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ELASTO-PLASTIC CONSTITUTIVE RELATIONS AND COUPLED FORMULATIONS FOR UNSATURATED SOILS

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Elasto-plastic Constitutive Relations and Coupled Formulations for Unsaturated Soils

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*To my parents,
my wife
and
my son*

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ABSTRACT

The mechanical behaviour of unsaturated soils is affected by the changes in pore-water and pore-air pressures. There are two classes of pore-water pressures that may develop in the field. The first one is pore-water pressure associated with seepage through soils. The second one is excess pore-water pressure that develops from the application of an external load. When excess pore-water pressures develop, a transient process of water flow occurs. The transient process of water flow changes the equilibrium conditions of the unsaturated soil because it changes the stress state in the porous media. Volume change occurs since the soil structure reaches a new equilibrium condition. On the other hand, volume change alters the hydraulic properties of the soil structure and thus influences the transient process of water flow through the soil. Therefore, the interactive processes between stress-strain and pore-water pressure are primary processes affecting the mechanical behaviour of unsaturated soils.

The stress-strain process is governed by the equilibrium equations. The water flow is governed by the water continuity equation. An accurate solution for the mechanical behaviour of an unsaturated soil requires that the equilibrium equations and the continuity equation be solved simultaneously as a coupled solution. In the coupled approach, the simultaneous solution of the two equations (i.e., equilibrium equations and continuity equation) means that the dynamic interdependence between seepage and deformation problems is fully realized. In the uncoupled approach, the water phase continuity (i.e., seepage) is solved independently from the equilibrium (i.e., stress-strain) equations.

In this study, elasto-plastic constitutive relations for unsaturated soil were proposed. Coupled formulations were also formulated based on the proposed elasto-plastic constitutive relations. The experimental data available in literature and in this study were used to verify the proposed elasto-plastic constitutive relations. The triaxial apparatus was modified to incorporate NTU mini suction probes in order to measure pore-water pressures in the specimen during the test. The modified triaxial apparatus was used to conduct a series of soil-water characteristic curve (SWCC) tests, a series of isotropic consolidation tests, a series of consolidated drained tests

and a series of shearing-infiltration tests. The soil sample was prepared using Kaolin.

The results from SWCC tests on compacted soil specimens show that the air-entry value, AEV , and the yield suction, s_o , increase with the increase in net mean stress. These yield suctions do not belong to the same yield curve. The results from isotropic consolidation tests on compacted soil specimens show that yield stress, p_o , increase with the increase in matric suction, s . Similar to the yield suction, these yield stress do not belong to the same yield curve. The prediction of yield curve on (p - s) plane (i.e., suction increase yield curve and loading-collapse yield curve) based on the equations proposed in this study show a closer agreement with the experimental data as compared with the predictions based on Alonso et al. (1990) equations.

Results from the consolidated drained triaxial tests show that matric suction was smaller than the initially applied matric suction. This was in agreement with the variation of water volume change during the shearing stage (i.e., water volume change decreased throughout the shearing stage). The behaviour of compacted soil during shearing is similar to that of the normally consolidated soil. Results from shearing infiltration tests show that most stress paths of the shearing-infiltration tests reached the failure envelope and the failure points were also close to the failure envelope.

The prediction results (i.e., shear strength, total volume change and matric suction change characteristic) obtained from the elasto-plastic constitutive relations proposed in this study show a reasonably good agreement with the experimental results during shearing under consolidated drained and shearing-infiltration conditions.

TABLE OF CONTENTS

ACKNOWLEDGEMENTS.....	V
ABSTRACT.....	VII
TABLE OF CONTENTS.....	IX
LIST OF TABLES	XIII
LIST OF FIGURES	XV
LIST OF SYMBOLS	XXIX
CHAPTER 1 INTRODUCTION.....	1
1.1 BACKGROUND.....	1
1.2 OBJECTIVES AND SCOPE OF THE RESEARCH.....	2
1.3 METHODOLOGY	3
1.4 OUTLINE OF THE REPORT.....	4
CHAPTER 2 LITERATURE REVIEW.....	7
2.1 INTRODUCTION	7
2.2 STRESS STATE VARIABLES	7
2.3 SOIL-WATER CHARACTERISTIC CURVE.....	7
2.4 CONSOLIDATION TESTS AND CONTROLLING FACTOR.....	9
2.5 SHEARING-INFILTRATION TEST AND THE CONTROLLING FACTORS.....	10
2.6 FLOW OF WATER	13
2.7 VOLUME CHANGE OF UNSATURATED SOILS	15
2.7.1 <i>General</i>	15
2.7.2 <i>Constitutive Relations</i>	16
2.7.2.1 Soil Structure Constitutive Relations.....	17
2.7.2.2 Water Phase Constitutive Relations.....	21
2.8 SHEAR STRENGTH OF UNSATURATED SOILS	23
2.9 REVIEW THE ELASTO-PLASTIC MODEL	27
2.9.1 <i>Basic Concept of Critical State Model</i>	27
2.9.2 <i>Review of Elasto-plastic Model for Unsaturated Soil</i>	31
2.10 COUPLED EQUATIONS.....	36
2.11 SUMMARY	39
CHAPTER 3 ESTABLISHED CONSTITUTIVE RELATIONS AND PROPOSED COUPLED FORMULATIONS	41
3.1 INTRODUCTION	41

3.2	CONSTITUTIVE RELATIONS FOR UNSATURATED SOILS	41
3.2.1	<i>Soil Structure Constitutive Relations</i>	41
3.2.1.1	Elasticity Form of Constitutive Relations	41
3.2.1.2	Elasto-plastic Form of Constitutive Relations	42
3.2.1.3	Incremental Stress-Strain Relations	59
3.2.2	<i>Water Phase Constitutive Relation</i>	60
3.2.2.1	Elasticity Form of Water Phase Constitutive Relations	60
3.2.2.2	Elasto-plastic Form of Water Phase Constitutive Relations	61
3.3	COUPLING BETWEEN EQUILIBRIUM PROBLEM AND SEEPAGE PROBLEM	64
3.3.1	<i>Equilibrium Equations of Soil Structure</i>	64
3.3.2	<i>Water Phase Continuity</i>	67
3.4	SUMMARY	67
CHAPTER 4 EXPERIMENTAL PROGRAMME AND PREDICTION FROM THE PROPOSED MODEL		69
4.1	INTRODUCTION	69
4.2	OUTLINE OF RESEARCH PROGRAMME.....	69
4.3	BASIC PROPERTIES OF SOIL AND SPECIMEN PREPARATION.....	70
4.3.1	<i>Criteria for Soil Selection and Specimen Preparation</i>	70
4.3.2	<i>Basic Soil Properties and Compaction Method</i>	71
4.3.3	<i>Static Compaction Method</i>	71
4.3.4	<i>Static Compaction Process</i>	73
4.4	TRIAXIAL SET UP AND ITS DEVELOPMENT	74
4.4.1	<i>Triaxial Cell</i>	77
4.4.2	<i>Special Adapters at Triaxial Base</i>	77
4.4.3	<i>Pore-water Pressure Control System</i>	79
4.4.4	<i>Pore-air Pressure Control System</i>	79
4.4.5	<i>Flushing System</i>	79
4.4.6	<i>Measurement of Matric Suction in the Soil Specimen</i>	80
4.4.7	<i>Force Actuator</i>	83
4.4.8	<i>Thermometer</i>	83
4.4.9	<i>Linear Variable Differential Transformer (LVDT)</i>	84
4.4.10	<i>Data acquisition system</i>	84
4.5	TESTING PROCEDURE	84
4.5.1	<i>Testing Procedure for SWCC Tests</i>	84
4.5.1.1	Saturation Stage	84
4.5.1.2	Consolidation Stage	85

4.5.1.3	Drying and Wetting Stages	85
4.5.2	<i>Testing Procedure for Isotropic Consolidation Tests</i>	86
4.5.2.1	Saturation and Matric Suction Equalization Stages	86
4.5.2.2	Isotropic Consolidation Stage	86
4.5.3	<i>Testing Procedure for Consolidated Drained Tests</i>	87
4.5.3.1	Saturation, Consolidation and Matric Suction Equalization Stages.....	87
4.5.3.2	Shearing Stage	87
4.5.4	<i>Testing Procedure for the Shearing-infiltration Tests</i>	87
4.5.4.1	Saturation, Consolidation, Matric Suction Equalization and Shearing Stages	87
4.5.4.2	Infiltration Stage	87
4.6	TESTING PROGRAMME.....	88
4.6.1	<i>SWCC Tests</i>	88
4.6.2	<i>Isotropic Consolidation Tests</i>	90
4.6.3	<i>Consolidated Drained Tests</i>	90
4.6.4	<i>Shearing-infiltration Tests</i>	91
4.7	PREDICTION FROM THE PROPOSED MODEL.....	92
4.7.1	<i>Prediction Programme</i>	92
4.7.2	<i>Prediction Procedure</i>	95
CHAPTER 5 EXPERIMENTAL RESULTS		99
5.1	INTRODUCTION	99
5.2	INDEX PROPERTIES	99
5.2.1	<i>Index Properties</i>	99
5.2.2	<i>Soil-Water Characteristic Curves</i>	102
5.2.3	<i>Isotropic Consolidation Curves</i>	110
5.3	CONSOLIDATED DRAINED (CD) TRIAXIAL TEST RESULTS	115
5.3.1	<i>Shear Strength Behaviours</i>	116
5.3.2	<i>Total Volume Changes Characteristics</i>	119
5.3.3	<i>Water Volume Changes Characteristics</i>	122
5.4	EXTENDED MORH-COULOMB FAILURE ENVELOPE FROM CONSOLIDATED DRAINED TESTS	124
5.4.1	<i>Failure Criterion</i>	124
5.4.2	<i>Morh-Coulomb Envelope from Consolidated Drained Tests</i>	125
5.5	SHEARING-INFILTRATION TESTS RESULTS	128
5.5.1	<i>Shear Strength Behaviours</i>	128
5.5.2	<i>Total Volume Changes Characteristics</i>	131

5.5.3	<i>Matric Suction Changes Characteristics.....</i>	132
5.5.4	<i>Stress Path from Shearing-infiltration Tests.....</i>	134
CHAPTER 6 DISCUSSION OF THE EXPERIMENTAL RESULTS AND COMPARISONS WITH PREDICTION FROM PROPOSED FORMULATIONS.....		139
6.1	INTRODUCTION	139
6.2	SOIL-WATER CHARACTERISTIC CURVES	139
6.2.1	<i>Matric Suction Changes during Drying Stage</i>	<i>139</i>
6.2.2	<i>Variations of Air-entry Value (AEV) and Yield Suction</i>	<i>140</i>
6.3	ISOTROPIC CONSOLIDATION TESTS.....	140
6.4	COMBINATION OF YIELD STRESS AND PREDICTED YIELD CURVE ON THE P-S PLANE	141
6.5	RESULTS OF TRIAXIAL SHEARING TESTS	146
6.5.1	<i>Consolidated Drained Triaxial Tests.....</i>	<i>146</i>
6.5.2	<i>Shearing-infiltration Tests.....</i>	<i>149</i>
6.6	COMPARISON OF PROPOSED MODEL PREDICTIONS WITH THE CONSOLIDATED DRAINED TRIAXIAL TEST RESULTS	150
6.6.1	<i>Comparison of Proposed Model Predictions with the Results from CD Triaxial Tests Conducted in this Study.....</i>	<i>150</i>
6.6.2	<i>Comparison of Proposed Model Predictions with the CD Triaxial Tests from Literature</i>	<i>156</i>
6.7	COMPARISON OF PROPOSED MODEL WITH THE SHEARING-INFILTRATION TESTS	161
6.7.1	<i>Comparison of Proposed Model with the Results from Shearing-infiltration Tests Conducted in this Study</i>	<i>161</i>
6.7.2	<i>Comparison of Proposed Model Predictions with the Results of Shearing-infiltration Tests from Literature</i>	<i>169</i>
CHAPTER 7 SUMMARY, CONSLUSION AND RECOMENDATIONS		175
7.1	SUMMARY	175
7.2	CONCLUSION.....	177
7.3	RECOMMENDATIONS	177
REFERENCES		179
APPENDIX A CALIBRATION DATA OF MODIFIED TRIAXIAL APPARATUS		193
APPENDIX B PREDICTION OF THE CONSOLIDATED DRAINED TRIAXIAL TESTS AND SHEARING-INFILTRATION TESTS USING PROPOSED MODEL PRESENTED IN CHAPTER 3		201

LIST OF TABLES

Table 4-1	Net mean stresses and matric suctions that were used in SWCC-10 test.....	89
Table 4-2	Programme for the CD triaxial tests.....	91
Table 4-3	Programme for the shearing-infiltration tests.....	92
Table 4-4	Index properties of Kaolin used by Meilani (2004), Meilani et al. (2005), Thu (2005) and Thu et al. (2007) (after Meilani (2004), Meilani et al. (2005), Thu (2005) and Thu et al. (2007))	93
Table 4-5	Summary of parameters obtained from the isotropic consolidation tests on compacted kaolin conducted by Thu (2005) and Thu et al. (2007) (after Thu (2005) and Thu et al. (2007)).....	93
Table 4-6	Consolidated drained triaxial tests conducted by Thu (2005).....	94
Table 4-7	Shearing-infiltration tests conducted by Meilani (2004).....	94
Table 5.1	Index properties of Kaolin.....	101
Table 5.2	Summary of soil parameter obtained from SWCC tests	107
Table 5.3	Summary of parameters obtained from the isotropic consolidation tests on compacted kaolin	114
Table 5.4	Void ratio, e , water content, w , and degree of saturation, S , of the CD triaxial tests on compacted kaolin.....	119
Table 5.5	Summary of deviator stresses and axial strains at failure for the CD triaxial tests under matric suction of 0 kPa, 100 kPa or 200 kPa and net confining stress of 100 kPa or 300 kPa	125
Table 5.6	Void ratio, e , water content, w , and degree of saturation, S , of the shearing-infiltration triaxial tests on compacted kaolin	130

LIST OF FIGURES

Figure 2.1	A typical soil-water characteristic curve	8
Figure 2.2	Deviator stress, matric suction, volumetric and water volumetric strain, and axial strain during shearing-infiltration test on soil specimen initially consolidated under 25 kPa net confining pressure and 100 kPa matric suction (after Meilani et al., 2005).....	12
Figure 2.3	Hydraulic head at point A (after Fredlund and Rahardjo, 1993)	13
Figure 2.4	Soil structure constitutive surface for an unsaturated soil (after Fredlund and Rahardjo, 1993).....	20
Figure 2.5	Void ratio constitutive surface for an unsaturated soil (after Fredlund and Rahardjo, 1993).....	21
Figure 2.6	Cross section at a constant void ratio (after Fredlund and Rahardjo, 1993).....	21
Figure 2.7	Water phase constitutive surface for an unsaturated soil (after Fredlund and Rahardjo, 1993).....	22
Figure 2.8	Water content constitutive surface for an unsaturated soil (after Fredlund and Rahardjo, 1993).....	23
Figure 2.9	Mohr-Coulomb failure envelope for saturated soils (after Fredlund and Rahardjo, 1993).....	25
Figure 2.10	Extended Mohr-Coulomb failure envelope for unsaturated soils (after Fredlund and Rahardjo, 1993)	25
Figure 2.11	Failure envelope for unsaturated soil glacial till specimens. (a) Failure envelope on the, τ , against $(u_a - u_w)$ plane; (b) ϕ^b values versus matric suction (after Gan, 1986; Fredlund et al. 1987).	26
Figure 2.12	Non-linearity relationship between ϕ^b and matric suction at failure of constant water content triaxial tests (after Thu et al., 2006).....	27

Figure 2.13	Stress increment expanding of yield surface (after Wood, 1990)	29
Figure 2.14	Critical state line and these parameter. (a) Yield surface; (b) CSL on $(v - p')$ space; (c) CSL on $(v - \ln p')$ space (after Schofield and Wroth, 1968; Wood, 1990; and Budhu, 2000).....	30
Figure 2.15	Relationship between yield stress p_o and p_o^* : (a) compression curves for saturated and unsaturated soil; (b) stress path and yield curve in (p, s) stress plane (after Alonso et al., 1990).....	33
Figure 2.16	Loading collapse (LC) and suction increase (SI) yield curves (after Alonso et al., 1990).....	34
Figure 3.1	Idealized variation of specific volume with net mean stress and suction along virgin and loading-unloading paths	46
Figure 3.2	Idealized relationships between specific volume and matric suction at different net mean stresses of p	46
Figure 3.3	Derivation of LC yield curve equation.....	50
Figure 3.4	Derivation of SI yield curve equation	52
Figure 3.5	Variation of $N(s)$ with respect to matric suction (from Thu et al. 2007).....	53
Figure 3.6	Variation of $\lambda(s)$ with respect to matric suction (from Alonso et al. 1990, Rampino et al. 2000 and Thu et al. 2007)	54
Figure 3.7	Variation of $\lambda(s)$ with respect to matric suction (from Alonso et al. 1990, Rampino et al. 2000 and Thu et al. 2007)	54
Figure 3.8	LC and SI yield curves	55
Figure 3.9	The LC, SI and triaxial stress state yield curve	57
Figure 3.10	Yield curve in the $s/p = \text{constant}$ plane.....	58
Figure 3.11	Relationship between specific water volume and net mean stress at a constant matric suction (modified after Wheeler 1996, Thu et al. 2007)	61

