828	1.061	1.280	-0.161	0.079	0.308	-0.116	0.945	1.061	0.828	-0.129	1.049	-0.032	0.262
30.0	0.214	-0.347	-0.385	0.802	1.149	1.433	-0.249	0.109	0.282	-0.173	0.976	1.149	0.802	-0.193	1.084	-0.048	0.271
50.0	0.402	-0.490	-0.544	0.778	1.268	1.629	-0.368	0.135	0.268	-0.245	1.023	1.268	0.778	-0.272	1.137	-0.068	0.284
69.0	0.635	-0.571	-0.635	0.766	1.338	1.745	-0.438	0.149	0.234	-0.286	1.052	1.338	0.766	-0.317	1.168	-0.079	0.292
89.0	0.884	-0.625	-0.694	0.751	1.376	1.832	-0.476	0.166	0.239	-0.312	1.063	1.376	0.751	-0.347	1.181	-0.087	0.295
109.0	1.691	-0.743	-0.826	0.702	1.446	2.058	-0.546	0.226	0.266	-0.372	1.074	1.446	0.702	-0.413	1.194	-0.103	0.299
209.0	2.572	-0.819	-0.911	0.664	1.483	2.232	-0.583	0.262	0.288	-0.410	1.074	1.483	0.664	-0.455	1.193	-0.114	0.298
269.0	3.400	-0.876	-0.974	0.631	1.507	2.387	-0.607	0.299	0.307	-0.438	1.069	1.507	0.631	-0.487	1.188	-0.122	0.297
329.0	4.209	-0.919	-1.021	0.612	1.531	2.580	-0.631	0.319	0.313	-0.459	1.072	1.531	0.612	-0.510	1.191	-0.128	0.298
389.0	5.053	-0.953	-1.059	0.598	1.552	2.591	-0.652	0.335	0.316	-0.477	1.076	1.552	0.598	-0.529	1.195	-0.132	0.299
449.0	5.916	-0.989	-1.099	0.585	1.574	2.695	-0.674	0.351	0.319	-0.495	1.079	1.574	0.585	-0.549	1.199	-0.137	0.300
509.0	6.812	-1.026	-1.139	0.568	1.594	2.809	-0.694	0.369	0.324	-0.513	1.081	1.594	0.568	-0.569	1.201	-0.142	0.300
569.0	7.662	-1.055	-1.173	0.557	1.612	2.890	-0.712	0.381	0.325	-0.528	1.085	1.612	0.557	-0.586	1.205	-0.147	0.301
629.0	8.502	-1.073	-1.193	0.558	1.631	2.924	-0.731	0.379	0.318	-0.537	1.095	1.631	0.558	-0.596	1.216	-0.149	0.304
689.0	9.339	-1.091	-1.212	0.559	1.650	2.950	-0.750	0.379	0.313	-0.546	1.104	1.650	0.559	-0.606	1.227	-0.152	0.307
750.0	10.234	-1.1125	-1.236	0.553	1.665	3.012	-0.765	0.386	0.312	-0.556	1.109	1.665	0.553	-0.618	1.232	-0.155	0.308
811.0	11.126	-1.132	-1.257	0.547	1.679	3.067	-0.779	0.392	0.312	-0.566	1.113	1.679	0.547	-0.629	1.237	-0.157	0.309
871.0	12.006	-1.138	-1.264	0.542	1.680	3.106	-0.780	0.398	0.314	-0.569	1.111	1.680	0.542	-0.632	1.235	-0.158	0.309
991.0	13.781	-1.035	-1.149	0.585	1.619	2.770	-0.720	0.349	0.304	-0.517	1.103	1.619	0.585	-0.575	1.225	-0.144	0.306
1111.0	15.614	-0.911	-1.013	0.603	1.514	2.513	-0.683	0.330	0.326	-0.456	1.059	1.514	0.603	-0.506	1.176	-0.127	0.294
1231.0	17.354	-0.851	-0.945	0.597	1.448	2.427	-0.609	0.336	0.356	-0.425	1.023	1.448	0.597	-0.473	1.136	-0.118	0.284
1351.0	19.144	-0.962	-0.847	0.616	1.378	2.237	-0.531	0.315	0.372	-0.381	0.997	1.378	0.616	-0.423	1.108	-0.106	0.277

(1) CORRECTED FOR _____

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

REMARKS:

SECTION D.1-3

TABULATION OF TEST DATA FROM $\overline{CK_oU}$DSS TESTS ON NC AND OC SHANSEP AND RECOMPRESSION AMHERST SAMPLES

DIRECT - SIMPLE SHEAR TEST

PROJECT 80142 TYPE OF TEST CK_oU NO. ADS 1 OCR 1.0

SOIL TYPE VARVED CLAY TESTED BY FDL & V.V. DEVICE _____ DATE 6/26/73

LOCATION AMHERST
BORING B-2
SAMPLE U-6
TEST NO: ADS-1

CONSOLIDATION (Stresses in Kg/cm^2)

$\bar{\sigma}_{vc}$ 4.0 τ_{hc} _____ $\bar{\sigma}_{vm}$ _____

t_c(Day) _____ ε_v(%) _____ γ_c(%) _____ t_c(Day) _____

	W,%	e	S,%	H (in)
Initial	61.9	1.666	37	0.875
Preshear	46.8	1.295	103	0.739
Final	48.4	1.353	103	0.7685

DURING SHEAR
Controlled Strain ✓ Stress _____
Rate (% / Hr.) 5.5

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\bar{\sigma}_{vc}}$	$\frac{\Delta u}{\bar{\sigma}_{vc}}$	$\frac{\bar{\sigma}_v}{\bar{\sigma}_{vc}}$	$\frac{\tau_h}{\bar{\sigma}_v}$	$\frac{\tau_h}{\bar{\sigma}_{vm}}$	$\frac{\bar{\sigma}_v}{\bar{\sigma}_{vm}}$	REMARKS
8:45	0	0	0	1.000	0			
8:58	0.03	0.026	0.028	0.972	0.026			
9:05	0.05	0.040	0.043	0.957	0.042			
9:11	0.11	0.056	0.057	0943	0.061			
9:15	0.16	0.065	0.081	0.919	0.076			
	0.21	0.079	0.096	0.904	0.090			
9:24	0.32	0.094	0.111	0889	0.110			
9:28	0.43	0.105	0.129	0.871	0.126			
	0.53	0.113	0.141	0.859	0.139			
9.36	0.69	0.124	0.144	0.856	0.153			
	0.85	0.131	0.178	0.822	0.164			
9.44	1.07	0.139	0.194	0.806	0.174			
	1.33	0.145	0.215	0.785	0.185			
9.52	1.60	0.150	0.229	0.771	0.195			
	2.13	0.158	0.253	0.747	0.211			
10:06	2.66	0.162	0.279	0.721	0 225			
10:12	3 20	0.166	0.301	0.699	0.237			
	4.00	0.166	0.334	0.666	0.250			
	4.80	0.169	0.364	0.636	0.265			
	5.33	0.169	0.378	0.622	0.272			
	6.40	0.170	0.411	0.589	0.289			
	7.46	0.171	0.436	0.564	0.303			
	8.53	0.171	0.443	0.557	0.307			
	9.59	0.171	0.468	0.532	0.321			
11:30	10.66	0.170	0.486	0.514	0.331			