Figure 3.12	Relationship between degree of saturation and suction, (a) typical SWCC; (b) idealized SWCC (solid lines represent current SWCC) (Modified after Sheng et al. 2008)	62
Figure 3.13	Normal stress and shear stress acting on a cubical soil element of infinitesimal dimensions (after Fredlund and Rahardjo, 1993).....	65
Figure 3.14	Water flow through a cubical soil element of infinitesimal dimension	67
Figure 4.1	Static compaction mould and stainless steel plugs, (a) Stainless steel mould; (b) Stainless steel plugs.	72
Figure 4.2	Stainless steel plug, (a) Connection between two adjacent disks; (b) removable disk; (c) small plug (after Ong 1999)	72
Figure 4.3	Compaction machine for static compaction of specimens	73
Figure 4.4	Extrusion of a compacted Kaolin specimen.....	74
Figure 4.5	Modified triaxial apparatus for unsaturated soil testing (Modified from Fredlund and Rahardjo, 1993)	75
Figure 4.6	The schematic diagram for the plumbing of the modified triaxial apparatus.	76
Figure 4.7	Water compartment in the pedestal with the high air entry disk (not installed).....	78
Figure 4.8	A typical wire passing through the extension ring	78
Figure 4.9	NTU mini probe	80
Figure 4.10	Installation details of NTU mini suction probes (after Meilani et al. 2002).....	81
Figure 4.11	Details of NTU mini suction probe on specimen (after Meilani et al. 2002).....	82
Figure 4.12	Two part of membrane stretcher with rubber holders	83

Figure 4.13	Stress paths for the SWCC under different net confining stresses.....	88
Figure 4.14	Stress paths for the isotropic consolidation tests under different matric suction.	89
Figure 4.15	Stress paths for CD triaxial tests under difference net confining stress and matric suction.	90
Figure 4.16	Stress paths for the shearing-infiltration tests under different net confining stresses, matric suctions and deviator stresses.....	91
Figure 4.17	Sketch of loading-collapse yield curve and suction increase yield curve at the beginning of the shearing stage and at the end of shearing stage together with the projection of yield path on the (p-s) plane during the shearing stage.....	97
Figure 4.18	Sketch of loading-collapse yield curve and suction increase yield curve at the beginning of the shearing stage, at the end of the shearing stage and at the end of the infiltration stage together with the projection of yield paths on the (p-s) plane during the shearing stage and infiltration stage.....	98
Figure 5.1	Classification of Kaolin in accordance with the Unified Soil Classification System plasticity chart (ASTM D 2487-00)	100
Figure 5.2	Grain size distribution curve of Kaolin	100
Figure 5.3	Compaction curve of Kaolin obtained from standard Proctor tests.....	101
Figure 5.4	Total volume change and water volume change versus elapsed time of specimen SWCC-250 for the case of matric suction increase from 100 kPa to 190 kPa.	102
Figure 5.5	Total volume change and water volume change versus elapsed time of specimen SWCC-100 for the case of matric suction decrease from 190 kPa to 100 kPa.	103

Figure 5.6	Pore-water pressure versus elapsed time during drying stage of specimen SWCC-250.	104
Figure 5.7	Variations of volumetric water content, specific volume and specific water volume with respect to matric suction of specimen SWCC-10	105
Figure 5.8	Variations of volumetric water content, specific volume and specific water volume with respect to matric suction of specimen SWCC-50	105
Figure 5.9	Variations of volumetric water content, specific volume and specific water volume with respect to matric suction of specimen SWCC-100	106
Figure 5.10	Variations of volumetric water content, specific volume and specific water volume with respect to matric suction of specimen SWCC-250	106
Figure 5.11	Soil water characteristic curves at different net mean stresses	107
Figure 5.12	Specific volume, v , versus matric suction at different net mean stresses.....	108
Figure 5.13	Specific water volume, v_w , versus matric suction at different net mean stresses	108
Figure 5.14	Air-entry value, AEV and yield suction, $s_o(p)$ at different net mean stresses as obtained from isotropic suction consolidation curves	109
Figure 5.15	Measured values of $N_s(p)$ at various net mean stresses as obtained from SWCCs	109
Figure 5.16	Slope of virgin curve and unloading/reloading curve at various net mean stresses as obtained from isotropic suction consolidation curves.....	110
Figure 5.17	Specific volume, v , versus net mean stresses at different matric suctions.....	111

Figure 5.18	Specific water volume, v_w , versus net mean stresses at different matric suctions.....	111
Figure 5.19	Yield stress, $p_o(s)$ at different matric suctions as obtained from isotropic consolidation curves	112
Figure 5.20	Measured values of $N(s)$ at various matric suctions as obtained from isotropic consolidation curves	112
Figure 5.21	Slope of normal consolidation curve and unloading/reloading curve at various matric suctions as obtained from isotropic consolidation curves	113
Figure 5.22	Measured values of $\kappa_w(s)$ and $\lambda_w(s)$ at various matric suctions as obtained from isotropic consolidation tests	113
Figure 5.23	Matric suction, $s=u_a-u_w$, versus net mean stress during isotropic consolidation tests, (a) for the case of initial matric suction of 30 kPa, (b) for the case of initial matric suction of 300 kPa	115
Figure 5.24	Variation of deviator stresses with respect to axial strain from CD triaxial tests under the same matric suction of zero kPa and net confining stress of 100 kPa or 300 kPa	116
Figure 5.25	Variation of deviator stresses with respect to axial strain from CD triaxial tests under the same matric suction of 100 kPa and net confining stress of 100 kPa or 300 kPa	117
Figure 5.26	Variation of deviator stresses with respect to axial strain from CD triaxial tests under the same matric suction of 200 kPa and net confining stress of 100 kPa or 300 kPa	117
Figure 5.27	Variation of deviator stresses with respect to axial strain from CD triaxial tests under the same net confining stresses of 100 kPa and matric suction of 0 kPa, 100 kPa or 200 kPa.....	118
Figure 5.28	Variation of deviator stresses with respect to axial strain from CD triaxial tests under the same net confining stresses of 300 kPa and matric suction of 0 kPa, 100 kPa or 200 kPa.....	118

Figure 5.29	Variation of volumetric strain with respect to axial strain from CD triaxial tests under the same matric suction of zero kPa and net confining stress of 100 kPa or 300 kPa	120
Figure 5.30	Variation of volumetric strain with respect to axial strain from CD triaxial tests under the same matric suction of 100 kPa and net confining stress of 100 kPa or 300 kPa	120
Figure 5.31	Variation of volumetric strain with respect to axial strain from CD triaxial tests under the same matric suction of 200 kPa and net confining stress of 100 kPa or 300 kPa	121
Figure 5.32	Variation of volumetric strain with respect to axial strain from CD triaxial tests under the same net confining stresses of 100 kPa and matric suction of 0 kPa, 100 kPa or 200 kPa.....	121
Figure 5.33	Variation of volumetric strain with respect to axial strain from CD triaxial tests under the same net confining stresses of 300 kPa and matric suction of 0 kPa, 100 kPa or 200 kPa.....	122
Figure 5.34	Variation of water volumetric strain with respect to axial strain from CD triaxial tests under the same matric suction of zero kPa and net confining stress of 100 kPa or 300 kPa	123
Figure 5.35	Variation of water volumetric strain with respect to axial strain from CD triaxial tests under the same matric suction of 100 kPa and net confining stress of 100 kPa or 300 kPa	123
Figure 5.36	Variation of water volumetric strain with respect to axial strain from CD triaxial tests under the same matric suction of 200 kPa and net confining stress of 100 kPa or 300 kPa	124
Figure 5.37	Peak deviator stresses of the consolidation drained triaxial tests under matric suction of 0 kPa, 100 kPa or 200 kPa and net confining stress of 100 kPa or 300 kPa.	125
Figure 5.38	Extended Mohr-Coulomb failure envelope for compacted kaolin obtained from CD triaxial tests under matric suction of 0	

	kPa, 100 kPa or 200 kPa and at net confining stress of 100 kPa or 300 kPa.....	126
Figure 5.39	Extended Mohr-Coulomb failure envelope for compacted kaolin obtained from CD triaxial tests under zero matric suction	126
Figure 5.40	Extended Mohr-Coulomb failure envelope for compacted kaolin obtained from CD triaxial tests under matric suction of 100 kPa.....	127
Figure 5.41	Extended Mohr-Coulomb failure envelope for compacted kaolin obtained from CD triaxial tests under matric suction of 200 kPa.....	127
Figure 5.42	Extended Mohr-Coulomb failure envelope for compacted kaolin as obtained from CD triaxial tests at the net normal stress of zero kPa plane	128
Figure 5.43	Variation of deviator stresses with respect to axial strain from SI 100-300 triaxial test	129
Figure 5.44	Variation of deviator stresses with respect to axial strain from SI 200-200 triaxial test	129
Figure 5.45	Variation of deviator stresses with respect to axial strain from SI 300-300 triaxial test	130
Figure 5.46	Variation of volumetric strain with respect to axial strain from SI 100-300 triaxial test	131
Figure 5.47	Variation of volumetric strain with respect to axial strain from SI 200-200 triaxial test	131
Figure 5.48	Variation of volumetric strain with respect to axial strain from SI 300-300 triaxial test	132
Figure 5.49	Variation of matric suction with respect to axial strain from SI 100-300 triaxial test.....	132

Figure 5.50	Variation of matric suction with respect to axial strain from SI 200-200 triaxial test.....	133
Figure 5.51	Variation of matric suction with respect to axial strain from SI 300-300 triaxial test.....	133
Figure 5.52	Failure envelope and stress path of shearing-infiltration stage of specimen SI 100-300.....	134
Figure 5.53	Failure envelope and stress path of shearing-infiltration stage of specimen SI 200-200.....	135
Figure 5.54	Failure envelope and stress path of shearing-infiltration stage of specimen SI 300-300.....	135
Figure 5.55	Failure envelope and stress path of shearing-infiltration stage of specimen SI 100-300 on the q - s plane	136
Figure 5.56	Failure envelope and stress path of shearing-infiltration stage of specimen SI 200-200 on the q - s plane	136
Figure 5.57	Failure envelope and stress path of shearing-infiltration stage of specimen SI 300-300 on the q - s plane	137
Figure 6.1	Yield suctions obtained from SWCC tests and predicted yield curves based on the equation proposed in this study	142
Figure 6.2	Yield suctions obtained from SWCC tests and predicted yield curves based on the equation proposed by Alonso et al. (1990)....	143
Figure 6.3	Yield suctions obtained from isotropic consolidation tests and predicted yield curves based on the equation proposed in this study	144
Figure 6.4	Yield suctions obtained from isotropic consolidation tests and predicted yield curves based on the equation proposed by Alonso et al. (1990).....	144
Figure 6.5	Yield stresses obtained from isotropic consolidation tests, yield suctions obtained from isotropic consolidation tests and	

	predicted yield curves based on the equations proposed in this study	145
Figure 6.6	Yield stresses obtained from isotropic consolidation tests, yield suctions obtained from isotropic consolidation tests and simulated yield curves based on the equations proposed by Alonso et al. (1990).....	145
Figure 6.7	Variation of matric suction with respect to axial strain during shearing stage of specimens CD 100-100 and CD 300-100.....	146
Figure 6.8	Variation of matric suction with respect to axial strain during shearing stage of specimens CD 100-200 and CD 300-200.....	147
Figure 6.9	Variation of matric suction with respect to axial strain during shearing stage of specimens SI 100-300 and SI 300-300.....	149
Figure 6.10	Variation of deviator stresses with respect to axial strain for CD300-0 test	152
Figure 6.11	Variation of volumetric strain with respect to axial strain for CD300-0 test	152
Figure 6.12	Variation of deviator stresses with respect to axial strain for CD300-100 test	153
Figure 6.13	Variation of volumetric strain with respect to axial strain for CD300-100 test	153
Figure 6.14	Variation of deviator stresses with respect to axial strain for CD300-200 test	154
Figure 6.15	Variation of volumetric strain with respect to axial strain for CD300-200 test	154
Figure 6.16	Yield curve at the beginning of the shearing stage and at the end of shearing stage together with the projection of yield path on the (p - s) plane during the shearing stage for CD 300-100 test	155

Figure 6.17	Yield curve and failure envelope at the beginning of the shearing stage on the $s/p = 1/3$ plane for CD 300-100 test.....	155
Figure 6.18	Yield curve and failure envelope at the end of the shearing stage on the $s/p = 0.19$ plane for CD 300-100 test	156
Figure 6.19	Variation of deviator stresses with respect to axial strain for CD300-0 (Thu 2005) test	157
Figure 6.20	Variation of volumetric strain with respect to axial strain for CD300-0 (Thu 2005) test	158
Figure 6.21	Variation of deviator stresses with respect to axial strain for CD300-100 (Thu 2005) test	158
Figure 6.22	Variation of volumetric strain with respect to axial strain for CD300-100 (Thu 2005) test	159
Figure 6.23	Variation of deviator stresses with respect to axial strain for CD300-200 (Thu 2005) test	159
Figure 6.24	Variation of volumetric strain with respect to axial strain for CD300-200 (Thu 2005) test	160
Figure 6.25	Variation of deviator stresses with respect to axial strain for CD300-300 (Thu 2005) test	160
Figure 6.26	Variation of volumetric strain with respect to axial strain for CD300-300 (Thu 2005) test	161
Figure 6.27	Variation of deviator stresses with respect to axial strain for SI200-200 test	163
Figure 6.28	Variation of matric suction with respect to axial strain for SI200-200 test	163
Figure 6.29	Variation of volumetric strain with respect to axial strain for SI200-200 test	164
Figure 6.30	Variation of deviator stresses with respect to axial strain for SI100-300 test	164

Figure 6.31	Variation of matric suction with respect to axial strain for SI100-300 test	165
Figure 6.32	Variation of volumetric strain with respect to axial strain for SI100-300 test	165
Figure 6.33	Variation of deviator stresses with respect to axial strain for SI300-300 test	166
Figure 6.34	Variation of matric suction with respect to axial strain for SI300-300 test	166
Figure 6.35	Variation of volumetric strain with respect to axial strain for SI300-300 test	167
Figure 6.36	Yield curve at the beginning of the shearing stage, at the end of the shearing stage and at the end of infiltration stage together with the projection of yield paths on the (p - s) plane during the shearing stage and infiltration stage for SI 200-200 test.....	167
Figure 6.37	Yield curve and failure envelope at the beginning of the shearing stage on the $s/p = 1$ plane for SI 200-200 test.....	168
Figure 6.38	Yield curve and failure envelope at the end of the shearing stage on the $s/p = 0.55$ plane for SI 200-200 test.....	168
Figure 6.39	Yield curve and failure envelope at the end of the infiltration stage on the $s/p = 0.07$ plane for SI 200-200 test.....	169
Figure 6.40	Variation of deviator stresses with respect to axial strain for SI25-100 (Meilani 2004) test	170
Figure 6.41	Variation of matric suction with respect to axial strain for SI25-100 (Meilani 2004) test	171
Figure 6.42	Variation of volumetric strain with respect to axial strain for SI25-100 (Meilani 2004) test	171
Figure 6.43	Variation of deviator stresses with respect to axial strain for SI100-100 (Meilani 2004) test	172

Figure 6.44	Variation of matric suction with respect to axial strain for SI100-100 (Meilani 2004) test	172
Figure 6.45	Variation of volumetric strain with respect to axial strain for SI100-100 (Meilani 2004) test	173
Figure 6.46	Variation of deviator stresses with respect to axial strain for SI200-100 (Meilani 2004) test	173
Figure 6.47	Variation of matric suction with respect to axial strain for SI200-100 (Meilani 2004) test	174
Figure 6.48	Variation of volumetric strain with respect to axial strain for SI100-100 (Meilani 2004) test	174

LIST OF SYMBOLS

AEV	= air-entry value
c'	= effective cohesion of saturated soil, intercept of the “extended” Mohr-Coulomb failure envelope on the shear stress axis when net normal stress are equal to zero
c_v	= coefficient of consolidation
dp	= increment of net mean stress
ds	= increment of matric suction
dV_v	= total volume change of a soil element
$d\varepsilon_v$	= volumetric strain change for each increment (dV_v/V_o)
$d\varepsilon_v^e$	= elastic volumetric strain increment
$d\varepsilon_v^p$	= plastic volumetric strain increment
$d\varepsilon_q$	= shear strain increment
$d\varepsilon_q^e$	= elastic shear strain increment
$d\varepsilon_q^p$	= plastic shear strain increment
$d\varepsilon$	= total strain increment
$d\varepsilon^e$	= elastic strain increment
$d\varepsilon^p$	= plastic strain increment
$d\varepsilon_{v(p)}^e$	= elastic volumetric strain increment induced by changes in net mean stress
$d\varepsilon_{v(s)}^e$	= elastic volumetric strain increment induced by changes in matric suction
$d\varepsilon_{v(p)}^p$	= plastic volumetric strain increment induced by changes in net mean stress

List of Symbols

$d\varepsilon_{v(s)}^p$	= plastic volumetric strain increment induced by changes in matric suction
dv_w^p	= plastic increment of specific water volume
E	= modulus of elasticity for the soil structure with respect to a change in the net normal stress, $(\sigma-u_a)$
E_w	= water volumetric parameter associated with a change in the net normal stress (i.e., $(\sigma-u_a)$)
e	= void ratio
G	= shear modulus (i.e., $G = \frac{E}{2(1+\mu)}$)
g	= gravitational acceleration
H	= modulus of elasticity for the soil structure with respect to a change in the matric suction, (u_a-u_w)
H_w	= water volumetric parameter associated with a change in the matric suction stress (i.e., (u_a-u_w))
h_w	= hydraulic head
K	= bulk modulus
k	= coefficient of permeability with respect to water at saturation
k_w	= coefficient of permeability with respect to water phase
M	= slope of the critical state line
M_*	= ratio between vertical diameter and horizontal diameter of the elliptical yield curve
M_a	= slope of critical state with respect to net mean stress on $(p-q)$ plane
M_w	= slope of critical state with respect to matric suction on $(q-(u_a-u_w))$ plane

m_1^s	= coefficient of volume change with respect to a change in net normal stress (i.e., $m_1^s = \frac{3(1-2\mu)}{E}$)
m_2^s	= coefficient of volume change with respect to a change in matric suction (i.e., $m_2^s = \frac{3}{H}$)
m_1^w	= coefficient of water volume change with respect to net normal stress
m_2^w	= coefficient of water volume change with respect to matric suction
m_v	= coefficient of volume change for saturated soils
$N(0)$	= the specific volume of the normal compression line at reference stress
$N(s)$	= specific volume of the normal consolidation line under a given matric suction at atmospheric pressure
$N_s(p)$	= specific volume under a given net mean stress at matric suction equal atmospheric pressure
n	= porosity of the soil
p'	= mean effective stress of a saturated soil
p	= net mean stress
$(p - u_a)$	= net mean stress
$(p - u_a)_{ref}$	= net mean stress at reference stress
p_{at}	= atmospheric pressure
p_o	= yield stress of saturated soil
p_f'	= mean effective stress at failure
p_o'	= preconsolidation stress of saturated soil
q	= deviator stress
s	= matric suction of the soil (i.e., $u_a - u_w$)

List of Symbols

s_0	= matric suction at yield point
S	= degree of saturation
u_a	= pore-air pressure
u_w	= pore-water pressure
$(u_a - u_w)_f$	= matric suction at failure
V_a	= volume of air in the infinitesimal unsaturated soil
V_o	= initial volume of an infinitesimal unsaturated soil
V_v	= volume of the soil voids
V_w	= volume of water in the infinitesimal unsaturated soil
v_w	= flow rate of water
v	= specific volume
v_f	= specific volume at failure
v_o	= the specific volume on the normal consolidation line at the preconsolidation stress p'_o
v_w	= specific water volume
y	= elevation of point A above the datum
∇^2	$= \frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial y^2} + \frac{\partial^2}{\partial z^2}$
$\Gamma(0)$	= specific volume on the critical state line for saturated soil at reference stress
Γ_{aw}	= the specific volume when both $(p - u_a)$ and $(u_a - u_w)$ equal unity
$\Gamma(s)$	= specific water volume at reference stress
$\Gamma_s(s)$	= degree of saturation at the reference stress of the critical state line
α	= soil parameter
χ	= a numerical coefficient ranging from 0 to 1

ε_v	= volumetric strain
ε_x	= normal strain component in the x -direction
ε_y	= normal strain component in the y -direction
ε_z	= normal strain component in the z -direction
μ	= Poisson's ratio
γ_{xy}	= shear strain on x -plane (i.e., $\gamma_{xy}=\gamma_{yx}$)
γ_{yz}	= shear strain on y -plane (i.e., $\gamma_{yz}=\gamma_{zy}$)
γ_{zx}	= shear strain on z -plane (i.e., $\gamma_{zx}=\gamma_{xz}$)
ρ_w	= density of water
σ	= total stress
σ_x	= total stress in the x -direction
σ_y	= total stress in the y -direction
σ_z	= total stress in the z -direction
σ_1	= major principal stress
σ_2	= intermediate principal stress
σ_3	= minor principal stress
σ_m	= average total normal stress (i.e., $(\sigma_x + \sigma_y + \sigma_z)/3$)
$(\sigma_f - u_a)_f$	= net normal stress on the failure plane at failure
σ_f	= total normal stress on the failure plane at failure
τ_{xy}	= shear stress on x -plane (i.e., $\tau_{xy}=\tau_{yx}$)
τ_{yz}	= shear stress on y -plane (i.e., $\tau_{yz}=\tau_{zy}$)
τ_{zx}	= shear stress on z -plane (i.e., $\tau_{zx}=\tau_{xz}$)
τ_{ff}	= shear stress on the failure plane at failure

List of Symbols

ϕ'	= the effective of friction angle of saturated soil, angle of internal friction associated with the net normal stress state variable
ϕ^b	= angle indicate the rate of change in shear strength relative to change in matric suction, $(u_a - u_w)_f$
κ	= elastic stiffness parameter for changes in net mean stress
κ_s	= elastic stiffness parameter for changes in matric suction
$\kappa(0)$	= elastic stiffness parameter from isotropic consolidation test under zero matric suction
$\kappa_w(s)$	= elastic stiffness parameter with respect to water phase from isotropic consolidation test under a given matric suction
$\lambda(s)$	= stiffness parameter for changes in net mean stress for a virgin state of soil under a given matric suction
λ_s	= stiffness parameter for changes in matric suction for virgin state of the soil, plastic stiffness parameter from SWCC under a given net confining stress
$\lambda(0)$	= the stiffness of the saturated soil, which obtained from an isotropic or one dimensional consolidation test
λ_a	= slope of critical state with respect to net mean stress on $(v-p)$ plane
λ_w	= slope of critical state with respect to matric suction on $(v-(u_a - u_w))$ plane
$\lambda_w(s)$	= plastic stiffness parameter with respect to water phase from isotropic consolidation test under a given matric suction
λ_{swcc}	= slope of the SWCC
$\psi_s(s)$	= slope of the critical state line for the degree of saturation

CHAPTER 1 INTRODUCTION

1.1 Background

A large number of geotechnical problems involve unsaturated soil zones where the voids between soil particles are filled with air and water. There are many practical situations associated with unsaturated soils that are challenging to geotechnical engineers in the field. For the case when fill materials are compacted or loaded, the excess pore-air pressure during compaction or loading will dissipate immediately, meanwhile the excess pore-water pressure will dissipate with time. During and after rainstorms the increase in pore-water pressure due to infiltration may result in slope instability.

There is increasing research development in unsaturated soils. Therefore, geotechnical engineers are now able to predict unsaturated soil behavior in the field and in the laboratory with certain accuracies. However, some problems associated with unsaturated soils are still very difficult to solve such as when excess pore-water pressures develop during the transient process of water flow. The transient process of water flow changes the equilibrium conditions of the unsaturated soils because it changes stress state in the porous media and volume change occurs since the soil structure reaches a new equilibrium condition. On the other hand, volume change alters the hydraulic properties of the soil structure and thus influences the transient process of water flow through the soil.

In the last few decades, a number of theoretical frameworks and constitutive relations based on elasto-plastic theory have been proposed to describe the mechanical behaviour of unsaturated soils. These models are capable to reproduce important behaviour of unsaturated soils. Alonso et al. (1987, 1990) proposed the general frame work constitutive relation for unsaturated soils. This model was futher refined by Toll (1990), Thomas and He (1994), Wheeler and Sivakumar (1995), Cui and Delage (1996), Wheeler (1996), Bolzon et al. (1996), Rampino et al. (1999), Simoni and Schrefler (2001), Tang and Graham (2002), Chiu and Ng, 2003, and Thu et al. (2007). These models were developed based on the independent stress state variables. On the other hand, there were different assumptions and constitutive relationships used in these models. Tang and Graham

(2002) pointed out that behaviour of compacted, highly plastic soils may not be able to be described using these models. Therefore, it would be appropriate to develop elasto-plastic constitutive relations for unsaturated soils that are applicable to wider range of soil types.

A general approach to study the coupled model for unsaturated soil has been proposed by Dakshanamurthy et al. (1984) and Fredlund and Rahardjo (1993). A number of researchers have used this approach in their study (Conte, 2004; Conte 2006; Vu and Fredlund, 2006). However, this approach uses an assumption that the relationships between stress and strain are reversible. In reality, the relationships between stress and strain are irreversible. Therefore, it would be appropriate to formulate the coupled equations based on the elasto-plastic model for unsaturated soils.

1.2 Objective and scope of the research

The objective of this research study was to develop elasto-plastic constitutive relations for unsaturated soils. Then, the elasto-plastic constitutive relations proposed in this study were used to develop the coupled formulations for unsaturated soils. The development and verification of elasto-plastic constitutive relations between stresses, strains, pore-water pressures and volume change of an unsaturated soil were the main objective of this research.

The triaxial apparatus was modified to incorporate NTU mini suction probes in order to measure pore-water pressures in the specimen during the test. The modified triaxial apparatus was used to conduct a series of soil-water characteristic curve (SWCC) tests, a series of isotropic consolidation tests, a series of consolidated drained tests and a series of shearing-infiltration tests.

The input parameters for elasto-plastic constitutive relation proposed in this study were determined from SWCC tests and isotropic consolidation tests. The results from these tests also show the effect of matric suction on consolidation curve, yield stress and the effect of net mean stress on SWCC curve, air-entry value, and yield suction.

The shear strength and total volume change characteristics obtained from consolidated drained triaxial tests and the shear strength, total volume change and

matric suction change characteristics obtained from shearing-infiltration tests described the behaviour of unsaturated compacted soil under consolidated drained and shearing-infiltration conditions that can be used to verify the elasto-plastic constitutive relations proposed in this study.

The scope of this research programme is as follows:

1. To develop elasto-plastic constitutive relations for unsaturated soils.
2. To develop coupled formulations for unsaturated soils which incorporate elasto-plastic constitutive relations proposed in this study.
3. To conduct a series of SWCC tests and a series of isotropic consolidation tests using the modified triaxial cell to obtain input parameters for the developed elasto-plastic constitutive relations.
4. To conduct a series of consolidated drained triaxial tests to obtain shear strength parameters, shear strength characteristics, water volume and total volume change characteristics.
5. To conduct a series of shearing-infiltration tests using the modified triaxial cell to obtain shear strength characteristics, total volume and matric suction change characteristics.
6. To compare the shear strength and total volume change during shearing under consolidated drained conditions; shear strength, total volume and matric suction changes during shearing-infiltration conditions obtained from the elasto-plastic constitutive relations proposed in this study with the measured results from consolidated drained triaxial and shearing-infiltration tests. The comparisons provide the verifications of the elasto-plastic constitutive relation proposed in this study.

1.3 Methodology

The study focuses on the development of elasto-plastic constitutive relations and develops coupled formulations using the developed elasto-plastic constitutive relations for unsaturated soils. Predicted solutions obtained from the developed constitutive relation were verified against the experimental data. The general framework constitutive relations for unsaturated soils as proposed by Alonso et al.

(1987, 1990) were adopted to develop the elasto-plastic constitutive relations. The general approach of the coupled model for unsaturated soil proposed by Dakshanamurthy et al. (1984) and Fredlund and Rahardjo (1993) was adopted in this study with the incorporation of the developed elasto-plastic constitutive relations.

In order to have identical specimen for laboratory test, the compacted sample using static compaction method was used. The triaxial apparatus was modified to incorporate NTU mini suction probes for measuring pore-water pressure of soil specimen during the tests. The SWCC tests were conducted under different net mean stresses using the modified triaxial apparatus. The isotropic consolidation tests were conducted under different matric suctions using the modified triaxial apparatus. The consolidated drained triaxial tests were conducted using the modified triaxial apparatus under different net mean stresses and different matric suctions to obtain the shear strength parameters, shear strength characteristics and total volume change characteristics. The shearing-infiltration tests were conducted using the modified triaxial apparatus under net mean stresses and different matric suctions to obtain shear strength, total volume change and matric suction change characteristics. For the case of shearing-infiltration tests, the soil specimens were sheared to a shear stress of around 80% of failure shear stress under the same matric suction. The developed elasto-plastic constitutive relations were used to predict the shear strength, total volume change characteristics during consolidated drained triaxial tests and shear strength, total volume and matric suction change characteristics during shearing-infiltration tests.

1.4 Outline of the report

The report is organized into seven chapters as follows:

Chapter 1 contains the introduction and objective of the research programme.

Chapter 2 presents a brief review of unsaturated soil mechanics and the literature on the volume change, elasto-plastic model and coupled equations for unsaturated soils.

Chapter 3 presents the development of elasto-plastic constitutive relation based on general frame work constitutive relation for unsaturated soils proposed by Alonso et

al. (1987, 1990) and formulation of coupled equations for unsaturated soils using the general approach proposed by by Dakshanamurthy et al. (1984) and Fredlund and Rahardjo (1993).

Chapter 4 presents the modification for the triaxial apparatus to incorporate NTU mini suction probes, selection of appropriate soil used for experimental programme, preparation of specimens, testing procedure for isotropic consolidation tests, SWCC tests consolidated drained triaxial tests and shearing-infiltration tests. The testing programme and prediction programme are also presented in this chapter.

Chapter 5 presents the effect of net mean stress on SWCC and the effect of matric suction on isotropic consolidation curve. The results of consolidated drained triaxial tests and shearing-infiltration tests are also presented.

Chapter 6 presents discussion of the results presented in Chapter 5. The results from SWCC tests and isotropic consolidation are discussed first. Then, the discussion on the results of consolidated drained triaxial tests and shearing-infiltration tests are presented. The prediction of the shear strength, total volume change during shearing under consolidated drained condition; shear strength, total volume change and matric suction change during shearing-infiltration condition using the developed elasto-plastic constitutive relations are also presented.

Chapter 7 presents the conclusions obtained from the study and recommendations for future research.

Appendix A presents the calibration data for the modified triaxial apparatus.

Appendix B presents the simulation results of consolidated drained tests and simulation results of shearing-infiltration tests.

CHAPTER 2 LITERATURE REVIEW

2.1 Introduction

This chapter presents the literature review on the topics that are related to this research.

2.2 Stress State Variables

In order to verify stress state for unsaturated soils ($(\sigma-u_a)$ and (u_a-u_w)), Bishop and Donald (1961) conducted null-type tests related to shear strength of an unsaturated Braehead silt. Fredlund (1973) conducted null-type tests related to volume change of an unsaturated compacted Kaolin. According to Lambe and Whitman (1979), unsaturated soil is a three-phase system (i.e., air phase, water phase and solid phase). Fredlund and Morgenstern (1977) proposed that contractile skin (i.e., air-water interface) is an additional phase (fourth phase) for an unsaturated soil. The mechanical behaviour of unsaturated soils (i.e., shear strength and volume change) can be described by three possible pairs of stress state variables. The three pair of stress state variables are $(\sigma-u_a)$ and (u_a-u_w) ; $(\sigma-u_w)$ and (u_a-u_w) ; $(\sigma-u_a)$ and $(\sigma-u_w)$.

where:

σ = total stress,

u_a = pore-air pressure,

u_w = pore-water pressure.

When a soil is unsaturated, the term (u_a-u_w) is greater than zero. The proposed stress state variables: $(\sigma-u_a)$ (i.e., net normal stress) and (u_a-u_w) (i.e., matric suction) for unsaturated soil are the most accepted combination of stress state variables for unsaturated soil mechanics.

2.3 Soil-water Characteristic Curve

The relationship between gravimetric water content, w , or volumetric water content, θ_w , and matric suction is commonly referred to as the soil-water characteristic curve (SWCC). The volumetric water content is defined as the ratio between volume of water and total volume of the soil (i.e., $\theta_w=V_w/V$). A typical SWCC is shown in

Figure 2.1. When the matric suction is increased from zero, the volumetric water content of the soil is nearly constant until it reaches the air-entry value (AEV) (i.e., the matric suction value at which air first enters the pore of the soil). Volumetric water content of the soil decreases rapidly when the matric suction is increased from AEV until it reaches the residual water content. The residual water content is defined as water content at which further increases in matric suction do not result in significant decreases in volumetric water content, and the matric suction at this point is called residual soil suction.

Figure 2.1 A typical soil-water characteristic curve

SWCC tests can be conducted in the laboratory using Tempe cell or a pressure plate apparatus. The procedure to conduct SWCC tests is described in ASTM D2325-68 (1994) and Fredlund and Rahardjo (1993). The axis translation technique proposed by Hilf (1956) is used in the apparatus. The soil specimen is placed on a high air-entry ceramic disk in a pressure plate. Water pressure inside the water compartment below the high air-entry ceramic disk remains constant at atmospheric pressure (i.e., $u_w=0$). During the test, air pressure, u_a , in the pressure plate is increased to a value above the atmospheric pressure. When the air pressure in the pressure plate is lower than the air-entry value of the disk, only water can pass through the ceramic disk. The difference between the air pressure and the water pressure is matric suction, $(u_a - u_w)$. When the air pressure is applied to the soil specimen, the pore-water is drained out of the specimen. When the matric suction of the soil specimen is

increased, air replaces water in soil pores, and the water content of the soil specimen decreases. At equilibrium, the soil has a water content corresponds to the controlled matric suction. A soil-water characteristic curve is obtained by measuring water contents of the specimen at different matric suctions.