SOIL MECHANICS LABORATORY
DEPT. OF CIVIL ENGINEERING
MASSACHUSETTS INSTITUTE OF TECHNOLOGY

REMARKS:

DIRECT - SIMPLE SHEAR (continued)

PROJECT 8C142 SOIL AMHERST VARVED CLAY TYPE OF TEST DSS NO. 1

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\bar{\sigma}_{vc}}$	$\frac{\Delta u}{\bar{\sigma}_{vc}}$	$\frac{\bar{\sigma}_v}{\bar{\sigma}_{vc}}$	$\frac{\tau_h}{\bar{\sigma}_v}$	$\frac{\tau_h}{\bar{\sigma}_{vm}}$	$\frac{\bar{\sigma}_v}{\bar{\sigma}_{vm}}$	REMARKS
11:42	11.99	0.170	0.507	0.493	0.344			
11:55	13.32	0.168	0.520	0.480	0.351			
	14.66	0.167	0.537	0.463	0.361			
	15.99	0.166	0.546	0.454	0.366			
	17.32	0.164	0.564	0.436	0.377			
	18.65	0.162	0.576	0.424	0.382			
13:01	19.99	0.161	0.581	0.419	0.383			
	21.32	0.160	0.596	0.404	0.395	max obliquity		
	22.65	0.153	0.597	0.403	0.380			
	23.98	0.144	0.640	0.360	0.399			
	26.12	0.129	0.683	0.317	0.408			
	26.65	0.127	0.688	0.312	0.406			
	27.98	0.119	0.706	0.294	0.405			
	29.31	0.114	0.725	0.275	0.413			

SOIL MECHANICS LABORATORY
DEPT. OF CIVIL ENGINEERING
MASSACHUSETTS INSTITUTE OF TECHNOLOGY

REMARKS:

DIRECT - SIMPLE SHEAR TEST

PROJECT 80142 TYPE OF TEST $\overline{CK_0U}$ NO. ADS2 OCR 1.0

SOIL TYPE VARVED CLAY TESTED BY F.D.L DEVICE _____ DATE 7/16/73

LOCATION AMHERST
BORING B-2
SAMPLE U-6
ADS-2

CONSOLIDATION (Stresses in Kg/cm^2)

$\overline{\sigma}_{vc}$ 3.00 τ_{hc} _____ $\overline{\sigma}_{vm}$ _____

t_c(Day) _____ ε_v(%) _____ γ_c(%) _____ t_c(Day) _____

	W,%	e	S,%	H (in)
Initial	60.0	1.786	95.4	0.932
Preshear	51.5	1.412	100	0.807
Final	53.0	1.453	100	0.821

DURING SHEAR
Controlled Strain ✓ Stress _____
Rate (% / Hr.) 5.5

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\overline{\sigma}_{vc}}$	$\frac{\Delta u}{\overline{\sigma}_{vc}}$	$\frac{\overline{\sigma}_v}{\overline{\sigma}_{vc}}$	$\frac{\tau_h}{\overline{\sigma}_v}$	$\frac{\tau_h}{\overline{\sigma}_{vm}}$	$\frac{\overline{\sigma}_v}{\overline{\sigma}_{vm}}$	REMARKS
	0	0	0	1.00	0			
8:40	0.03	0.032	0.002	0.998	0.032			
	0.05	0.049	0.037	0.963	0.049			
	0.10	0.065	0.062	0.938	0.068			
	0.15	0.075	0.105	0.895	0.083			
9:01	0.20	0.084	0.121	0.879	0.095			
	0.30	0.099	0.128	0.872	0.112			
	0.40	0.110	0.132	0.868	0.125			
	0.50	0.118	0.140	0.860	0.135			
9:19	0.75	0.123	0.149	0.851	0.143			
	1.00	0.126	0.166	0.834	0.149			
	1.25	0.128	0.190	0.810	0.156			
	1.50	0.130	0.204	0.796	0.164			
	2.00	0.134	0.226	0.774	0.173			
	2.51	0.136	0.251	0.749	0.182			
	3.01	0.139	0.278	0.722	0.193			
10:04	3.51	0.141	0.302	0.698	0.202			
10:12	4.01	0.143	0.326	0.674	0.212			
	4.51	0.146	0.344	0.656	0.222			
	5.01	0.148	0.382	0.618	0.239			
	7.01	0.153	0.436	0.564	0.271			
	8.02	0.157	0.460	0.540	0.291			
	9.02	0.159	0.481	0.519	0.306			
	10.02	0.159	0.497	0.503	0.316			
	11.27	0.157	0.522	0.478	0.329			

SOIL MECHANICS LABORATORY
DEPT. OF CIVIL ENGINEERING
MASSACHUSETTS INSTITUTE OF TECHNOLOGY

REMARKS:

DIRECT - SIMPLE SHEAR (continued)

PROJECT 80142 SOIL AMHERST VARVED CLAY TYPE OF TEST CKoU NO. 2

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\sigma_{vc}}$	$\frac{\Delta u}{\sigma_{vc}}$	$\frac{\bar{\sigma}_v}{\sigma_{vc}}$	$\frac{\tau_h}{\bar{\sigma}_v}$	$\frac{\tau_h}{\sigma_{vm}}$	$\frac{\bar{\sigma}_v}{\sigma_{vm}}$	REMARKS
	12.53	0.156	0.546	0.454	0.344			
	13.78	0.155	0.557	0.443	0.351			
	15.03	0.153	0.582	0.418	0.367			
	16.28	0.151	0.583	0.417	0.362			
	17.54	0.149	0.598	0.402	0.369			
	18.79	0.146	0.603	0.397	0.368			
	20.04	0.142	0.613	0.387	0.366			
10:46	21.29	0.141	0.624	0.376	0.374			
	22.55	0.140	0.632	0.368	0.380			
	23.80	0.138	0.640	0.360	0.385			
	25.05	0.135	0.653	0.347	0.390			
	26.30	0.131	0.671	0.329	0.398			
	27.56	0.125	0.684	0.316	0.393			
	28.81	0.119	0.701	0.299	0.398			
	30.06	0.114	0.716	0.284	0.403			
	30.70	0.111	0.732	0.268	0.415			

SOIL MECHANICS LABORATORY
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MASSACHUSETTS INSTITUTE OF TECHNOLOGY

REMARKS:

DIRECT - SIMPLE SHEAR TEST

PROJECT 80142 TYPE OF TEST CKoU NO. ADS 3 OCR 1.50

SOIL TYPE VARVED CLAY TESTED BY F.D.L DEVICE _____ DATE 7/23/73

LOCATION AMHERST
BORING B-2
SAMPLE U-6
ADS-3 (RECOMPRESSION)

CONSOLIDATION (Stresses in Kg/cm^2)

$\bar{\sigma}_{vc}$ 1.08 τ_{hc} _____ $\bar{\sigma}_{vm}$ _____

t_c(Day) _____ ε_v(%) _____ γ_c(%) _____ t_c(Day) _____

	W,%	e	S,%	H (in)
Initial	67.7	1.931	99.6	0.851
Preshear	63.8	1.586	114.3	0.751
Final	63.4	1.583	114.5	0.750

DURING SHEAR
Controlled Strain ✓ Stress _____
Rate (% / Hr.) 5.5

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\bar{\sigma}_{vc}}$	$\frac{\Delta u}{\bar{\sigma}_{vc}}$	$\frac{\bar{\sigma}_v}{\bar{\sigma}_{vc}}$	$\frac{\tau_h}{\bar{\sigma}_v}$	$\frac{\tau_h}{\bar{\sigma}_{vm}}$	$\frac{\bar{\sigma}_v}{\bar{\sigma}_{vm}}$	REMARKS
8:55	0	0	0	1.00	0	0	0	
	0.025	0.039	0.019	0.981	0.040	0.03	0.65	
9:01	0.05	0.057	0.034	0.966	0.059	0.038	0.644	
	0.10	0.080	0.030	0.970	0.082			
	0.15	0.086	0.072	0.928	0.093			
9:14	0.20	0.095	0.142	0.858	0.110			
	0.30	0.109	0.194	0.806	0.135	0.073	0.537	
	0.40	0.122	0.213	0.787	0.155			
9:20	0.50	0.131	0.220	0.780	0.168			
	0.65	0.140	0.235	0.765	0.184			
9:29	0.95	0.156	0.235	0.765	0.204	0.104	0.510	
	1.20	0.165	0.246	0.754	0.219			
	1.50	0.171	0.261	0.739	0.232			
	1.80	0.178	0.280	0.720	0.248			
	2.15	0.182	0.295	0.705	0.258	0.121	0.470	
	2.51	0.192	0.306	0.694	0.276			
	3.01	0.197	0.321	0.679	0.290			
	3.51	0.194	0.344	0.656	0.296	0.129	0.437	
	4.01	0.197	0.379	0.621	0.317			
	4.51	0.199	0.391	0.609	0.327			
	5.01	0.200	0.409	0.591	0.338			
	5.51	0.200	0.426	0.572	0.349	0.133	0.381	
	6.26	0.200	0.436	0.564	0.354			
	6.51	0.199	0.469	0.531	0.374			
	7.26	0.197	0.474	0.526	0.375			

SOIL MECHANICS LABORATORY
DEPT. OF CIVIL ENGINEERING
MASSACHUSETTS INSTITUTE OF TECHNOLOGY

REMARKS:

DIRECT - SIMPLE SHEAR (continued)

PROJECT _____ SOIL AMHERST VARVED CLAY TYPE OF TEST CKoU NO. 3

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\sigma_{vc}}$	$\frac{\Delta u}{\sigma_{vc}}$	$\frac{\bar{\sigma}_v}{\sigma_{vc}}$	$\frac{\tau_h}{\bar{\sigma}_v}$	$\frac{\tau_h}{\sigma_{vm}}$	$\frac{\bar{\sigma}_v}{\sigma_{vm}}$	REMARKS
	8.02	0.196	0.488	0.512	0.383			
	9.52	0.196	0.510	0.490	0.400			
	10.02	0.195	0.525	0.475	0.411			
	11.52	0.192	0.562	0.438	0.438			
	13.03	0.189	0.569	0.431	0.439			
	15.03	0.150	0.602	0.398	0.453			
	17.34	0.169	0.636	0.364	0.464			
	20.54	0.156	0.687	0.313	0.500			
	22.55	0.144	0.706	0.294	0.489			
13:32	23.80	0.141	0.729	0.271	0.521			
	25.05	0.137	0.739	0.261	0.525			
13:58	26.30	0.133	0.747	0.253	0.524			
	27.56	0.129	0.758	0.242	0.529			
	28.81	0.125	0.773	0.227	0.584			
	30.06	0.117	0.790	0.210	0.597			

SOIL MECHANICS LABORATORY
DEPT. OF CIVIL ENGINEERING
MASSACHUSETTS INSTITUTE OF TECHNOLOGY

REMARKS:

DIRECT - SIMPLE SHEAR TEST

PROJECT _____ TYPE OF TEST CKoU NO. ADS 4 OCR 3.0

SOIL TYPE VARVED CLAY TESTED BY F.D.L DEVICE _____ DATE 8/3/73

LOCATION AMHERST
BORING B-2
SAMPLE U-6
ADS-4

CONSOLIDATION (Stresses in Kg/cm^2)

$\bar{\sigma}_{vc}$ 1.0 τ_{hc} _____ $\bar{\sigma}_{vm}$ 3.0

t_c(Day) _____ ε_v(%) _____ γ_c(%) _____ t_c(Day) _____

	W,%	e	S,%	H (in)
Initial	67.9			0.891
Preshear				
Final	54.3			

DURING SHEAR
Controlled Strain ✓ Stress _____
Rate (% / Hr.) 5.5

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\bar{\sigma}_{vc}}$	$\frac{\Delta u}{\bar{\sigma}_{vc}}$	$\frac{\bar{\sigma}_v}{\bar{\sigma}_{vc}}$	$\frac{\tau_h}{\bar{\sigma}_v}$	$\frac{\tau_h}{\bar{\sigma}_{vm}}$	$\frac{\bar{\sigma}_v}{\bar{\sigma}_{vm}}$	REMARKS
7:57	0	0	0	1.0	0	0	0.346	
	0.026	0.016	0	1.0	0.016	0.006	0.346	
	0.052	0.039	-0.024	1.024	0.039	0.014	0.354	
	0.104	0.067	-0.040	1.040	0.065	0.023	0.360	
	0.157	0.086	-0.044	1.044	0.082	0.030	0.361	
	0.209	0.099	-0.057	1.057	0.094	0.034	0.366	
	0.261	0.121	-0.066	1.066	0.115	0.042	0.369	
	0.365	0.150	-0.073	1.073	0.140	0.052	0.371	
	0.522	0.182	-0.093	1.093	0.167	0.063	0.378	
	0.731	0.211	-0.106	1.106	0.191	0.073	0.383	
	1.04	0.243	-0.126	1.126	0.215	0.084	0.390	
	1.31	0.261	-0.143	1.143	0.228	0.090	0.395	
	1.57	0.276	-0.150	1.150	0.240	0.095	0.398	
	1.93	0.293	-0.163	1.163	0.252	0.101	0.402	
	2.40	0.311	-0.174	1.174	0.265	0.108	0.406	
	3.03	0.327	-0.191	1.191	0.274	0.113	0.412	
	3.65	0.337	-0.191	1.191	0.283	0.117	0.412	
	4.18	0.346	-0.191	1.191	0.290	0.120	0.412	
	4.70	0.352	-0.191	1.191	0.295	0.122	0.412	
	5.22	0.356	-0.187	1.187	0.300	0.123	0.411	
	5.74	0.359	-0.163	1.163	0.309	0.124	0.402	
	6.26	0.368	-0.199	1.199	0.307	0.127	0.415	
	6.79	0.372	-0.179	1.179	0.315	0.129	0.408	
	7.31	0.377	-0.179	1.179	0.319	0.130	0.408	
	7.83	0.382	-0.179	1.179	0.324	0.132	0.408	