The soil specimen in the Temple cell or the pressure plate is under a zero net normal stress. The volume of the soil specimen is normally assumed to remain constant during the test. However, the stress state in the field may not be under a zero net normal stress.

Thu et al. (2007) conducted SWCC tests in a triaxial apparatus. The soil specimens were isotropically consolidated to reach a target value before the matric suction was applied. The data show that the AEV increases with the increase in net confining stress. The slopes of the SWCC when the matric suction becomes greater than the AEV do not appear to depend on the net confining stress.

2.4 Consolidation Tests and the Controlling Factors

ASTM D2435 (1985) describes the conventional oedometer test for saturated soil specimens. In order to perform one-dimensional consolidation tests on unsaturated soil specimens, Barden and Sides (1970) modified Rowe cell for applying matric suction to the soil specimens. This modified apparatus was used by other researcher such as Fredlund and Morgenstern (1977) and Escario and Juca (1989). However, it was difficult to control the matric suction during the consolidation test.

Fredlund and Rahardjo (1986) used the modified Anteus oedometers to perform one dimensional consolidation tests on Kaolin specimens. The data show that the volume changes obtained from the experimental results and the theoretical analyses of volume change are similar.

Alonso et al. (1990); Sivakumar (1993); Wheeler and Sivakurmar (1995); Sivakumar and Ng (1998); Rampino et al. (1999); Cunningham et at. (2003); Thu et al. (2007) conducted isotropic consolidation tests on unsaturated specimen using triaxial apparatus under different matric suctions. The isotropic loading was applied by increasing the cell pressure at a constant rate.

Usually, the preconsolidation stress of a soil, p'_o , is determined from the result of oedometer tests. A series of methods to determine p'_o had been proposed (i.e., Casagrande, 1936; Schmertmann, 1955; and Janbu et al., 1981). The Casagrande's construction method to determine yield stress, p'_o , for consolidation tests has been used by most researchers (i.e., Leroueil et al., 1983; Sivakumar, 1993; Rampino et al., 2000; Thu et al. 2007)

2.5 Shearing-infiltration Tests and the Controlling Factors

Brand (1981) proposed that shear strength test under a constant total stress and increasing the pore-water pressures, could be used to simulate rainfall-induced slope failures. The stress path proposed by Brand (1981) is similar to the field stress path during rainfall. The stress path proposed by Brand (1981) was used by Anderson and Sitar (1995), Anderson and Riemer (1995), Han (1997), Melinda (1998), Wong et al. (2001), Ng and Chiu (2001), Ng and Chiu (2003) and Meilani et al. (2005).

Anderson and Sitar (1995) conducted constant shear drained tests in triaxial apparatus to simulate the field stress path on saturated soil specimens. The data show that the yield envelope is nonlinear. The axial strain increases significantly once the soil specimen reaches the yield point. The relationship between stress and strain is similar for all confining stresses.

Han (1997) conducted shearing-infiltration tests on unsaturated soil specimens in a modified direct shear apparatus. The study took into account the effect of hysteresis on soil strength. The soil specimens were sheared to a shear stress of 85-90% failure shear stress under the same matric suction. Water was then injected to the soil specimen while shear stress remained constant in order to bring the soil specimen to failure. The data from Han (1997) tests show that the shear stress at failure in the shearing-infiltration tests is higher than that from the constant suction shearing tests. For the same matric suction and on the same path (i.e., wetting or drying), the shear strength obtained for soils following a drying path is higher than the shear strength obtained for soils following a wetting path. The higher shear strength on the drying path than that on the wetting path could be attributed to the different in moisture contents at the same matric suction on the drying and wetting paths.

Melinda (1998) conducted the same tests as Han's (1997) on the same soil. The tests conducted by Melinda (1998) were at higher net normal stress than the tests conducted by Han (1997). The conclusion derived from the tests by Melinda (1998) was the same as the conclusion obtained from the tests by Han (1997).

In the studies conducted by Han (1997) and Melinda (1998), the pore-water pressure in the soil specimen was assumed to be the same as the water pressure recorded at the base of the soil specimen. The transient pore-water pressure in the soil specimen was unknown.

Wong et al. (2001) conducted shearing-infiltration tests in a triaxial apparatus with mini-pore water pressure probes installed along the soil specimen and found that the water pressure at the base compartment was higher than the pore-water pressure at the middle of the soil specimen. However, the mini pore-water pressure probe could not sustain high matric suctions. Data from Wong et al. (2001) show that the axial strain and volumetric strain increased significantly once the soil specimen reached the yield point.

Ng and Chiu (2003) conducted shearing-infiltration tests on decomposed granitic soil. The soil specimens were anisotropically consolidated under a constant stress ratio and under a constant matric suction of 40 kPa. Then, the matric suction in the soil specimen was reduced while the deviator stress and net mean stress remained constant. The matric suction at failure of the specimen under a high net mean stress was lower than matric suction at failure of the specimen under a low net mean stress. The behaviour of the soil specimen changed from dilative to contractive along with the change in net mean stress.

Meilani et al. (2005) conducted shearing-infiltration tests on compacted Kaolin specimens in a triaxial apparatus with three NTU mini probes installed along the soil specimen. The NTU mini probes developed by Meilani et al. (2002), have fast response and can measure matric suctions up to 500 kPa. Results from the test show that the failure envelope obtained from the shearing-infiltration tests is independent from the infiltration rate. A typical result from shearing-infiltration test is shown in **Figure 2.2**. The water pressure in water compartment is 10 kPa higher than the pore-water pressure in the soil specimen.

Figure 2.2 Deviator stress, matric suction, volumetric and water volumetric strain, and axial strain during shearing-infiltration test on soil specimen initially consolidated under 25 kPa net confining pressure and 100 kPa matric suction (after Meilani et al., 2005).

2.6 Flow of Water

An analysis of fluid flow requires a flow law to relate the flow rate with the driving potential using appropriate coefficients. The driving potential for water phase is hydraulic head (i.e., total head) gradient. The hydraulic head at point A in **Figure 2.3** can be calculated as follows (Fredlund and Rahardjo, 1993):

$$h_w = y + \frac{u_w}{\rho_w g} \quad (2.1)$$

where:

y = elevation of point A above the datum,

g = gravitational acceleration,

ρ_w = density of water,

u_w = pore-water pressure at point A.

Figure 2.3 Hydraulic head at point A (after Fredlund and Rahardjo, 1993)

“The driving potential causing flow in water phase has the same form for both saturated and unsaturated soils”. (Freeze and Cherry, 1979).

The flow of water in a saturated soil is generally described using Darcy’s law. Darcy (1856) postulated that the rate of water flow through a soil mass was proportional to the hydraulic head gradient.

$$v_w = -k_w \left\{ \frac{\partial h_w}{\partial y} \right\} \quad (2.2)$$

where:

v_w = flow rate of water

k_w = coefficient of permeability with respect to water phase

$\frac{\partial h_w}{\partial y}$ = hydraulic head gradient in y-direction which can be designated as i_{wy} .

The coefficient of permeability is relatively constant for a specific saturated soil. Equation (2.2) can also be written for the x - and z -direction. The negative sign in Equation (2.2) indicates that water flows in the direction of decreasing hydraulic head.

The coefficient of permeability with respect to water phase, k_w , is a measure of the space available for water to flow through the soil. The coefficient of permeability depends on the fluid properties and the porous medium.

Lambe and Whitman (1979) published numerous experimental results from variety of soils. The data show that the saturated coefficient of permeability, k_s , can be expressed in a logarithmic scale versus the void ratio, e , and the relationship appears to be close to a straight line for most of the soils.

Based on the tests conducted by Childs and Collis-George (1950), Darcy's law is also applicable to the flow of water through unsaturated soils. Their results indicated that the soil coefficient of permeability with respect to water is a function of water content in unsaturated soil. Lloret and Alonso (1980) and Fredlund (1981) reported that the soil permeability with respect to water must be expressed as a function of the volume mass soil properties.

Fredlund and Rahardjo (1993) emphasized that in unsaturated soil, the hydraulic conductivity is dependent on the change in void ratio and the change in degree of saturation (or water content) of the soil. When a soil becomes unsaturated, air first replaces some of the water in the large pores and this causes water to flow through the smaller pores with an increasing tortuosity to the flow path. A further increase in the matric suction of the soil leads to a further decrease in the pore volume

occupied by water. Therefore, the soil permeability with respect to water decreases rapidly as the space available for the water flow reduces.

For an unsaturated soil, the change in void ratio due to a change in net stress state is not significant and the soil permeability with respect to water can be expressed as a function of water content of the soil. A change in matric suction usually produces a more significant change in water content than a change in net normal stress does. As a result, SWCC has been used to derive semi-empirical relationships for the soil permeability with respect to water as a function of matric suction when the change in void ratio is not significant (Gardner and Fireman, 1958; Gardner, 1958; Brooks and Corey, 1964; van Genuchten, 1980; Mualem, 1986; Fredlund and Xing, 1994).

When the change in void ratio due to a change in net normal stress is significant, the soil permeability with respect to water can be expressed as a function of degree of saturation and void ratio of the unsaturated soil. Lloret and Alonso (1980) presented an empirical equation to estimate the soil permeability with respect to water as a function of void ratio and degree of saturation.

Chang and Duncan (1983) presented the soil permeability with respect to water as a function of both void ratio and degree of saturation.

Huang (1994) proposed theoretical functions for the soil coefficient of permeability with respect to water for a deformable unsaturated soil. There were two proposed equations, one for a compressible soil and another for a swelling soil. A compressible soil is defined as the soil where the change in the net normal stress produces significant volume changes. Swelling soil is defined as the soil which produces significant volume changes as a result of changes in matric suction.

2.7 Volume Change of Unsaturated Soils

2.7.1 General

Unsaturated soil consists of four phases. When a stress gradient is applied, two phases can flow (i.e., air phase and water phase), the other two phases come to equilibrium (i.e., soil structure and contractile skin). The volume changes associated with the contractile skin can be neglected. The constitutive relationships for the

three phases can be formulated by relating volume changes to changes in stress state variables.

Terzaghi (1936), Biot (1941), Bishop (1959) Coleman (1962) Bishop and Blight (1963), Matyas and Radhakrishna (1968), Barden et al. (1969), Aitchison et al. (1973), Fredlund and Morgenstern (1976), Lloret and Alonso (1985), Ho (1988), Fredlund and Rahardjo (1993), Wheeler and Sivakumar (1995) have proposed many constitutive relationships for volume change of unsaturated soil. In general, the volumetric behaviour of an unsaturated soil can be described in a three-dimensional space as state surface $\{(\sigma - u_a), e, (u_a - u_w)\}$.

In most cases, two constitutive relationships are presented to describe the volume change associated with an unsaturated soil, one relationship for soil structure (in terms of void ratio or volumetric strain) and another relationship for the water phase (in terms of degree of saturation or water content). Fredlund and Morgenstern (1976) proposed constitutive relationships for unsaturated soil that prove to be suitable in explaining the behaviour of unsaturated soils.

2.7.2 Constitutive Relations

Fredlund and Morgenstern (1976) proposed the continuity requirement for an infinitesimal element of unsaturated soil as follows:

$$\frac{\Delta V_v}{V_o} = \frac{\Delta V_w}{V_o} + \frac{\Delta V_a}{V_o} \quad (2.3)$$

where:

V_o = initial volume of an infinitesimal unsaturated soil,

V_v = volume of the soil voids,

V_w = volume of water in the infinitesimal unsaturated soil, and

V_a = volume of air in the infinitesimal unsaturated soil.

The total volume change can be calculated as follows:

$$\varepsilon_v = \frac{\Delta V_v}{V_o} = \varepsilon_x + \varepsilon_y + \varepsilon_z \quad (2.4)$$

where:

- ε_v = volumetric strain,
- ε_x = normal strain component in the x -direction,
- ε_y = normal strain component in the y -direction,
- ε_z = normal strain component in the z -direction.

2.7.2.1 Soil Structure Constitutive Relations

For a saturated soil, the constitutive relations for the soil structure can be formulated based on the generalized Hooke's law using the effective stress variable, $(\sigma - u_w)$. For an isotropic and linear elastic soil structure, the constitutive relations can be expressed using the following equations:

$$\begin{aligned}\varepsilon_x &= \frac{1}{E}(\sigma_x - u_w) - \frac{\mu}{E}[(\sigma_y - u_w) + (\sigma_z - u_w)] \\ \varepsilon_y &= \frac{1}{E}(\sigma_y - u_w) - \frac{\mu}{E}[(\sigma_z - u_w) + (\sigma_x - u_w)] \\ \varepsilon_z &= \frac{1}{E}(\sigma_z - u_w) - \frac{\mu}{E}[(\sigma_x - u_w) + (\sigma_y - u_w)]\end{aligned}\tag{2.5}$$

where:

- σ_x = total stress in the x -direction,
- σ_y = total stress in the y -direction,
- σ_z = total stress in the z -direction,
- E = modulus of elasticity or Young's modulus for the soil structure,
- μ = Poisson's ratio.

The constitutive relations for unsaturated soil are formulated based on similar equations used for saturated soil using the appropriate stress state variables for unsaturated soil. Fredlund and Morgenstern (1976) proposed the soil structure constitutive relations as follows:

$$\begin{aligned}
 \varepsilon_x &= \frac{1}{E}(\sigma_x - u_a) - \frac{\mu}{E}[(\sigma_y - u_a) + (\sigma_z - u_a)] + \frac{1}{H}(u_a - u_w) \\
 \varepsilon_y &= \frac{1}{E}(\sigma_y - u_a) - \frac{\mu}{E}[(\sigma_z - u_a) + (\sigma_x - u_a)] + \frac{1}{H}(u_a - u_w) \\
 \varepsilon_z &= \frac{1}{E}(\sigma_z - u_a) - \frac{\mu}{E}[(\sigma_x - u_a) + (\sigma_y - u_a)] + \frac{1}{H}(u_a - u_w) \quad (2.6) \\
 \gamma_{xy} &= \frac{1}{G}\tau_{xy} \\
 \gamma_{yz} &= \frac{1}{G}\tau_{yz} \\
 \gamma_{zx} &= \frac{1}{G}\tau_{zx}
 \end{aligned}$$

where:

E = modulus of elasticity for the soil structure with respect to a change in the net normal stress, $(\sigma - u_a)$,

H = modulus of elasticity for the soil structure with respect to a change in the matric suction, $(u_a - u_w)$,

γ_{xy} = shear strain on x-plane (i.e., $\gamma_{xy} = \gamma_{yx}$),

γ_{yz} = shear strain on y-plane (i.e., $\gamma_{yz} = \gamma_{zy}$),

γ_{zx} = shear strain on z-plane (i.e., $\gamma_{zx} = \gamma_{xz}$),

τ_{xy} = shear stress on x-plane (i.e., $\tau_{xy} = \tau_{yx}$),

τ_{yz} = shear stress on y-plane (i.e., $\tau_{yz} = \tau_{zy}$),

τ_{zx} = shear stress on x-plane (i.e., $\tau_{zx} = \tau_{xz}$),

G = shear modulus.

The soil structure constitutive Equations (2.6) can also be applied to the cases where the relationships between stress and strain are non-linear. An incremental procedure using small increments of stress and strain can be used to apply linear elastic formulation to a non-linear elastic problem. The non-linear stress strain curve is

assumed to be linear within each stress and strain increment. The elastic modulus, E , and H , may vary in magnitude from one increment to the other. The soil structure constitutive relationships can be written in an incremental form as follows:

$$\begin{aligned}
 d\varepsilon_x &= \frac{1}{E}d(\sigma_x - u_a) - \frac{\mu}{E}d[(\sigma_y - u_a) + (\sigma_z - u_a)] + \frac{1}{H}d(u_a - u_w) \\
 d\varepsilon_y &= \frac{1}{E}d(\sigma_y - u_a) - \frac{\mu}{E}d[(\sigma_z - u_a) + (\sigma_x - u_a)] + \frac{1}{H}d(u_a - u_w) \\
 d\varepsilon_z &= \frac{1}{E}d(\sigma_z - u_a) - \frac{\mu}{E}d[(\sigma_x - u_a) + (\sigma_y - u_a)] + \frac{1}{H}d(u_a - u_w) \quad (2.7) \\
 d\gamma_{xy} &= \frac{1}{G}d\tau_{xy} \\
 d\gamma_{yz} &= \frac{1}{G}d\tau_{yz} \\
 d\gamma_{zx} &= \frac{1}{G}d\tau_{zx}
 \end{aligned}$$

Volumetric strain of the soil structure for each increment, $d\varepsilon_v$, can be calculated by summing the change in the normal strain in the x -, y -, and z -directions:

$$d\varepsilon_v = d\varepsilon_x + d\varepsilon_y + d\varepsilon_z \quad (2.8)$$

where:

$d\varepsilon_v$ = change in volumetric strain for each increment.

Substituting Equation (2.7) into Equation (2.8) yields the volumetric strain for a particular loading increment:

$$d\varepsilon_v = 3\left(\frac{1-2\mu}{E}\right)d\left(\frac{\sigma_x + \sigma_y + \sigma_z}{3} - u_a\right) + \frac{3}{H}d(u_a - u_w) \quad (2.9)$$

Equation (2.9) can be simplified to the following form:

$$d\varepsilon_v = 3\left(\frac{1-2\mu}{E}\right)d(\sigma_m - u_a) + \frac{3}{H}d(u_a - u_w) \quad (2.10)$$

where:

σ_m = average total normal stress (i.e., $(\sigma_x + \sigma_y + \sigma_z)/3$).

Fredlund and Rahardjo (1993) proposed the constitutive equation for soil structure in a compressible form for the three-dimensional loading condition as follows:

$$d\epsilon_v = m_1^s d(\sigma_m - u_a) + m_2^s d(u_a - u_w) \quad (2.11)$$

where:

m_1^s = coefficient of volume change with respect to a change in net normal stress (i.e., $m_1^s = 3(1-2\mu)/E$),

m_2^s = coefficient of volume change with respect to a change in matric suction (i.e., $m_2^s = 3/H$).

The constitutive Equations (2.7) have been experimentally tested for uniqueness (Fredlund and Morgenstern, 1976; Matyas and Radhakrskna, 1968; Braden et al., 1969). The results of the tests indicate that as long as the deformation conditions are monotonic, Equation (2.7) is unique. The loading constitutive relations for soil structure is presented graphically in the form of a constitutive surface in **Figure 2.4**

Figure 2.4 Soil structure constitutive surface for an unsaturated soil (after Fredlund and Rahardjo, 1993)

Figure 2.5 presents the plot of the void ratio constitutive surface with respect to the logarithm of the stress state variables. The cross section at a constant void ratio plotted using logarithmic scale is shown in **Figure 2.6**

Figure 2.5 Void ratio constitutive surface for an unsaturated soil (after Fredlund and Rahardjo, 1993)

Figure 2.6 Cross section at a constant void ratio (after Fredlund and Rahardjo, 1993)

2.7.2.2 Water Phase Constitutive Relations

The water phase constitutive relations describe the change in the volume of water in the infinitesimal element of soil under various stress conditions. Fredlund and Rahardjo (1993) proposed the water phase constitutive relations using a semi-

empirical approach based on a linear combination of the stress state variables. The constitutive equations can be written in an elastic form as follows:

$$d\varepsilon_w = \frac{3}{E_w} d(\sigma_m - u_a) + \frac{1}{H_w} d(u_a - u_w) \quad (2.12)$$

where:

E_w = water volumetric parameter associated with a change in the net normal stress (i.e., $(\sigma - u_a)$),

H_w = water volumetric parameter associated with a change in the matric suction (i.e., $(u_a - u_w)$).

The compressibility form for the water constitutive relationship under a general loading can be written as follows:

$$\frac{dV_w}{V_o} = m_1^w d(\sigma_m - u_a) + m_2^w d(u_a - u_w) \quad (2.13)$$

where:

m_1^w = coefficient of water volume change with respect to net normal stress,

m_2^w = coefficient of water volume change with respect to matric suction.

The loading constitutive relations for the water phase is presented graphically in the form of the constitutive surface in **Figure 2.7**. **Figure 2.8** presents the plot of the void ratio constitutive surface with respect to the logarithm of the stress state variables.

Figure 2.7 Water phase constitutive surface for an unsaturated soil (after Fredlund and Rahardjo, 1993)

ϕ' = the effective of friction angle of saturated soil,

σ = total normal stress,

χ = a numerical coefficient ranging from 0 to 1,

u_a = pore-air pressure,

u_w = pore-water pressure.

Fredlund et al. (1978) proposed a shear strength equation for unsaturated soil using two independent stress state variables (i.e., net normal stress, $(\sigma - u_a)$, and matric suction, $(u_a - u_w)$) as follows:

$$\tau_{ff} = c' + (\sigma_f - u_a)_f \tan \phi' + (u_a - u_w)_f \tan \phi^b \quad (2.15)$$

where:

τ_{ff} = shear stress on the failure plane at failure,

c' = intercept of the “extended” Mohr-Coulomb failure envelope on the shear stress axis when net normal stress are equal to zero. It is also referred to as “effective cohesion”,

ϕ' = angle of internal friction associated with the net normal stress state variable, $(\sigma_f - u_a)_f$,

ϕ^b = angle indicate the rate of change in shear strength relative to change in matric suction, $(u_a - u_w)_f$,

$(\sigma_f - u_a)_f$ = net normal stress on the failure plane at failure,

$(u_a - u_w)_f$ = matric suction at failure,

σ_f = total normal stress on the failure plane at failure,

u_a = pore-air pressure at failure,

u_w = pore-water pressure at failure.

Equation (2.15) can be considered as an extension of shear strength equation for a saturated soil. For an unsaturated soil u_w is smaller than u_a . At saturation u_w

approaches u_a , causing matric suction to approach zero (i.e., $u_w = u_a$). Equation (2.15) reduces to the familiar shear strength equation of a saturated soil as follows:

$$\tau_{ff} = c' + (\sigma_f - u_w)_f \tan \phi' \quad (2.16)$$

The Mohr-Coulomb failure envelope for saturated soil is usually drawn as a two-dimensional graph of shear stress versus effective normal stress. The failure envelope for unsaturated soils can be described in a three-dimensional plot. The three dimensional plot of failure envelope for unsaturated soils is an extension of the two-dimensional graph of shear stress versus effective net normal stress into the third dimension with the matric suction as the third axis, $(u_a - u_w)$. **Figure 2.9** shows the two dimensional graph of failure envelope for saturated soils, and **Figure 2.10** shows the three-dimensional plot of failure envelope for unsaturated soils.

Figure 2.9 Mohr-Coulomb failure envelope for saturated soils (after Fredlund and Rahardjo, 1993)

Figure 2.10 Extended Mohr-Coulomb failure envelope for unsaturated soils (after Fredlund and Rahardjo, 1993)

Based on the results of shear strength tests on unsaturated soils that were conducted by Escario and Saez (1986), Gan (1986), Fredlund et al. (1987), Gan et al. (1988), the magnitude of ϕ^b varies over a large range of matric suction. Usually, the tangent of ϕ^b is smaller or equal to tangent of ϕ' . On the other hand, Escario and Saez (1986), Melinda et al. (2004) found that ϕ^b can be greater than ϕ' angle.

Gan (1986), Fredlund et al. (1987) and Gan et al. (1988) conducted a multistage direct shear tests on unsaturated glacial till. Gan's results show that the failure envelope on the shear stress versus matric suction is non-linear. The ϕ^b angle is 25.5° at low matric suctions and decreases to 5° - 7° at high matric suctions.

Figure 2.11 Failure envelope for unsaturated soil glacial till specimens. (a) Failure envelope on the, τ , against $(u_a - u_w)$ plane; (b) ϕ^b values versus matric suction (after Gan, 1986; Fredlund et al. 1987).

Thu et al. (2006) conducted constant water content tests on unsaturated Kaolin in a modified triaxial apparatus. The results from the tests show that the relationship between ϕ^b and matric suction is non-linear.

Figure 2.12 Non-linearity relationship between ϕ^b and matric suction at failure of constant water content triaxial tests (after Thu et al., 2006).

2.9 Review the Elasto-plastic Model

2.9.1 Basic Concept of Critical State Model

Critical state soil mechanics for saturated soil can be defined in terms of three state variables, mean effective stress, p' , deviator stress, q , and specific volume, v , (Schofield and Wroth, 1968). For the conditions of the triaxial test, ($\sigma_2 = \sigma_3$), the state variables are defined as follows:

$$q = \sigma_1 - \sigma_3 \quad (2.17)$$

$$p' = \left(\frac{\sigma_1 + \sigma_2 + \sigma_3}{3} \right) - u_w \quad (2.18)$$

$$v = 1 + e \quad (2.19)$$

where:

σ_1 = major principal stress,

σ_2 = intermediate principal stress,

σ_3	= minor principal stress,
q	= deviator stress,
p'	= mean effective stress of a saturated soil,
u_w	= pore-water pressure,
v	= specific volume,
e	= void ratio

Casagrande (1936) named a critical state density as the density at which soil continues to deform at a constant shear strength. Rendulic (1936) indentified that the relationship between void ratio and changes in stress is unique for a given soil at specified initial stress and stress history. This concept was supported by conducting drained and undrained triaxial tests on saturated soils and it was found that constant void ratio contours plotted from the drained test data on axes of q and p' coincided with the contour plotted from the undrained test data.

When a saturated soil specimen is sheared, it will reach a critical state where shear strain continues to increase without any change in deviator stress, q , or specific volume, v . At critical state the soil behaves like frictional fluid (Schofield and Wroth, 1968).

In general, yield is associated with a transition from stiff to less stiff response (Wood, 1990). A typical yield surface in stress space is shown in **Figure 2.13**. Yield surface separates stress states which produce elastic response from stress states which produce plastic response. Usually, yield surface for a saturated soil is assumed to be an ellipse and its initials size on the major axis is determined by the preconsolidation stress, p'_o . All stress states (i.e., combination of p' and q) that lie within yield surface (e.g., point A in **Figure 2.13**) will cause the soil to respond elastically. If the stress state lies on the initial yield surface (e.g., point B in **Figure 2.13**) the soil is yielding. Any tendency of a stress combination to move outside the initial yield surface cause the yield surface to expand to a lager yield surface corresponding to a new preconsolidation stress (e.g., p'_{oi} in **Figure 2.13**).

Figure 2.13 Stress increment expanding of yield surface (after Wood, 1990)

According to Schofield and Wroth (1968), Wood (1990) and Budhu (2000), the relationship between specific volume, v , and logarithm of mean effective stress, p' , of soil in an isotropic normally consolidated state is linear. **Figure 2.14** shows the critical state line and critical state parameters.

Normal compression line (i.e. NCL) is the combinations of specific volume v and mean effective stress, p' , at various stages of normal compression (Wood, 1990). When a soil is under an isotropic stress state, the deviator stress, q , will be equal to zero. As a result, the isotropic normal compression line is determined by two equations as follows:

$$q = 0 \quad (2.20)$$

$$v_o = N(0) - \lambda(0) \ln p'_o \quad (2.21)$$

where:

$N(0)$ = the specific volume of the normal compression line at reference stress,

$\lambda(0)$ = the stiffness of the saturated soil, which is obtained from an isotropic or one dimensional consolidation test,

κ = unloading/reloading index or stiffness, which is obtained from an isotropic or one-dimensional consolidation test,

v_o = the specific volume on the normal consolidation line at the preconsolidation stress p'_o .

Figure 2.14 Critical state line and these parameter. (a) Yield surface; (b) CSL on $(v - p')$ space; (c) CSL on $(v - \ln p')$ space (after Schofield and Wroth, 1968; Wood, 1990 and Budhu, 2000)

When a soil is sheared it will reach a critical state. The critical states were found to lie on a unique line in the $(q - v - p')$ space. This line is called the critical state line (i.e., CSL in **Figure 2.14**). The critical state line is determined by two equations as follows (Schofield and Wroth, 1968):

$$q = Mp' \quad (2.22)$$

$$v_f = \Gamma(0) - \lambda(0) \ln p'_f \quad (2.23)$$

where:

$\Gamma(0)$ = specific volume on the critical state line for saturated soil at reference stress,

v_f = specific volume at failure,

p'_f = mean effective stress at failure.

Schofield and Wroth (1968) and Roscoe and Burland (1968) proposed an equation for ellipsoidal yield surface as follows:

$$(p')^2 - p'p'_o + \frac{q^2}{M^2} = 0 \quad (2.24)$$

where:

p'_o = preconsolidation stress of saturated soil,

M = slope of the critical state line.

The critical state line (CSL) and the yield surface intersect at $p' = p'_o / 2$. Budhu (2000) proposed a failure line equation in the $(q - p')$ space as follows:

$$q_f = Mp_f' \quad (2.25)$$

where:

q_f = deviator stress at failure,

p_f' = mean effective stress at failure.

2.9.2 Review of Elasto-plastic Model for Unsaturated Soil

The role of matric suction or degree of saturation on the volume change, shear strength and excess pore-water pressure have been investigated by Bishop and Donald (1961), Fredlund and Morgenstern (1977), Fredlund et al. (1978), Alonso et al. (1987), Fredlund and Rahardjo (1993), Wheeler and Karube (1995), Wheeler and Sivakumar (1995), Delage and Graham (1995), Graham et al. (1995), Wheeler (1996), Rampino et al. (1999), Tang and Graham (2002), Chiu and Ng (2003), Thu et al. (2006).

Alonso et al. (1987, 1990) proposed the general framework for the constitutive model for unsaturated soil. This model has been refined by Toll (1990), Thomas and He (1994), Wheeler and Sivakumar (1995), Cui and Delage (1996), Wheeler (1996), Bolzon et al. (1996), Rampino et al. (1999), Simoni and Schrefler (2001), Tang and Graham (2002), Chiu and Ng, 2003, and Thu et al. (2007). These models were developed based on the independent stress state variables. On the other hand, there were different assumptions and constitutive relationships used in these models.

Alonso et al. (1990) proposed an elasto-plastic model with four state variables: net mean stress, p , deviator stress, q , matric suction, s , and specific volume, v . The model includes yield surface, flow rules, hardening law, elastic and plastic strains. The plastic strains occur when the yield surface is reached. Wheeler and Sivakumar (1995) proposed an elasto-plastic model that is similar to the Alonso et al. (1990) model. Wheeler et al. (2003) and Gallipoli et al. (2003b) formulated an elasto-plastic model for unsaturated soils using a single stress state variable. On the other hand, Fredlund and Morgenstern (1977) suggested that matric suction should be treated independently, and proposed that the constitutive relationships of unsaturated soils be described using two independent stress state variables: net normal stress, $(\sigma - u_a)$ and matric suction $(u_a - u_w)$.

In the model for unsaturated soils as proposed by Alonso et al. (1990), the critical state framework for saturated soils was extended to include matric suction as one of the stress state variables for unsaturated soil. The loading-collapse (LC) yield curve (i.e., Equation (2.26)) was derived by considering the response to isotropic loading of two specimens subjected to different matric suctions: $s = 0$ (i.e., saturated case) and a higher suction s , and a virtual path through point 1, 2 and 3 as shown in **Figure 2.15**.

$$\left(\frac{p_o}{p^c} \right) = \left(\frac{p_o^*}{p_c} \right)^{(\lambda(0)-\kappa)/(\lambda(s)-\kappa)} \quad (2.26)$$

where:

p^c = reference stress,

p_o = yield stress,

p_o^* = yield stress for saturated condition,

κ = elastic stiffness parameter for change in net mean stress,

$\lambda(s)$ = stiffness parameter for change in net mean stress for virgin state of the soil.

β = parameter controlling the rate of increase of soil stiffness with suction,

r_c = parameter defining the maximum soil stiffness.

The yield suction curve was assumed as a straight line parallel to p axis and was named SI (i.e., suction increase). The loading-collapse (LC) yield curve and suction increase (SI) yield curve are shown in **Figure 2.16**

Figure 2.16 Loading collapse (LC) and suction increase (SI) yield curves (after Alonso et al., 1990)

Delage and Graham (1995) suggested that there is a link between the LC and the SI yield curves. Sivakumar and Ng (1998) conducted five isotropic consolidation tests on unsaturated compacted kaolin in the modified Bishop Wesley stress path apparatus. The results from the tests show that the SI is inclined at a positive angle in the p - s space. Sivakumar and Ng (1998) also suggested a possible form of unique yield curve.

Toll (1990) conducted triaxial tests on compacted Kiunyu gravel. The specimens were statically compacted at different water contents. Most of the tests were conducted by shearing the specimens at constant water content conditions. The degree of saturation was considered as a factor affecting the slope of the critical

state line on the $(p-q)$ and $(v-p)$ planes (i.e., M and λ). The equations for deviator stress, q , and specific volume, v , at critical state were proposed as follows:

$$q = M_a (p - u_a) + M_w (u_a - u_w) \quad (2.28)$$

$$v = \Gamma_{aw} - \lambda_a \ln(p - u_a) - \lambda_w \ln(u_a - u_w) \quad (2.29)$$

where:

M_a = slope of critical state with respect to net mean stress on $(p-q)$ plane,

$(p - u_a)$ = net mean stress,

M_w = slope of critical state with respect to matric suction on $(q-(u_a-u_w))$ plane,

v = specific volume,

Γ_{aw} = the specific volume when both $(p-u_a)$ and (u_a-u_w) equal unity,

λ_a = slope of critical state with respect to net mean stress on $(v-p)$ plane,

λ_w = slope of critical state with respect to matric suction on $(v-(u_a-u_w))$ plane.

The parameters M_a , M_w , λ_a , λ_w , and Γ_{aw} were factors which are a function of degree of saturation, S .

Fredlund, Rahardjo and Gan (1991) proposed that a unique critical state relationships for q and v might be anticipated if the initial soil structure was destroyed during shear to produce a similar structure at the end of the tests (i.e., critical state). Fredlund, Rahardjo and Gan (1991) proposed that the soils compacted at different water contents or compaction efforts should be considered as different soils. Fredlund, Rahardjo and Gan (1991) also suggested that identical soil specimens should be used in order to establish constitutive relationships (i.e., shear strength, volume change) for unsaturated soils.

Re-analyzing some of Toll (1990) data, Wheeler (1991) showed that it was difficult to apply Equation (2.28) and (2.29) because Toll (1990) equations depend on degree of saturation, and there would be no way to predict the change in degree of saturation.