SOIL MECHANICS LABORATORY
DEPT. OF CIVIL ENGINEERING
MASSACHUSETTS INSTITUTE OF TECHNOLOGY

REMARKS:

DIRECT - SIMPLE SHEAR (continued)

PROJECT _____ SOIL AMHERST VARVED CLAY TYPE OF TEST CKoU NO. 4

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\sigma_{vc}}$	$\frac{\Delta u}{\sigma_{vc}}$	$\frac{\bar{\sigma}_v}{\sigma_{vc}}$	$\frac{\tau_h}{\bar{\sigma}_v}$	$\frac{\tau_h}{\bar{\sigma}_{vm}}$	$\frac{\bar{\sigma}_v}{\bar{\sigma}_{vm}}$	REMARKS
	8.35	0.386	−0.174	1.174	0.329	0.134	0.406	
	8.87	0.386	−0.143	1.143	0.335	0.134	0.395	
	9.40	0.389	−0.143	1.143	0.340	0.135	0.395	
	9.92	0.392	−0.143	1.143	0.343	0.136	0.395	
	10.44	0.392	−0.111	1.111	0.353	0.136	0.385	
	10.96	0.393	−0.111	1.111	0.353	0.136	0.385	
	11.75	0.394	−0.111	1.111	0.354	0.136	0.385	
	12.27	0.396	−0.093	1.093	0.362	0.137	0.378	←
	12.53	0.393	−0.066	1.066	0.369	0.136	0.369	
	13.36	0.393	−0.061	1.061	0.371	0.136	0.367	
	15.91	0.391	−0.053	1.053	0.371	0.135	0.364	
	17.28	0.390	0.016	0.984	0.397	0.135	0.340	
	18.58	0.381	0.040	0.960	0.397	0.132	0.332	
	20.08	0.378	0.069	0.931	0.406	0.131	0.332	
	21.40	0.375	0.077	0.923	0.406	0.130	0.319	
	23.33	0.365	0.106	0.894	0.408	0.126	0.309	
	25.06	0.359	0.138	0.862	0.417	0.124	0.298	
	26.36	0.356	0.150	0.850	0.420	0.123	0.294	
	28.19	0.343	0.187	0.813	0.422	0.119	0.281	
	29.75	0.334	0.207	0.793	0.422	0.116	0.274	
	31.32	0.321	0.264	0.736	0.436	0.111	0.255	
	32.89	0.311	0.313	0.687	0.453	0.108	0.238	
	34.71	0.301	0.326	0.674	0.446	0.104	0.233	
	36.02	0.290	0.366	0.634	0.457	0.100	0.219	
14:58	37.06	0.279	0.410	0.590	0.474	0.097	0.204	

SOIL MECHANICS LABORATORY
DEPT. OF CIVIL ENGINEERING
MASSACHUSETTS INSTITUTE OF TECHNOLOGY

REMARKS:

DIRECT - SIMPLE SHEAR TEST

PROJECT _____ TYPE OF TEST CKoU NO. ADS 5 OCR 6.0.

SOIL TYPE VARVED CLAY TESTED BY F.D.L DEVICE _____ DATE 8/13/73

LOCATION AMHERST
BORING B-2
SAMPLE U-4
ADS-5.

CONSOLIDATION (Stresses in Kg/cm^2)

$\bar{\sigma}_{vc}$ 0.535 τ_{hc} _____ $\bar{\sigma}_{vm}$ 3.0

t_c(Day) _____ ε_v(%) _____ γ_c(%) _____ t_c(Day) _____

	W,%	e	S,%	H (in)
Initial	59.2			0.838
Preshear				0.738
Final	51.6			

DURING SHEAR
Controlled Strain ✓ Stress _____
Rate (% / Hr.) 5.5

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\bar{\sigma}_{vc}}$	$\frac{\Delta u}{\bar{\sigma}_{vc}}$	$\frac{\bar{\sigma}_v}{\bar{\sigma}_{vc}}$	$\frac{\tau_h}{\bar{\sigma}_v}$	$\frac{\tau_h}{\bar{\sigma}_{vm}}$	$\frac{\bar{\sigma}_v}{\bar{\sigma}_{vm}}$	REMARKS
10:50	0	0	0	1.0	0	0	0.178	
	0.026	0.051	-0.017	1.017	0.059	0.009	0.181	
	0.052	0.069	-0.017	1.017	0.077	0.012	0.181	
10:58	0.156	0.125	-0.032	1.032	0.131	0.022	0.184	
	0.209	0.142	-0.039	1.039	0.146	0.025	0.185	
	0.313	0.166	-0.047	1.047	0.168	0.030	0.186	
	0.418	0.185	-0.056	1.056	0.184	0.033	0.188	
	0.522	0.202	-0.071	1.071	0.198	0.036	0.191	
	0.679	0.225	-0.096	1.096	0.214	0.040	0.195	
	0.992	0.260	-0.103	1.103	0.244	0.046	0.196	
	1.253	0.292	-0.150	1.150	0.262	0.052	0.205	
	1.566	0.322	-0.165	1.165	0.284	0.057	0.207	
	1.827	0.337	-0.228	1.228	0.282	0.060	0.219	
	2.088	0.354	-0.251	1.251	0.290	0.063	0.223	
	2.349	0.369	-0.260	1.260	0.300	0.066	0.224	
	2.610	0.391	-0.307	1.307	0.307	0.070	0.233	
11:42	3.132	0.421	-0.339	1.339	0.315	0.075	0.238	
	3.654	0.444	-0.378	1.378	0.322	0.079	0.245	
	4.176	0.463	-0.401	1.401	0.330	0.082	0.249	
	4.698	0.481	-0.418	1.418	0.339	0.086	0.252	
	5.533	0.504	-0.433	1.433	0.352	0.090	0.255	
	6.003	0.513	-0.489	1.489	0.345	0.091	0.265	
	6.316	0.519	-0.449	1.449	0.358	0.092	0.258	
	6.525	0.522	-0.449	1.449	0.360	0.093	0.258	
	7.047	0.536	-0.479	1.479	0.362	0.095	0.263	