Wheeler (1991) defined a critical state volumetric (i.e., specific water volume) relationship in terms of net mean stress and matric suction.

$$v_w = \Gamma(s) - \lambda(s) \ln(p - u_a) + f(u_a - u_w) \quad (2.30)$$

where:

v_w = specific water volume,

$\Gamma(s)$ = specific water volume at reference stress,

$\lambda(s)$ = slope of the normal compression curve.

Rampino et al. (2000) proposed critical state relationships for the degree of saturation, S , as a replacement for specific water volume which was proposed by Wheeler (1991) as follows:

$$S = \Gamma_s(s) - \psi_s(s) \ln \left[\frac{(p - u_a)}{(p - u_a)_{ref}} \right] \quad (2.31)$$

where:

$\Gamma_s(s)$ = degree of saturation at the reference stress of the critical state line,

$\psi_s(s)$ = slope of the critical state line for the degree of saturation,

$(p - u_a)_{ref}$ = net mean stress at reference stress.

2.10 Coupled Equations

The mechanical behaviour of unsaturated soils is directly dependent on the change in pore-water pressure and pore-air pressure. There are two classes of pore-water pressures that may develop in the field. The first one is pore-water pressures associated with seepage through soils. The second one is pore-water pressures that are developed from the application of an external load. When pore-water pressures are developed, the transient process of water flow occurs. Transient process of water flow changes the equilibrium conditions of the unsaturated soil because it changes stress state in the soil. Volume change occurs since the soil structure reaches a new equilibrium condition. In sequence, volume change alters the hydraulic properties of soil structure and thus influences the transient process of

water flow through the soil. Therefore, the interactive processes between stress and strain are primary mechanical behaviour of unsaturated soils.

The stress-strain process is governed by the equilibrium equations. The water flow is governed by the water continuity equation. An accurate solution for mechanical behaviour of an unsaturated soil requires that the equilibrium equations and the continuity equation be solved simultaneously as a coupled solution. In the coupled approach, the simultaneous solution of the two equations (i.e., equilibrium equations and continuity equation) means that the dynamic interdependence between seepage and deformation problems is fully realized. In the uncoupled approach, the water phase continuity (i.e., seepage) is solved independently from the equilibrium (i.e., stress-strain) equations.

The coupled equations are usually formulated based on some general assumptions: 1) material is isotropic, 2) reversibility of stress-strain relations, 3) small strains, 4) pore-water is incompressible, 5) coefficient of permeability of water and air phases are functions of volume-mass properties during the consolidation process, 6) effect of air diffusing through water, air dissolving in the water phase, and the movement of water vapor are ignored (Biot, 1941; Dakshanamurthy et al., 1984; Fredlund and Rahardjo, 1993)

Terzaghi (1943) proposed the classical theory for one-dimensional consolidation for saturated soil. This theory was formulated based on the constitutive equation (i.e., the deformation of the soil structure with respect to changes in the stress state) and a flow law (i.e., Darcy law). The effective stress, $(\sigma - u_w)$, was used to describe the behaviour of saturated soil. The continuity requirement for a saturated soil element was that the volume change of a soil element be equal to the change in the volume water in the element. Terzaghi's one-dimensional equation can be written as follows:

$$\frac{\partial u_w}{\partial t} = c_v \frac{\partial^2 u}{\partial y^2} \quad (2.32)$$

where:

c_v = coefficient of consolidation,

$$c_v = \frac{k}{m_v \rho_w g}$$

k = coefficient of permeability with respect to water at saturation,

m_v = coefficient of volume change for saturated soils,

ρ_w = water density,

g = gravitational acceleration.

Biot (1941) presented a general coupled solution for consolidation of unsaturated soils. The principle of effective stress was used to formulate two constitutive equations relating stress and strain and pore-water pressure, u_w . One equation relates the stress state to the void ratio and the other equation relates the stress state to the water content of the soil. The coupled equations were formulated based on the assumption that occluded air bubbles existed in soil during the consolidation process. For one-dimensional consolidation, the equation of Biot was similar to the Terzaghi's equation (i.e., Equation (2.32)). The difference was that the coefficient of consolidation, c_v , was modified to take into account the compressibility of the pore-fluid (i.e., air existed in the soil during consolidation). The other researchers such as Larmour (1966), Hill (1967), and Olson (1986) came to the same conclusion with respect to the modified coefficient of consolidation, Equation (2.32) can be used to describe the consolidation of unsaturated soil with occluded air bubbles.

Fredlund and Hasan (1979) proposed that pore-air pressure and pore-water pressures during consolidation process could be solved by two partial differential equations. Air phase was assumed to be continuous; Fick's law was used to describe air flow. Darcy's law was used to describe water flow. The coefficients of permeability with respect to both water phase and air phase were functions of matric suction. The constitutive relations for volume change in unsaturated soil as proposed by Fredlund and Morgenstern (1976) were used. Fredlund and Rahardjo (1986) used these Equations to simulate the volume change behaviour of compacted Kaolin specimens during total stress change and matric suction change. Lloret and Alonso (1981) proposed similar consolidation equations as Fredlund and Hasan's equations (1979).

Dakshanamurthy et al. (1984) and Fredlund and Rahardjo (1993) proposed a coupled three-dimensional consolidation theory for unsaturated porous media. The coupled equations were formulated based on the use of two independent stress state variables and the constitutive relations for volume change in unsaturated soil as proposed by Fredlund and Morgenstern (1976), (1977). The air phase was assumed to be continuous.

Rahardjo and Fredlund (1995) conducted five series of tests on silty sand to study the behaviour of unsaturated soils during undrained loading and consolidation. The data showed that the pore-air pressures dissipated instantaneously when the air phase is continuous. The experimental pore pressure parameters obtained from the undrained loading and constant water content loading agreed reasonably well with the theory.

Following the approach proposed by Dakshanamurthy et al. (1984), Conte (2004) derived the differential equations governing the coupled and uncoupled consolidation for unsaturated soil under plane strain condition and then solved them numerically. Conte (2006) extended Conte (2004) solution from plane strain condition to axial symmetry condition and the soil properties were updated at each time step according to the current stress state.

Vu and Fredlund (2006) used similar coupled equations as Dakshanamurthy et al. (1984), Fredlund and Rahardjo (1993), Conte (2004) and Conte (2006) to predict heave in expansive soils. The soil properties (i.e., m_1^s , m_2^s , m_1^w and m_2^w) could be obtained from the constitutive surface for soil structure and the constitutive surface for the water phase. A finite element computer program was developed to solve the coupled equations.

2.11 Summary

The SI yield curve as proposed by Alonso et al., (1990) is a straight line parallel to p axis. The experimental data on the SI yield curve from Sivakumar and Ng (1998) and Thu et al. (2007) show that the SI yield curve is inclined at a positive angle in the p-s space. Delage and Graham (1995) suggested that there is a link between the LC and the SI yield curves. Sivakumar and Ng (1998) also suggested a possible

form of a unique yield curve. Therefore, it is necessary to develop an improved formulation for the SI yield curve that can describe well its characteristics.

The coupled equations proposed by Biot (1941), Fredlund and Hasan (1979), Dakshanamurthy et al. (1984), Fredlund and Rahardjo (1993), Conte (2004), Conte (2006), Vu and Fredlund (2006) are based on an assumption that the relationships between stress and strain are reversible. In reality, the relationships between stress and strain are irreversible. The elasto-plastic models for unsaturated soils can be used for estimating the shear strength, excess pore-water pressure and volume change (Wheeler, 1996; Thu, 2005). Therefore, it would be appropriate to formulate the coupled equations based on the elasto-plastic model for unsaturated soils.

The soil-water characteristic curve (SWCC) tests and isotropic consolidation tests provide useful information related to the behaviour of unsaturated soils in (p-s) plane and provide input parameters for the proposed elasto-plastic model. The consolidated drained (CD) triaxial tests and shearing-infiltration tests illustrate the behaviour of unsaturated soils along the two stress paths commonly occur in the field. Therefore, SWCC tests, isotropic consolidation tests, CD triaxial tests and shearing-infiltration tests could be used in the experimental programme.

CHAPTER 3

ESTABLISHED CONSTITUTIVE RELATIONS AND PROPOSED COUPLED FORMULATIONS

3.1 Introduction

In this chapter the theoretical development of the elasto-plastic constitutive relations and coupled formulations for unsaturated soils are presented. The coupled formulations are based on elastic, plastic theory and Darcy's flow law.

3.2 Constitutive Relations for Unsaturated Soils

3.2.1 Soil Structure Constitutive Relations

3.2.1.1 Elasticity Form of Constitutive Relations

Equation (2.12) in Chapter 2 presents the soil structure constitutive relations for the case when stress and strain relationships are elastic. The change in net normal stresses can be expressed as a function of the change in normal strain and matric suction:

$$\begin{aligned} d(\sigma_x - u_a) &= \frac{E}{1+\mu} \left(d\varepsilon_x + \frac{\mu}{1-2\mu} d\varepsilon_v \right) - \frac{E}{H(1-2\mu)} d(u_a - u_w) \\ d(\sigma_y - u_a) &= \frac{E}{1+\mu} \left(d\varepsilon_y + \frac{\mu}{1-2\mu} d\varepsilon_v \right) - \frac{E}{H(1-2\mu)} d(u_a - u_w) \\ d(\sigma_z - u_a) &= \frac{E}{1+\mu} \left(d\varepsilon_z + \frac{\mu}{1-2\mu} d\varepsilon_v \right) - \frac{E}{H(1-2\mu)} d(u_a - u_w) \end{aligned} \quad (3.1)$$

where:

$d\varepsilon_v$ = volumetric strain change for each increment (dV_v/V_0);

$d\varepsilon_v$ = $d\varepsilon_x + d\varepsilon_y + d\varepsilon_z$;

dV_v = total volume change of a soil element;

V_0 = initial volume of a soil element.

The relationship between the change in net normal stress and matric suction can be written in a matrix form as follows:

$$d\boldsymbol{\sigma}'' = \mathbf{D}^e d\boldsymbol{\varepsilon} - \mathbf{h}^e ds \quad (3.2)$$

where:

$$\boldsymbol{\varepsilon}^T = [\varepsilon_x \quad \varepsilon_y \quad \varepsilon_z \quad \gamma_{xy} \quad \gamma_{yz} \quad \gamma_{zx}]$$

$$\boldsymbol{\sigma}''^T = [\sigma_x - u_a \quad \sigma_y - u_a \quad \sigma_z - u_a \quad \tau_{xy} \quad \tau_{yz} \quad \tau_{zx}]$$

$$s = u_a - u_w$$

$$\mathbf{D}^e, \mathbf{h}^e = \text{elastic constitutive matrices;}$$

$$D_{11}=D_{22}=D_{33} = \frac{E(1-\mu)}{(1+\mu)(1-2\mu)}$$

$$D_{12}=D_{13}=D_{21}=D_{23}=D_{31}=D_{32} = \frac{E\mu}{(1+\mu)(1-2\mu)}$$

$$D_{44}=D_{55}=D_{66} = \frac{E}{2(1+\mu)}$$

$$h_1=h_2=h_3 = \frac{E}{H(1-2\mu)}$$

3.2.1.2 Elasto-plastic Form of Constitutive Relations

In order to take into account features of soil behavior such as yield and irrecoverable strain, the elasto-plastic models are used. Almost all elasto-plastic models are based on same elemental principles, but use different yield criteria, flow rule, and compressibility functions.

The basic principle of elasto-plastic models is that strains and strain rate are divided into elastic part and plastic part (Hill, 1950).

Alonso et al., 1990; Josa et al., 1992; Wheeler and Sivakumar, 1995 used four state variables, which are net mean stress, p , deviator stress, q , matric suction, s , and specific volume, v to describe the behaviour of unsaturated soils. On the other hand,

Toll (1990), Wheeler (1996); Rampino et al. (2000); Wang et al. (2002) and Chiu and Ng (2003) indicated that specific water volume or degree of saturation should be taken into account in the elasto-plastic constitutive framework model. Alternatively, the behaviour of unsaturated soil can be described using five state variables:

$$p = \frac{\sigma_1 + \sigma_2 + \sigma_3}{3} - u_a \quad (3.3)$$

$$q = \sigma_1 - \sigma_3 \quad (3.4)$$

$$s = u_a - u_w \quad (3.5)$$

$$\nu = 1 + e \quad (3.6)$$

$$\nu_w = 1 + Se \quad (3.7)$$

where:

- p = net mean stress,
- $\sigma_1, \sigma_2, \sigma_3$ = total principal normal stresses,
- u_a = pore-air pressure,
- q = deviator stress,
- s = matric suction of the soil (i.e., $u_a - u_w$),
- u_w = pore-water pressure,
- ν = specific volume,
- e = void ratio,
- ν_w = specific water volume, and
- S = degree of saturation.

The total strain increment ($d\varepsilon$) is the sum of the elastic strain increment ($d\varepsilon^e$) and the plastic strain increment ($d\varepsilon^p$) (Hill, 1950).

$$d\varepsilon = d\varepsilon^e + d\varepsilon^p \quad (3.8)$$

where:

$d\varepsilon$ = total strain increment,

$d\varepsilon^e$ = elastic strain increment,

$d\varepsilon^p$ = plastic strain increment.

Volumetric strain increment and shear strain increment can be calculated as follows:

$$d\varepsilon_v = d\varepsilon_v^e + d\varepsilon_v^p \quad (3.9)$$

$$d\varepsilon_q = d\varepsilon_q^e + d\varepsilon_q^p \quad (3.10)$$

$d\varepsilon_v$ = volumetric strain increment,

$d\varepsilon_v^e$ = elastic volumetric strain increment,

$d\varepsilon_v^p$ = plastic volumetric strain increment,

$d\varepsilon_q$ = shear strain increment,

$d\varepsilon_q^e$ = elastic shear strain increment,

$d\varepsilon_q^p$ = plastic shear strain increment.

Elastic volumetric strain increment and plastic volumetric strain increment can be calculated as follows:

$$d\varepsilon_v^e = d\varepsilon_{v(p)}^e + d\varepsilon_{v(s)}^e \quad (3.11)$$

$$d\varepsilon_v^p = d\varepsilon_{v(p)}^p + d\varepsilon_{v(s)}^p \quad (3.12)$$

where:

$d\varepsilon_{v(p)}^e$ = elastic volumetric strain increment induced by changes in net mean stress;

$d\varepsilon_{v(s)}^e$ = elastic volumetric strain increment induced by changes in matric suction;

$d\varepsilon_{v(p)}^p$ = plastic volumetric strain increment induced by changes in net mean stress;

$d\varepsilon_{v(s)}^p$ = plastic volumetric strain increment induced by changes in matric suction;

Elastic strains

Any stress change in the elastic region causes only elastic strain. The elastic strains are recoverable.

The idealized variations of specific volume with respect to net mean stress, p , at different matric suctions are shown in **Figure 3.1**. The elastic volumetric strain increment due to the change in net mean stress can be calculated using Equation (3.13) (Alonso et al. 1990):

$$d\varepsilon_{v(p)}^e = \frac{\kappa}{v} \frac{dp}{p} \quad (3.13)$$

where:

κ = elastic stiffness parameter for changes in net mean stress,

dp = increment of net mean stress.

In addition, the elastic volumetric strain can be calculated using Equation (3.14) (Alonso et al. 1990):

$$d\varepsilon_{v(p)}^e = \frac{dp}{K} \quad (3.14)$$

where:

K = bulk modulus, $\left(K = \frac{vp}{\kappa} \right)$

Figure 3.1 Idealized variation of specific volume with net mean stress and suction along virgin and loading-unloading paths

Figure 3.2 Idealized relationships between specific volume and matric suction at different net mean stresses of p

The elastic shear strain increment can be calculated using Equation (3.15) (Alonso et al. 1990):

$$d\varepsilon_q^e = \frac{dq}{3G} \quad (3.15)$$

where:

G = shear modulus;

dq = increment of deviator stress.

The relationship between bulk modulus K and shear modulus can be expressed as follows:

$$G = \frac{3}{2} K \frac{1-2\mu}{1+\mu} \quad (3.16)$$

The idealized variation between specific volume, v , and matric suction at different net mean stresses are shown in **Figure 3.2**. The elastic volumetric strain increment due to change in matric suction can be calculated using Equation (3.17) (Alonso et al. 1990):

$$d\varepsilon_{v(s)}^e = \frac{\kappa_s}{v} \frac{ds}{s + p_{at}} \quad (3.17)$$

where:

κ_s = elastic stiffness parameter for changes in suction;

p_{at} = atmospheric pressure.

Plastic strains

An increment in stress from a stress located at any yield surface will cause the soil to yield. The total volumetric strain can be calculated using Equation (3.18) (Alonso, et al. 1990):

$$d\varepsilon_{v(p)} = \frac{\lambda(s)}{v} \frac{dp}{p} \quad (3.18)$$

where:

$\lambda(s)$ = stiffness parameter for changes in net mean stress for a virgin state of soil under a given matric suction.

The plastic volumetric strain caused by the changes in net mean stress can be calculated using Equation (3.19) (Alonso et al. 1990):

$$d\varepsilon_{v(p)}^p = \frac{\lambda(s) - \kappa}{\nu} \frac{dp}{p} \quad (3.19)$$

When the matric suction reaches the yield suction, the increment in matric suction causes total volumetric strain that can be calculated using Equation (3.20) (Alonso et al. 1990):

$$d\varepsilon_{v(s)} = \frac{\lambda_s}{\nu} \frac{ds}{s + p_{at}} \quad (3.20)$$

where:

λ_s = stiffness parameter for changes in matric suction for virgin state of the soil.

ds = increment of matric suction.

The plastic strain caused by the increment of matric suction can be calculated using Equation (3.21) (Alonso et al. 1990):

$$d\varepsilon_{v(s)}^p = \frac{\lambda_s - \kappa_s}{\nu} \frac{ds}{s + p_{at}} \quad (3.21)$$

Loading-collapse (LC) yield curve

Consider a variation between specific volume, ν , and net mean stress, p , at a particular matric suction (i.e., an isotropic consolidation test), when a yield stress is exceeded, the soil state falls on an isotropic normal compression line (i.e., virgin line). The specific volume, ν , can be calculated using Equation (3.22) (Wheeler and Sivakumar, 1995):

$$\nu = N(s) - \lambda(s) \ln \left(\frac{p}{p_{at}} \right) \quad (3.22)$$

where

$N(s)$ = specific volume of the normal consolidation line under a given matric suction at atmospheric pressure.

The equation of LC yield curve can be derived based on the elastic behaviour inside the yield surface.

Consider a stress state moving along the yield curve under an isotropic stress state (see **Figure 3.3**), from point A at zero suction and net mean stress $p_o(0)$ to the new position, point C at suction s and net mean stress p_o . In the case, the stress path moves from A to C by following the elastic stress path ABC and the change in specific volume Δv can be calculated based on two elastic indexes κ and κ_s using Equation (3.23) (Alonso et al. 1990):

$$\Delta v = -\kappa_s \ln \left(\frac{s + p_{at}}{p_{at}} \right) - \kappa \ln \left(\frac{p_o}{p_o(0)} \right) \quad (3.23)$$

Point A and point C not only lie on the yield curve, but they also lie on the corresponding isotropic normal compression line for an appropriate value of suction. Therefore, based on Equation (3.22) an alternative expression for the change in specific volume between point A and point C can be calculated using Equation (3.24) (Wheeler and Sivakumar 1995):

$$\Delta v = N(s) - \lambda(s) \ln \left(\frac{p_o}{p_{at}} \right) - N(0) - \lambda(0) \ln \left(\frac{p_o(0)}{p_{at}} \right) \quad (3.24)$$

The LC yield curve equation can be obtained by elimination of Δv from Equations (3.23) and (3.24)

$$\begin{aligned} (\lambda(s) - \kappa) \ln \left(\frac{p_o}{p_{at}} \right) &= (\lambda(0) - \kappa) \ln \left(\frac{p_o(0)}{p_{at}} \right) + N(s) - N(0) \\ &\quad + \kappa_s \ln \left(\frac{s + p_{at}}{p_{at}} \right) \end{aligned} \quad (3.25)$$

Suction increase (SI) yield curve

50

between specific volume and matric suction (**Figure 3.2**) can be expressed as follows:

$$\nu = N_s(p) - \lambda_s(p) \ln \left(\frac{s}{p_{at}} \right) \quad (3.26)$$

where

$N_s(p)$ = specific volume under a given net mean stress at matric suction equal to atmospheric pressure.

The equation of the SI yield curve can be derived based on the elastic behaviour inside the yield surface.

Consider a stress state moving along the yield curve under an isotropic stress state (see **Figure 3.4**), from point A at net mean stress p and matric suction $s_o(p)$ to a new position, point C at suction s_o and net mean stress $p=0$. The stress path from A to C following the elastic stress path ABC and the change in specific volume $\Delta \nu$ can be calculated based on two elastic indexes κ and κ_s as follows:

$$\Delta \nu = -\kappa_s \ln \left(\frac{s_o(p)}{s_o(0)} \right) - \kappa \ln \left(\frac{p + p_{at}}{p_{at}} \right) \quad (3.27)$$

p_{at} has been added to p to avoid infinite values when p approaches zero.

Point A and point C not only lie on the yield curve, but they also lie on the corresponding virgin line that relates the specific volume to matric suction for an appropriate value of net mean stress. Therefore, based on Equation (3.26) an alternative expression for the change in specific volume between point A and point C can also be calculated as follows:

$$\Delta \nu = N_s(p) - \lambda_s(p) \ln \left(\frac{s_o(p)}{p_{at}} \right) - N_s(0) + \lambda_s(0) \ln \left(\frac{s_o(0)}{p_{at}} \right) \quad (3.28)$$

(a) Variation of specific volume with respect to matric suction at net mean stress of zero kPa and p kPa

(b) Stress path and yield curve in $(p-s)$ plane

Figure 3.4 Derivation of SI yield curve equation

The SI yield curve equation can be obtained by equating Equation (3.27) to Equation (3.28)

$$\begin{aligned}
(\lambda_s(p) - \kappa_s) \ln\left(\frac{s_o(p)}{p_{at}}\right) &= (\lambda_s(0) - \kappa_s) \ln\left(\frac{s_o(0)}{p_{at}}\right) + N_s(p) \\
&\quad - N_s(0) + \kappa \ln\left(\frac{p + p_{at}}{p_{at}}\right)
\end{aligned} \tag{3.29}$$

Yield curve in (p-s) plane

The LC yield equation (i.e., Equation (3.25)) shows that the relationship between yield stress, p_o , and matric suction, s , depends on the variation of $N(s)$, $\lambda(s)$, κ and κ_s with respect to matric suction. The variation of $N(s)$, $\lambda(s)$, κ and κ_s with respect to matric suction as obtained from literature are shown in **Figure 3.5**, **Figure 3.6** and **Figure 3.7**, respectively. **Figure 3.6** and **Figure 3.7** show that the $\lambda(s)$ and $\kappa(s)$ decrease to a constant value with the increase in matric suction. Meanwhile, Figure 2.6 in Chapter 2 and **Figure 3.5** show that the $N(s)$ value continues to decrease with the increase in matric suction. As a result, the yield stress, p_o , increases first and then decreases.

Figure 3.5 Variation of $N(s)$ with respect to matric suction (from Thu et al. 2007)

Figure 3.6 Variation of $\lambda(s)$ with respect to matric suction (from Alonso et al. 1990, Rampino et al. 2000 and Thu et al. 2007)

Figure 3.7 Variation of $\kappa(s)$ with respect to matric suction (from Alonso et al. 1990, Rampino et al. 2000 and Thu et al. 2007)

Figure 2.6 in Chapter 2 also shows that $N_s(p)$ continues to decrease with the increase in net mean stress. The soil specimen for a SWCC test under high net mean stress is denser than the soil specimen for a SWCC test under lower net mean stress.

As a result, the values of $\lambda_s(p)$ and $\kappa_s(p)$ could decrease with the increase in net mean stress. From the SI yield curve equation (i.e., Equation (3.29)) and the variation of $N_s(p)$, $\lambda_s(p)$ and $\kappa_s(p)$ with respect to net mean stress, the variation of yield suction, s_o , with respect to net mean stress should be the same as the variation of yield stress, p_o , with respect to matric suction (i.e., increases first and then decreases).

The yield curve on (p - s) plane is a combination of LC yield curve and SI yield curve. **Figure 3.8** shows the sketch of the yield curve on (p - s) plane.

Figure 3.8 LC and SI yield curves

Hardening parameter

The evolution of yield surface is controlled by the hardening parameters, p_o and s_o (Alonso et al., 1990). The hardening parameters can be calculated based on the total plastic volumetric strain as follows:

$$\frac{dp_o}{p_o} = \frac{vd\varepsilon_v^p}{\lambda(0) - \kappa(0)} \quad (3.30)$$

$$\frac{ds_o}{s_o} = \frac{vd\varepsilon_v^p}{\lambda_s(0) - \kappa_s(0)} \quad (3.31)$$

where:

- p_o = yield stress of saturated soil,
- s_0 = matric suction at yield point,
- $d\varepsilon_v^p$ = increment of plastic volume strain,
- $\lambda(0)$ = stiffness parameter for changes in net mean stress for a virgin state of soil under zero matric suction,
- $\lambda_s(0)$ = stiffness parameter for changes in matric suction for virgin state of the soil under zero net confining stress,
- $\kappa(0)$ = elastic stiffness parameter for change in net mean stress under zero matric suction.
- $\kappa_s(0)$ = elastic stiffness parameter for change in matric suction under zero net mean stress.

The isotropic yield stress, p_o , can be calculated from the current value of net mean stress, p , and specific volume, v (Sivakumar, 1993). The elastic parameter, $\kappa(s)$, is made to be a function of matric suction instead of independent of matric suction as in Sivakumar (1993) as follows:

$$\ln p_o = \frac{N(s) - v - \kappa(s) \ln p + \lambda(s) \ln p_{at}}{\lambda(s) - \kappa(s)} \quad (3.32)$$

Yield surface in the (q-p-s) space

The stress path in p-s plane (**Figure 3.9**) can reach SI yield curve such as OCD or reach LC yield curve such as OAB. Consider stress path OAB where this stress path reaches LC yield curve at B. The yield curve in q-OAB plane can be assumed to be ellipse and can be expressed as follows:

$$q^2 = M_*^2 \sqrt{p^2 + s^2} \left(\sqrt{\left(p_o(s_x) \right)^2 + s_x^2} - \sqrt{p^2 + s^2} \right) \quad (3.33)$$

The yield curve in q-OCD plane can be assumed to be ellipse and can be expressed as follows:

$$q^2 = M_*^2 \sqrt{p^2 + s^2} \left(\sqrt{p_x^2 + (s_o(p_x))^2} - \sqrt{p^2 + s^2} \right) \quad (3.34)$$

where:

M_* = ratio between vertical diameter and horizontal diameter of the elliptical yield curve,

$p_x, s_o(p_x)$ = stress state locate on SI yield curve (i.e., point D in **Figure 3.9**),

$p_o(s_x), s_x$ = stress state locate on LC yield curve (i.e., point B in **Figure 3.9**).

The ratio between vertical diameter and horizontal diameter of the elliptical yield curve (i.e., M_*) is a function of net mean stress and matric suction. The value of M_* satisfies that the apex of the ellipse coincides with the failure envelope. Figure 3.10 shows the yield curve in the $s/p = \text{constant}$ plane.

Figure 3.9 The LC, SI and triaxial stress state yield curve

Figure 3.10 Yield curve in the $s/p = \text{constant}$ plane

Flow rule

Gens and Potts (1982) pointed out that the conventional critical state models often overestimated K_o value. In order to avoid the overestimation of the K_o value, the associated flow rule was modified by introducing an α parameter (Ohmaki, 1982). The non-associated flow rule can be expressed as follows:

$$G^{LC} = \alpha q^2 - M_*^2 \sqrt{p^2 + s^2} \left(\sqrt{(p_o(s_x))^2 + s_x^2} - \sqrt{p^2 + s^2} \right) = 0 \quad (3.35)$$

$$G^{SI} = \alpha q^2 - M_*^2 \sqrt{p^2 + s^2} \left(\sqrt{p_x^2 + (s_o(p_x))^2} - \sqrt{p^2 + s^2} \right) = 0 \quad (3.36)$$

where:

α = soil parameter.

The soil parameter can be calculated as follows (Alonso et al., 1990):

$$\alpha = \frac{M(M-9)(M-3)}{9(6-M)} \frac{1}{1 - \frac{\kappa(0)}{\lambda(0)}} \quad (3.37)$$

3.2.1.3 Incremental Stress-Strain Relations

The LC and SI yield surfaces that are presented in Equation (3.33) and Equation (3.34) can be rewritten as follows:

$$F^{LC} = q^2 - M_*^2 \sqrt{p^2 + s^2} \left(\sqrt{(p_o(s_x))^2 + s_x^2} - \sqrt{p^2 + s^2} \right) = 0 \quad (3.38)$$

$$F^{SI} = q^2 - M_*^2 \sqrt{p^2 + s^2} \left(\sqrt{p_x^2 + (s_o(p_x))^2} - \sqrt{p^2 + s^2} \right) = 0 \quad (3.39)$$

The plastic potential functions are presented in Equation (3.35) and Equation (3.36). The hardening laws are presented in Equation (3.30) and Equation (3.31).

Plastic strain increment can be calculated based on flow rules as follows:

$$d\boldsymbol{\varepsilon}^p = \left(\frac{\partial G^{LC}}{\partial \boldsymbol{\sigma}} + \frac{\partial G^{LC}}{\partial s} \right) d\lambda^{LC} + \left(\frac{\partial G^{SI}}{\partial \boldsymbol{\sigma}} + \frac{\partial G^{SI}}{\partial s} \right) d\lambda^{SI} \quad (3.40)$$

In order to ensure the stress state to remain on the yield surfaces, the consistency condition can be written as follows:

$$\frac{\partial F^{LC}}{\partial \boldsymbol{\sigma}} d\boldsymbol{\sigma} + \frac{\partial F^{LC}}{\partial s} ds + \frac{\partial F^{LC}}{\partial p_o} \frac{\partial p_o}{\partial p_o(0)} dp_o(0) + \frac{\partial F^{LC}}{\partial p_o} \frac{\partial p_o}{\partial s} ds + \frac{\partial F^{LC}}{\partial p_o} \frac{\partial p_o}{\partial \varepsilon_v^p} d\varepsilon_v^p = 0 \quad (3.41)$$

$$\frac{\partial F^{SI}}{\partial \boldsymbol{\sigma}} d\boldsymbol{\sigma} + \frac{\partial F^{SI}}{\partial s} ds + \frac{\partial F^{SI}}{\partial s_0} \frac{\partial s_0}{\partial s_0(0)} ds_0(0) + \frac{\partial F^{SI}}{\partial s_0} \frac{\partial s_0}{\partial p} dp + \frac{\partial F^{SI}}{\partial s_0} \frac{\partial s_0}{\partial \varepsilon_v^p} d\varepsilon_v^p = 0 \quad (3.42)$$

Elastic strain increment can be calculated based on Equation (3.8) and Equation (3.40) as follows:

$$d\boldsymbol{\varepsilon}^e = d\boldsymbol{\varepsilon} - \left(\frac{\partial G^{LC}}{\partial \boldsymbol{\sigma}} + \frac{\partial G^{LC}}{\partial s} \right) d\lambda^{LC} - \left(\frac{\partial G^{SI}}{\partial \boldsymbol{\sigma}} + \frac{\partial G^{SI}}{\partial s} \right) d\lambda^{SI} \quad (3.43)$$

Incremental stress-strain relations can be calculated as follows:

$$d\boldsymbol{\sigma} = \mathbf{D}^e d\boldsymbol{\varepsilon}^e - \mathbf{h}^e ds \quad (3.44)$$

Substituting Equation (3.43) into Equation (3.44) gives,

$$d\boldsymbol{\sigma} = \mathbf{D}^e \left(d\boldsymbol{\varepsilon} - \left(\frac{\partial G^{LC}}{\partial \boldsymbol{\sigma}} + \frac{\partial G^{LC}}{\partial s} \right) d\lambda^{LC} - \left(\frac{\partial G^{SI}}{\partial \boldsymbol{\sigma}} + \frac{\partial G^{SI}}{\partial s} \right) d\lambda^{SI} \right) - \mathbf{h}^e ds \quad (3.45)$$

Substituting Equation (3.45) into Equation (3.41) and Equation (3.42) yield the following equations:

$$\begin{aligned} \frac{\partial F^{LC}}{\partial \boldsymbol{\sigma}} \left[\mathbf{D}^e \left(d\boldsymbol{\varepsilon} - \left(\frac{\partial G^{LC}}{\partial \boldsymbol{\sigma}} + \frac{\partial G^{LC}}{\partial s} \right) d\lambda^{LC} - \left(\frac{\partial G^{SI}}{\partial \boldsymbol{\sigma}} + \frac{\partial G^{SI}}{\partial s} \right) d\lambda^{SI} \right) - \mathbf{h}^e ds \right] \\ + \frac{\partial F^{LC}}{\partial s} ds + \frac{\partial F^{LC}}{\partial p_o} \frac{\partial p_o}{\partial p_o(0)} dp_o(0) + \frac{\partial F^{LC}}{\partial p_o} \frac{\partial p_o}{\partial s} ds + \frac{\partial F^{LC}}{\partial p_o} \frac{\partial p_o}{\partial \varepsilon_v^p} d\varepsilon_v^p = 0 \end{aligned} \quad (3.46)$$

$$\begin{aligned} \frac{\partial F^{SI}}{\partial \boldsymbol{\sigma}} \left[\mathbf{D}^e \left(d\boldsymbol{\varepsilon} - \left(\frac{\partial G^{LC}}{\partial \boldsymbol{\sigma}} + \frac{\partial G^{LC}}{\partial s} \right) d\lambda^{LC} - \left(\frac{\partial G^{SI}}{\partial \boldsymbol{\sigma}} + \frac{\partial G^{SI}}{\partial s} \right) d\lambda^{SI} \right) - \mathbf{h}^e ds \right] \\ + \frac{\partial F^{SI}}{\partial s} ds + \frac{\partial F^{SI}}{\partial s_0} \frac{\partial s_0}{\partial s_0(0)} ds_0(0) + \frac{\partial F^{SI}}{\partial s_0} \frac{\partial s_0}{\partial p} dp + \frac{\partial F^{SI}}{\partial s_0} \frac{\partial s_0}{\partial \varepsilon_v^p} d\varepsilon_v^p = 0 \end{aligned} \quad (3.47)$$

For the case when the stress state is located on both LC and SI yield surfaces, Equations (3.46) and (3.47) can be solved to obtain $d\lambda^{LC}$ and $d\lambda^{SI}$.