SOIL MECHANICS LABORATORY
DEPT. OF CIVIL ENGINEERING
MASSACHUSETTS INSTITUTE OF TECHNOLOGY

REMARKS:

DIRECT - SIMPLE SHEAR (continued)

PROJECT _____ SOIL AMHERST VARVED CLAY TYPE OF TEST CK_oU NO. 5

TIME (Hr.)	STRAIN (%)	$\frac{\tau_h}{\sigma_{vc}}$	$\frac{\Delta u}{\sigma_{vc}}$	$\frac{\bar{\sigma}_v}{\sigma_{vc}}$	$\frac{\tau_h}{\bar{\sigma}_v}$	$\frac{\tau_h}{\bar{\sigma}_{vm}}$	$\frac{\bar{\sigma}_v}{\bar{\sigma}_{vm}}$	REMARKS
	7.308	0.539	-0.479	1.479	0.365	0.096	0.263	
	7.830	0.545	-0.479	1.479	0.368	0.097	0.263	
	8.352	0.552	-0.479	1.479	0.373	0.098	0.263	
	8.874	0.562	-0.479	1.479	0.380	0.100	0.263	
	9.396	0.571	-0.479	1.479	0.386	0.102	0.263	
	9.918	0.579	-0.479	1.479	0.391	0.103	0.263	
	10.440	0.584	-0.479	1.479	0.395	0.104	0.263	
	11.484	0.586	-0.489	1.489	0.394	0.104	0.265	
	12.53	0.594	-0.489	1.489	0.399	0.106	0.265	
	13.57	0.605	-0.489	1.489	0.406	0.108	0.265	
	14.62	0.609	-0.479	1.479	0.411	0.108	0.263	
	15.82	0.616	-0.489	1.489	0.414	0.110	0.265	
	17.23	0.618	-0.489	1.489	0.415	0.110	0.265	
	18.69	0.622	-0.464	1.464	0.425	0.111	0.261	
	21.45	0.616	-0.418	1.418	0.435	0.110	0.252	
	23.13	0.599	-0.346	1.346	0.445	0.107	0.240	
	25.32	0.552	-0.174	1.174	0.470	0.098	0.209	
	26.62	0.500	-0.125	1.125	0.444	0.089	0.200	
	28.71	0.496	-0.039	1.039	0.477	0.088	0.185	
	30.54	0.472	0.007	0.993	0.475	0.084	0.177	
	31.37	0.459	0.101	0.899	0.510	0.082	0.160	
	33.51	0.444	0.115	0.875	0.508	0.079	0.156	
	35.60	0.419	0.165	0.835	0.502	0.075	0.149	
	37.06	0.404	0.219	0.781	0.518	0.072	0.139	

SOIL MECHANICS LABORATORY
DEPT. OF CIVIL ENGINEERING
MASSACHUSETTS INSTITUTE OF TECHNOLOGY

REMARKS:

SECTION D.2-1

TABULATION OF TEST DATA FROM $\overline{CIUC}$ AND $\overline{CIUE}$ TESTS ON NC AND OC SHANSEP EAST WINDSOR SAMPLES

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. EIC-1-1

SOIL EAST WINDSOR VARVED CLAY

PROJECT 80142

TESTED BY SAME JNL DATE 6 April 1975

ALL STRESSES IN kg/cm².

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL				79.10	7.97	9.93
PRESHEAR				71.15	7.38	9.64

G_s = _____ TYPE CELL GEO

PRESHEAR: $\bar{\sigma}_{1c}$ = 4.0, t_c = 2 days; $\bar{\sigma}_{3c}$ = 4.0, P.P.R. = — %; $\bar{\sigma}_{ac}$ = 4.0, u_s = 2.0

PRESHEAR STRESS HISTORY Isotropic consolidated to $\bar{\sigma}_{vc}$ = 4.0 kg/cm²

DURING SHEAR: CONTROLLED STRAIN ✓ STRESS ____ RATE 0.7%/hr PATH compression loading

ELAPSED TIME	AXIAL STRAIN, %	$(\bar{\sigma}_1 - \bar{\sigma}_3)$ (1)	$\frac{(\bar{\sigma}_1 - \bar{\sigma}_3)}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1/\bar{\sigma}_3$	Δu (2)	$\frac{\Delta u}{\bar{\sigma}_{1c}}$ (2)	A (3)	q	$\bar{p}$	$\bar{\sigma}_r$	$\bar{\sigma}_c$	$\frac{q}{\bar{\sigma}_{vc}}$	$\frac{\bar{p}}{\bar{\sigma}_{vc}}$	$\frac{q}{\bar{\sigma}_{vm}}$	$\frac{\bar{p}}{\bar{\sigma}_{vm}}$
0	0	0	0	4.000	4.000	1.000	0	0	—	0	4.000	↓	↓	0	1.000	0	1.000
18	0.031	0.031	0.089	3.664	4.019	1.097	0.336	0.084	0.948	0.177	3.841			0.044	0.960	0.044	0.960
27	0.055	0.055	0.124	3.553	4.050	1.140	0.447	0.112	0.899	0.248	3.802			0.062	0.950	0.062	0.950
39	0.093	0.093	0.173	3.390	4.084	1.205	0.609	0.152	0.879	0.347	3.737			0.087	0.934	0.087	0.934
51	0.139	0.139	0.215	3.240	4.100	1.265	0.760	0.190	0.884	0.430	3.670			0.108	0.918	0.108	0.918
72	0.234	0.234	0.277	2.993	4.099	1.370	1.008	0.252	0.910	0.554	3.546			0.138	0.887	0.138	0.887
102	0.425	0.425	0.344	2.662	4.040	1.518	1.338	0.335	0.971	0.689	3.351			0.172	0.838	0.172	0.838
132	0.664	0.664	0.389	2.403	3.958	1.647	1.597	0.399	1.028	0.777	3.180			0.194	0.795	0.194	0.795
162	0.959	0.959	0.421	2.186	3.872	1.771	1.814	0.453	1.076	0.843	3.029			0.211	0.757	0.211	0.757
192	1.289	1.289	0.436	2.018	3.764	1.865	1.982	0.495	1.135	0.873	2.891			0.218	0.723	0.218	0.723
231	1.695	1.695	0.456	1.840	3.665	1.992	2.160	0.540	1.183	0.913	2.753			0.228	0.688	0.228	0.688
273	2.146	2.146	0.468	1.703	3.575	2.099	2.297	0.574	1.227	0.930	2.639	$\bar{\sigma}_r = \bar{\sigma}_3$	$\bar{\sigma}_a = \bar{\sigma}_1$	0.234	0.660	0.234	0.660
313	2.615	2.615	0.470	1.600	3.482	2.176	2.400	0.600	1.276	0.941	2.541			0.235	0.635	0.235	0.635
343	2.945	2.945	0.477	1.522	3.432	2.254	2.477	0.619	1.297	0.955	2.477			0.239	0.619	0.239	0.619
373	3.292	3.292	0.480	1.455	3.375	2.319	2.544	0.636	1.325	0.960	2.415			0.240	0.604	0.240	0.604
413	3.759	3.759	0.482	1.396	3.322	2.380	2.604	0.651	1.352	0.963	2.359			0.241	0.589	0.241	0.589
473	4.453	4.453	0.483	1.300	3.232	2.485	2.670	0.675	1.398	0.966	2.266			0.241	0.566	0.241	0.566
533	5.138	5.137	0.479	1.238	3.158	2.550	2.762	0.691	1.439	0.960	2.198			0.240	0.549	0.240	0.549
593	5.834	5.834	0.478	1.178	3.091	2.622	2.822	0.705	1.476	0.956	2.135			0.239	0.533	0.239	0.533
653	6.542	6.512	0.472	1.145	3.032	2.648	2.855	0.714	1.513	0.944	2.089			0.236	0.522	0.236	0.522
767	7.907	7.907	0.469	1.058	2.934	2.774	2.943	0.736	1.568	0.938	1.996			0.235	0.499	0.235	0.499
887	9.347	9.347	0.463	0.998	2.848	2.854	3.002	0.751	1.622	0.925	1.923			0.231	0.481	0.231	0.481
1007	10.766	10.766	0.456	0.957	2.781	2.906	3.043	0.761	1.668	0.912	1.869			0.228	0.467	0.228	0.467
1127	12.176	12.756	0.451	0.923	2.728	2.955	3.077	0.769	1.705	0.902	1.826			0.226	0.456	0.226	0.456
1247	13.539	13.539	0.444	0.899	2.675	2.974	3.100	0.775	1.746	0.888	1.788	↓	↓	0.222	0.447	0.222	0.447

(1) CORRECTED FOR _____

(2) Δu FOR $\Delta\sigma_3 = 0$

(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

REMARKS:

$\overline{CIUC}$; OCR = 2.0

CONSOLIDATED - UNDRAINED TRIAXIAL TEST - DATA SUMMARY SHEET

SOIL MECHANICS LABORATORY, DEPT OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

TEST NO. EIC-1-2
SOIL EAST WINDSOR VARVED CLAY
PROJECT 80142
TESTED BY SAM. DATE 15 June 1975
ALL STRESSES IN kg/cm².

	W, %	e	S, %	V, cc	L, cm	A, cm²
INITIAL	58.2			79.70	8.03	9.93
PRESHEAR	47.4			70.43	7.56	9.32

G_s = ____ TYPE CELL GEO.

PRESHEAR: $\bar{\sigma}_{1c}$ = 2.0, t_c = 1 day; $\bar{\sigma}_{3c}$ = 2.0, P.P.R. = ____ %; $\bar{\sigma}_{ac}$ = 2.0, u_b = ____

DURING SHEAR: CONTROLLED STRAIN ✓ STRESS ____; RATE 0.7%/hr.; PATH compression loading

PRESHEAR STRESS HISTORY ISOTROPIC CONSOLIDATED TO 4.0 kg/cm² and rebound to 2.0 kg/cm².

ELAPSED TIME	AXIAL STRAIN, %	$(\bar{\sigma}_1 - \bar{\sigma}_3)$ (1)	$\frac{(\bar{\sigma}_1 - \bar{\sigma}_3)}{\bar{\sigma}_{1c}}$	$\bar{\sigma}_3$	$\bar{\sigma}_1$	$\bar{\sigma}_1/\bar{\sigma}_3$	ΔJ (2)	$\frac{\Delta J}{\bar{\sigma}_{1c}}$ (2)	A (3)	q	$\bar{p}$	$\bar{\sigma}_r$	$\bar{\sigma}_a$	$\frac{q}{\bar{\sigma}_{vc}}$	$\frac{\bar{p}}{\bar{\sigma}_{vc}}$	$\frac{q}{\bar{\sigma}_{vm}}$	$\frac{\bar{p}}{\bar{\sigma}_{vm}}$
0	0	0	0	2.000	2.000	1.000	0	0	–	0	2.000	↑	↑	0	1.000	0	0.500
84	0.24	0.537	0.268	1.724	1.724	1.311	0.138	0.138	0.513	0.269	1.992			0.134	0.996	0.067	0.498
102	0.35	0.744	0.372	1.610	1.610	1.461	0.195	0.195	0.524	0.186	1.982			0.186	0.990	0.093	0.495
117	0.46	0.839	0.420	1.529	1.529	1.349	0.1235	0.1235	0.560	0.210	1.948			0.210	0.974	0.105	0.487
172	0.91	1.231	0.616	1.350	1.350	1.912	0.325	0.325	0.528	0.616	1.965			0.308	0.982	0.154	0.491
304	2.24	1.546	0.773	1.162	1.162	2.330	0.419	0.419	0.542	0.773	1.935			0.387	0.967	0.194	0.484
401	3.29	1.608	0.804	1.064	1.064	2.511	0.468	0.468	0.582	0.804	1.868			0.402	0.934	0.201	0.467
409	3.37	1.613	0.806	1.051	1.051	2.535	0.475	0.475	0.588	0.806	1.858			0.403	0.928	0.201	0.464
529	4.71	1.648	0.824	0.975	0.975	2.690	0.512	0.512	0.622	0.824	1.799	$\bar{\sigma}_r = \bar{\sigma}_3$	$\bar{\sigma}_a = \bar{\sigma}_1$	0.412	0.899	0.206	0.449
649	6.02	1.671	0.836	0.915	0.915	2.826	0.542	0.542	0.649	0.836	1.751			0.418	0.875	0.214	0.438
769	7.40	1.682	0.841	0.868	0.868	2.938	0.566	0.566	0.673	0.842	1.709			0.421	0.854	0.210	0.427
889	8.08	1.676	0.838	0.848	0.848	2.976	0.576	0.576	0.687	0.838	1.686			0.419	0.843	0.210	0.422
1009	9.48	1.668	0.834	0.816	0.816	3.044	0.592	0.592	0.709	0.834	1.650			0.417	0.825	0.208	0.413
1069	10.18	1.660	0.830	0.811	0.811	3.047	0.595	0.595	0.716	0.830	1.641			0.415	0.820	0.208	0.410
1189	11.56	1.652	0.826	0.793	0.793	3.083	0.604	0.604	0.730	0.826	1.619			0.413	0.809	0.206	0.405
1249	12.20	1.651	0.825	0.780	0.780	3.117	0.610	0.610	0.740	0.824	1.605			0.412	0.803	0.206	0.402
1369	13.66	1.606	0.803	0.767	0.767	3.093	0.616	0.616	0.768	0.802	1.570			0.401	0.785	0.200	0.393
1489	14.96	1.549	0.774	0.767	0.767	3.010	0.616	0.616	0.796	0.774	1.541	↓	↓	0.387	0.770	0.194	0.385

460

(1) CORRECTED FOR ____

(2) ΔJ FOR $\Delta\sigma_3 = 0$

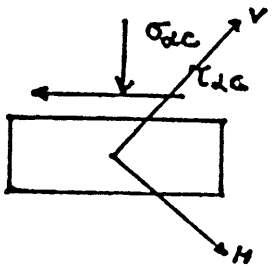
(3) $A = \frac{\Delta u - \Delta\sigma_r}{\Delta\sigma_a - \Delta\sigma_r}$ FOR COMPRESSION TESTS

$A = \frac{\Delta u - \Delta\sigma_a}{\Delta\sigma_r - \Delta\sigma_a}$ FOR EXTENSION TESTS

REMARKS:

SPECIAL CKoUDSS TESTS

CONSOLIDATION DATA.



Stress in kg/cm²

TEST NO ESDS-1

SOIL EAST WINDSOR VARVED CLAY

Project. 80142

TESTED BY SAM,SNL DATE.____

initial Height of sample 0.874

initial Area 50 cm².

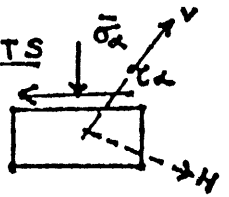
tc 2 days $\bar{\sigma}_{vm}$ = 4.010

σ_{1c}	τ_{1c}	Ko	σ_{1c}	$\bar{\sigma}_{3c}$	ε_1 %	γ %	REMARK.
0.104	0	-	0.104	-	0.11	-	
0.207	0	-	0.207	-	0.92	-	
0.508	0	-	0.508	-	3.24	-	
0.961	0.445	0.91	1.006	0.917	4.74	1.02	
1.340	0.1410	0.81	1.481	1.199	5.62	1.82	
1.842	0.2758	0.74	2.118	1.566	6.98	3.22	
2.097	0.3931	0.68	2.490	1.704	7.92	4.51	
2.509	0.4731	0.68	2.982	2.036	9.31	6.32	
3.016	0.569	0.68	3.585	2.447	10.98	8.67	
3.378	0.632	0.68	4.010	2.746	12.86	11.04	

V: Vertical direction in the ground : H: Horizontal direction in the ground.

stress in kg/cm²

SPECIAL $\bar{C}k_o\bar{U}DSS$ TESTS



TEST NO. ESDS-1 PAGE 1 OF 2

$\bar{\sigma}_{dc}$ = 3.378

τ_{dc} = 0.639

$\bar{\sigma}_{vc}$ = 4.01

ASSUMED K_o = 0.68

OCR = 1.0

t_c = 2 days ω_N = — ω_f = —

soil EAST WINDSOR V.C.

Project 80142

TESTED BY JNL DATE 5/6/75

γ%	$\Delta\tau_d$	τ_d	$\bar{\sigma}_d$	Δu	$\frac{\tau_d}{\bar{\sigma}_{vc}}$	$\frac{\tau_d}{\bar{\sigma}_d}$	$\frac{\Delta y}{\bar{\sigma}_{vc}}$	$\frac{\bar{\sigma}_d}{\bar{\sigma}_{vc}}$			REMARK
0	—	0.639	3.378	0	0.159	0.189	0	0.842			
0.013	0.017	0.656	3.364	0.014	0.164	0.195	0.003	0.839			
0.026	0.046	0.685	3.358	0.020	0.171	0.204	0.005	0.837			
0.052	0.074	0.713	3.332	0.046	0.178	0.214	0.011	0.831			
0.129	0.130	0.769	3.315	0.063	0.192	0.232	0.016	0.827			
0.233	0.170	0.809	3.262	0.116	0.202	0.248	0.029	0.813			
0.543	0.237	0.876	3.151	0.227	0.219	0.278	0.057	0.786			
1.19	0.286	0.925	2.984	0.394	0.231	0.310	0.098	0.744			
1.86	0.303	0.942	2.855	0.523	0.235	0.330	0.130	0.712			
3.10	0.310	0.949	2.651	0.727	0.237	0.358	0.181	0.661			
3.62	0.311	0.950	2.582	0.796	0.237	0.368	0.199	0.644			
4.29	0.309	0.948	2.515	0.863	0.236	0.377	0.215	0.627			
5.44	0.297	0.936	2.431	0.947	0.233	0.385	0.236	0.606			
7.02	0.283	0.922	2.299	1.079	0.229	0.401	0.269	0.573			
8.43	0.263	0.902	2.179	1.199	0.225	0.414	0.299	0.543			
9.58	0.248	0.887	2.102	1.276	0.221	0.422	0.318	0.524			

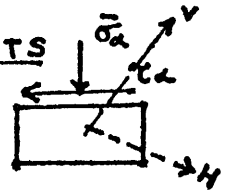
$\dot{\gamma}$ = 5 %/hr

V : Vertical direction in the ground
H : Horizontal direction in the ground.

Consolidation History

Stress in kg/cm²

SPECIAL $\bar{C}K_o\bar{U}DSS$ TESTS



TEST NO. ESDS-1 PAGE 2 OF 2

$\bar{\sigma}_{dc}$ = 3.378

τ_{dc} = 0.632

$\bar{\sigma}_{vc}$ = 4.01

ASSUMED K_o = 0.68

OCR = 1.0

t_c = 2 days ω_N = — ω_f = —

Soil EAST WINDSOR VC.

Project 80142

TESTED BY SNL DATE 5/6/75

γ%	$\Delta\tau_d$	τ_d	$\bar{\sigma}_d$	Δu	$\frac{\tau_d}{\bar{\sigma}_{vc}}$	$\frac{\gamma_{id}}{(\tau_d)}$	$\frac{\Delta u}{\bar{\sigma}_{vc}}$	$\frac{\bar{\sigma}_d}{\bar{\sigma}_{vc}}$			REMARK
10.62	0.239	0.869	2.045	1.333	0.217	0.425	0.099	0.510			
11.62	0.206	0.836	1.949	1.429	0.208	0.429	0.106	0.486			
13.01	0.119	0.749	1.746	1.632	0.187	0.429	0.120	0.435			
13.91	0.057	0.687	1.601	1.777	0.171	0.429	0.131	0.399			
15.09	0.035	0.635	1.501	1.877	0.158	0.423	0.139	0.374			

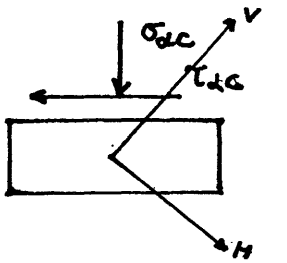
$\dot{\gamma}$ = 5 %/hr

V : Vertical direction in the ground
H : Horizontal direction in the ground.

Consolidation History

SPECIAL $\overline{CK_oUDSS}$ TESTS

CONSOLIDATION DATA.



Stress in kg/cm²

TEST NO ESDS-2

SOIL EAST WILDSIDE V.C

Project. 80142

TESTED BY JUL DATE.____

Initial Height of sample 0.985"

Initial Area 50 cm²

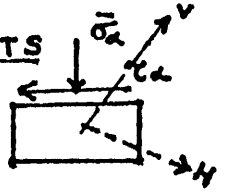
tc 2 days $\bar{\sigma}_{vm}$ = 4.027

σ_{dc}	τ_{dc}	Ko	$\bar{\sigma}_{ic}$	$\bar{\sigma}_{3c}$	ε_d %	γ %	REMARK.
0.104	0	-	0.104	-	0.60	-	
0.211	0	-	0.211	-	1.29	-	
0.520	0	-	0.520	-	1.64	-	
0.971	0.049	0.91	1.016	0.927	3.57	0.80	
1.342	0.146	0.81	1.483	1.201	4.52	1.60	
1.838	0.281	0.74	2.114	1.562	6.22	3.56	
2.097	0.399	0.68	2.490	1.704	7.37	5.08	
2.505	0.479	0.68	2.978	2.032	9.34	7.64	
3.020	0.575	0.68	3.588	2.452	11.62	10.66	
3.395	0.639	0.686	4.027	2.763	13.11	12.56	

V= Vertical direction in the ground : H= Horizontal direction in the ground.

Stress in kg/cm²

SPECIAL CKoUDSS TESTS



TEST NO. ESDS-2 PAGE 1 OF 2

$\bar{\sigma}_{dc}$ = 3.395

τ_{dc} = 0.1632

$\bar{\sigma}_{vc}$ = 4.027

ASSUMED Ko = 0.68

OCR = 1.0

t_c = 2 days ω_N = ____ ω_f = ____

Soil EAST WINDSOR V.C

Project 80142

TESTED BY JNK DATE 5/27/75

γ%	$\Delta\tau_d$	τ_d	$\bar{\sigma}_d$	Δu	$\frac{\tau_d}{\bar{\sigma}_{vc}}$	$\frac{\tau_d}{\bar{\sigma}_d}$	$\frac{\Delta u}{\bar{\sigma}_{vc}}$	$\frac{\bar{\sigma}_d}{\bar{\sigma}_{vc}}$			REMARK
0	0	0.1632	3.395	0	0.157	0.186	0	0.843			
0.097	-0.230	0.1402	3.428	-0.033	0.100	0.117	-0.008	0.851			
0.271	-0.1403	0.1229	3.415	-0.020	0.057	0.067	-0.005	0.848			
0.543	-0.575	0.1057	3.354	0.041	0.014	0.017	0.010	0.833			
0.672	-0.631	0.1001	3.321	0.074	0.0002	0.0003	0.018	0.825			
0.805	-0.663	-0.031	3.222	0.173	-0.008	-0.010	0.043	0.800			
1.049	-0.738	-0.106	3.189	0.206	-0.026	-0.033	0.051	0.792			
1.440	-0.842	-0.210	3.119	0.276	-0.052	-0.067	0.069	0.775			
2.038	-0.947	-0.315	2.999	0.396	-0.078	-0.105	0.098	0.745			
2.494	-1.010	-0.378	2.913	0.482	-0.094	-0.130	0.120	0.723			
3.000	-1.068	-0.436	2.835	0.560	-0.108	-0.154	0.139	0.704			
4.105	-1.153	-0.521	2.678	0.717	-0.129	-0.195	0.178	0.665			
5.025	-1.207	-0.575	2.563	0.832	-0.143	-0.224	0.207	0.636			
5.945	-1.246	-0.614	2.468	0.927	-0.152	-0.249	0.230	0.613			
7.326	-1.288	-0.656	2.386	1.009	-0.163	-0.275	0.251	0.593			
9.396	-1.339	-0.707	2.279	1.116	-0.176	-0.310	0.277	0.566			

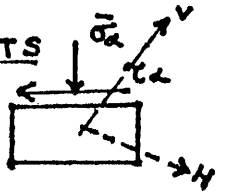
$\dot{\gamma}$ = 5 %/hr

V : Vertical direction in the ground
H : Horizontal direction in the ground.

Consolidation History ____

Stress in kg/cm²

SPECIAL $\bar{C}K_0$UDSS TESTS



TEST NO. E SBS-2 PAGE 2 OF 2

$\bar{\sigma}_{dc}$ = 3.395

τ_{dc} = 0.632

$\bar{\sigma}_{vc}$ = 1.027

ASSUMED K_0 = 0.68

OCR = 1.0

t_c = 2 days ω_N = ____ ω_f = ____

soil EAST WINDSOR VC

Project 80142

TESTED BY JVL DATE 5/27/75

γ%	$\Delta\tau_d$	τ_d	$\bar{\sigma}_d$	Δu	$\frac{\tau_d}{\bar{\sigma}_{vc}}$	$\frac{\tau_d}{\bar{\sigma}_{dc}}$	$\frac{\Delta u}{\bar{\sigma}_{vc}}$	$\frac{\bar{\sigma}_d}{\bar{\sigma}_{vc}}$			REMARK
12.387	-1.369	-0.737	2.151	1.244	-0.183	-0.343	0.309	0.534			
15.838	-1.403	-0.771	2.069	1.326	-0.191	-0.373	0.329	0.514			
19.888	-1.421	-0.789	2.003	1.392	-0.196	-0.394	0.346	0.497			
20.440	-1.421	-0.789	1.995	1.400	-0.196	-0.395	0.348	0.495			
21.406	-1.421	-0.789	1.982	1.413	-0.196	-0.398	0.351	0.492			
22.050	-1.423	-0.791	1.974	1.421	-0.196	-0.401	0.353	0.490			
23.753	-1.418	-0.786	1.929	1.466	-0.195	-0.407	0.364	0.479			
26.652	-1.397.	-0.765	1.871	1.524	-0.190	-0.409	0.378	0.465			

$\dot{\gamma}$ = ____ %/hr

V = Vertical direction in the ground
H = Horizontal direction in the ground.

Consolidation History ____