For the case when the stress state is located on LC yield surface $d\lambda^{SI} = 0$, Equation (3.46) can be solved to obtain $d\lambda^{LC}$.

For the case when the stress state is located on SI yield surfaces $d\lambda^{LC} = 0$, Equation (3.47) can be solved to obtain $d\lambda^{SI}$.

Substitute $d\lambda^{LC}$ and $d\lambda^{SI}$ into Equation (3.45) yields the incremental stress-strain relations.

$$d\boldsymbol{\sigma} = \mathbf{D}^{ep} d\boldsymbol{\varepsilon} - \mathbf{h}^{ep} ds \quad (3.48)$$

3.2.2 Water Phase Constitutive Relations

3.2.2.1 Elasticity Form of Water Phase Constitutive Relations

Equation (2.12) in Chapter 2 presents the water phase constitutive relationships for the case when stress and water volumetric strain relationships are elastic.

3.2.2.2 Elasto-plastic Form of Water Phase Constitutive Relations

The relationship between specific water volume, v_w , and net mean stress at a certain matric suction proposed by Wheeler (1996) and modified by Thu (2005) is adopted. The idealized relationship between specific water volume and net mean stress is presented in **Figure 3.11**.

Figure 3.11 Relationship between specific water volume and net mean stress at a constant matric suction (modified after Wheeler 1996, Thu et al. 2007)

The change in specific water volume with respect to the change in net mean stress can be calculated as follows:

$$dv_w = -(\lambda_w(s) - \kappa_w(s)) \frac{dp_o}{p_o} - \kappa_w(s) \frac{dp}{p} \quad (3.49)$$

where:

$\kappa_w(s)$ = elastic stiffness parameter with respect to water phase from isotropic consolidation test under a given matric suction,

$\lambda_w(s)$ = plastic stiffness parameter with respect to water phase from isotropic consolidation test under a given matric suction.

The relationship between volumetric water content, θ_w , or degree of saturation, S , and matric suction is commonly referred to as the soil-water characteristic curve (SWCC) (see **Figure 3.12a**). The relationship between volumetric water content and logarithmic matric suction can be assumed as a piecewise linear function.

$$dS = \lambda_{swcc} \frac{ds}{s} \quad (3.50)$$

where:

λ_{swcc} = slope of the SWCC.

(a) typical SWCC

(b) idealized SWCC (solid lines represent current SWCC)

Figure 3.12 Relationship between degree of saturation and suction, (a) typical SWCC; (b) idealized SWCC (solid lines represent current SWCC) (Modified after Sheng et al. 2008)

When matric suction is increased from zero until saturation suction, s_{sa} degree of saturation remains constant (**Figure 3.12a**). The degree of saturation decreases rapidly when matric suction is increased from the air-entry value, s_{ae} , until it reaches the residual soil suction, s_{re} . The slope λ_{swcc} is assumed to be constant between the air-entry value and the residual suction. When matric suction is increased from the residual suction, s_{re} , to 1 GPa, the degree of saturation decreases from the residual degree of saturation to zero (Fredlund and Rahardjo, 1993). In order to take into account the hysteresis, drying and wetting soil-water characteristic curves are considered. For matric suctions greater than the water-entry value and lower than the saturation suction, wetting curve and drying curve coincide with each other. Slope of the wetting curve for matric suction from the water-entry value, s_{we} , to the saturation suction is the same as the slope of the drying curve from the air-entry value to the residual suction. The changes from wetting to drying and vice versa are represented by scanning curves.

A typical SWCC (**Figure 3.12a**) can be simplified as an idealized SWCC (**Figure 3.12b**). Slope of the scanning curves, slope of the drying curve from the saturation suction to the air-entry value and slope of the wetting curve for matric suctions higher than the water-entry value are considered equal. This simplified SWCC is similar to the model presented by Wheeler et al. (2003) and Sheng et al. (2008).

An unsaturated state always lies within the main drying and wetting curves. Drying or wetting within the hysteresis loop follows the scanning curve until matric suction reaches the main drying or the main wetting curve. Once soil suction reaches the main drying at s_D suction, or the main wetting curve at s_W , further drying or wetting will follow the main drying or the main wetting curve, respectively. The main wetting curve for suction from zero to s_W suction, the scanning curve for suction from s_W to s_D , and the main drying curve for suction from s_D suction to 1 GPa can be called “current” SWCC (i.e. the ABCDEF in **Figure 3.12b**). The slope of the “current” SWCC is determined as follows:

$$\lambda_{swcc} = \begin{cases} 0 & s < s_{sa} \\ \lambda_{ws} & s_{sa} \leq s < s_W \\ \kappa_{ws} & s_W \leq s < s_D \\ \lambda_{ws} & s_D \leq s < s_{re} \\ \kappa_{ws} & s \geq s_{re} \end{cases} \quad (3.51)$$

The variation of specific water content with respect to matric suction can be obtained based on Equation (3.7) as follows:

$$dv_w = (\lambda_{swcc}e + \kappa_s S) \frac{ds}{s} + \lambda_s S \frac{ds_o}{s_o} \quad (3.52)$$

Based on Equation (3.49) and Equation (3.52), the change in specific water volume with respect to net mean stress and matric suction can be expressed as follows:

$$dv_w = -(\lambda_w(s) - \kappa_w(s)) \frac{dp_o}{p_o} - \kappa_w(s) \frac{dp}{p} + (\lambda_{swcc}e + \kappa_s S) \frac{ds}{s} + \lambda_s S \frac{ds_o}{s_o} \quad (3.53)$$

3.3 Coupling between Equilibrium Problem and Seepage Problem

3.3.1 Equilibrium Equations of Soil Structure

The following static stress equilibrium equations can be used for an unsaturated soil element (e.g., Fredlund and Rahardjo, 1993):

$$\begin{aligned} \frac{\partial \sigma_x}{\partial x} + \frac{\partial \tau_{xy}}{\partial y} + \frac{\partial \tau_{xz}}{\partial z} + F_x &= 0 \\ \frac{\partial \tau_{xy}}{\partial x} + \frac{\partial \sigma_y}{\partial y} + \frac{\partial \tau_{yz}}{\partial z} + F_y &= 0 \\ \frac{\partial \tau_{xz}}{\partial x} + \frac{\partial \tau_{yz}}{\partial y} + \frac{\partial \sigma_z}{\partial z} + F_z &= 0 \end{aligned} \quad (3.54)$$

where:

F_i : body force acting on the i direction.

Figure 3.13 Normal stress and shear stress acting on a cubical soil element of infinitesimal dimensions (after Fredlund and Rahardjo, 1993)

The relationship between normal, shear strain and displacement in the x-, y-, and z-direction are expressed as follows (Timoshenko and Goodier, 1970):

$$\begin{aligned} \epsilon_x &= \frac{\partial u}{\partial x}, \quad \epsilon_y = \frac{\partial v}{\partial y}, \quad \epsilon_z = \frac{\partial w}{\partial z} \\ \gamma_{xy} &= \frac{\partial u}{\partial y} + \frac{\partial v}{\partial x}, \quad \gamma_{yz} = \frac{\partial v}{\partial z} + \frac{\partial w}{\partial y}, \quad \gamma_{zx} = \frac{\partial w}{\partial x} + \frac{\partial v}{\partial z} \end{aligned} \quad (3.55)$$

where:

ϵ_i = normal strain in the i -direction;

γ_{ij} = shear strain with respect to the i and j reference axes;

u, v, w = displacement in the x-, y-, and z-direction, respectively.

Substituting the stress-strain relationships (i.e., Equation (3.2)), and the strain-displacement relationships (i.e., Equation (3.55)) into the static stress equilibrium equations (i.e., Equation (3.54)) yields:

$$\begin{aligned}
 & \frac{\partial}{\partial x} \left[D_{11} \frac{\partial u}{\partial x} + D_{12} \frac{\partial v}{\partial y} + D_{13} \frac{\partial w}{\partial z} \right] + \frac{\partial}{\partial y} \left[D_{44} \left(\frac{\partial u}{\partial y} + \frac{\partial v}{\partial x} \right) \right] \\
 & \quad + \frac{\partial}{\partial z} \left[D_{66} \left(\frac{\partial u}{\partial z} + \frac{\partial w}{\partial x} \right) \right] - \frac{\partial}{\partial x} [h_1 (u_a - u_w)] + \frac{\partial u_a}{\partial x} = 0 \\
 & \frac{\partial}{\partial x} \left[D_{44} \left(\frac{\partial u}{\partial y} + \frac{\partial v}{\partial x} \right) \right] + \frac{\partial}{\partial y} \left[D_{21} \frac{\partial u}{\partial x} + D_{22} \frac{\partial v}{\partial y} + D_{23} \frac{\partial w}{\partial z} \right] \\
 & \quad + \frac{\partial}{\partial z} \left[D_{55} \left(\frac{\partial v}{\partial z} + \frac{\partial w}{\partial y} \right) \right] - \frac{\partial}{\partial y} [h_2 (u_a - u_w)] + \frac{\partial u_a}{\partial y} = -\gamma_{nat} \\
 & \frac{\partial}{\partial x} \left[D_{66} \left(\frac{\partial u}{\partial z} + \frac{\partial w}{\partial x} \right) \right] + \frac{\partial}{\partial y} \left[D_{55} \left(\frac{\partial v}{\partial z} + \frac{\partial w}{\partial y} \right) \right] \\
 & \quad + \frac{\partial}{\partial z} \left[D_{31} \frac{\partial u}{\partial x} + D_{32} \frac{\partial v}{\partial y} + D_{33} \frac{\partial w}{\partial z} \right] - \frac{\partial}{\partial z} [h_3 (u_a - u_w)] + \frac{\partial u_a}{\partial z} = 0
 \end{aligned} \tag{3.56}$$

Rearranging Equation (3.56) yields:

$$\begin{aligned}
 & G \nabla^2 u + \frac{G}{1-2\mu} \frac{\partial \varepsilon_v}{\partial x} - \frac{E}{H(1-2\mu)} \frac{\partial (u_a - u_w)}{\partial x} + \frac{\partial u_a}{\partial x} = 0 \\
 & G \nabla^2 v + \frac{G}{1-2\mu} \frac{\partial \varepsilon_v}{\partial y} - \frac{E}{H(1-2\mu)} \frac{\partial (u_a - u_w)}{\partial y} + \frac{\partial u_a}{\partial y} = -\gamma_{nat} \\
 & G \nabla^2 w + \frac{G}{1-2\mu} \frac{\partial \varepsilon_v}{\partial z} - \frac{E}{H(1-2\mu)} \frac{\partial (u_a - u_w)}{\partial z} + \frac{\partial u_a}{\partial z} = 0
 \end{aligned} \tag{3.57}$$

where:

$$\nabla^2 = \frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial y^2} + \frac{\partial^2}{\partial z^2} .$$

$$G = \frac{E}{2(1+\mu)}$$

The elastic stress-strain relations (i.e., Equation (3.2)) are used to derive Equation (3.57). Therefore, the solutions of Equation (3.57) are elastic solutions. In order to obtain elasto-plastic solution, Equation (3.48) is used instead of Equation (3.2).

(3.34)) are defined in radial direction. The incremental stress-strain relation is derived in Equation (3.48). The “current” SWCC is introduced in Section 3.2.2 and the variation of water phase constitutive equation is proposed as given in Equation (3.53). The coupled solution between soil structure equilibrium and seepage is also proposed as given in Equation (3.54), Equation (3.55), Equation (3.48), Equation (3.59) and Equation (3.53).

CHAPTER 4

EXPERIMENTAL PROGRAMME AND PREDICTION FROM THE PROPOSED MODEL

4.1 Introduction

This chapter mainly focuses on experimental procedures used to conduct basic soil property tests, soil-water characteristic curve (SWCC) tests, isotropic consolidation tests, consolidated drained (CD) triaxial tests and shearing-infiltration tests. The proposed model presented in Chapter 3 was used to predict the response of unsaturated soil specimen during shearing under consolidated drained and shearing-infiltration conditions (i.e., the two stress paths commonly occur in the field) using the results from isotropic consolidation tests and SWCC tests as input parameters (i.e., $N(s)$, $\lambda(s)$, $\kappa(s)$, $N_s(p)$, $\lambda_s(p)$ and $\kappa_s(p)$) and shear strength parameters. The comparisons between the predicted results and the measurement results from consolidated drained triaxial and shearing-infiltration tests provide the verifications of the proposed model presented in Chapter 3.

4.2 Outline of Research Programme

The research programme consisted of several main parts as follows:

1. Selection of appropriate soil used for the experimental programme and determination of basic properties of the selected soil.
2. Modification of triaxial apparatus in order to measure the pore-water pressure in soil specimen during SWCC tests, isotropic consolidation tests, consolidated drained triaxial tests and shearing-infiltration tests using the triaxial apparatus.
3. Using the modified triaxial apparatus to conduct a series of tests for obtaining soil-water characteristic curves under different net mean stresses on compacted specimens and a series of isotropic consolidation tests under different matric suctions on compacted specimens to obtain the input parameters (i.e., $N(s)$, $\lambda(s)$, $\kappa(s)$, $N_s(p)$, $\lambda_s(p)$ and $\kappa_s(p)$) for the proposed model presented in Chapter 3.

4. Conducting a series of consolidated drained triaxial tests to obtain shear strength parameters, total volume change and water volume change characteristics during shearing of an unsaturated soil under drained conditions. The shear strength and total volume change characteristics describe the behavior of unsaturated compacted soil in consolidated drained triaxial test that can be used to verify the proposed model presented in Chapter 3.
5. Conducting a series of shearing-infiltration tests to obtain shear strength, total volume change and matric suction change characteristics under shearing-infiltration conditions. The shear strength, total volume change and matric suction change characteristics describe the behavior of unsaturated compacted soil in shearing-infiltration test that can be used to verify the proposed model presented in Chapter 3.
6. Prediction of the shear strength, total volume change during shearing under consolidated drained condition; shear strength, total volume change and matric suction changes during shearing-infiltration condition using the theory developed in Chapter 3. The comparisons between results obtained from the proposed model and the measurement results from consolidated drained triaxial and shearing-infiltration tests provide the verifications of the proposed model presented in Chapter 3.

4.3 Basic Properties of Soil and Specimen Preparation

4.3.1 Criteria for Soil Selection and Specimen Preparation

In order to have suitable data from laboratory tests, soil specimens should meet the following conditions:

1. the soil particles should not be too large in order to avoid difficulties associated with specimen preparation, and poor contact between soil specimen and NTU mini suction probe.
2. the unsaturated water coefficient of permeability should not be too low at high matric suctions in order to have continuous air and water phases and reasonable testing time.

3. the soil specimens must be identical (i.e., uniform and homogenous in initial dry density and initial water content).
4. the volume change of the soil should not be too excessive during wetting process.

Based on the data from literature (Meilani et al., 2005; Thu et al., 2006; Thu et al., 2007), Kaolin, made by Kaolin Malaysia SDN BHD could satisfy the above requirements.

4.3.2 Basic Soil Properties and Compaction Method

The Atterberg limits, specific gravity, hydrometer were conducted to determine the index properties of the compacted Kaolin. Basic soil properties test were conducted in accordance with the ASTM standards. The standard compaction test was conducted according with ASTM No D698.

The specimen obtained from dynamic compaction has non-uniform densities along the length of the specimen. In order to obtain identical specimens with a uniform density along the specimen for the experimental programme, the static compaction method was used.

Identical statically compacted specimens with the same dry density of 1.35 Mg/m^3 and water content of 21.7% were used in the experimental programme.

4.3.3 Static Compaction Method

The static compaction method described by Ong (1999) was adopted. The static compaction apparatus consists of compaction mould and two stainless steel plugs. The compaction mould consists of two symmetrical parts that are connected together on two sides by screws as shown in **Figure 4.1a**. The outer dimension of the stainless steel compaction mould is 70mm x 70 mm x 200mm and the inner diameter is 50 mm. **Figure 4.1** shows the static compaction mould and stainless steel plugs.

The stainless steel plug consists of a number of removable disks. The thickness of each removable disk is 10 mm. The removable disks are connected together by a screw at the center of the disks. The disk is made in a T shape in order to reduce the friction between soil particles and the wall of the mould during compaction. The

diameter at the base of the disk is 50 mm, the diameter at the top part is 40 mm. The thickness of each part of the disk is 5 mm.

(a)

(b)

Figure 4.1 Static compaction mould and stainless steel plugs, (a) Stainless steel mould; (b) Stainless steel plugs.

(a)

(b)

(c)

Figure 4.2 Stainless steel plug, (a) Connection between two adjacent disks; (b) removable disk; (c) small plug (after Ong 1999)

In order to connect the disks together, each disk has one threaded screw protruding out from one face of the disk and one threaded hole on another face of this disk.

Figure 4.2 illustrates how two disks are connected together.

4.3.4 Static Compaction Process

The Kaolin mass and water mass were calculated to obtain the required dry density and water content. The specimen was statically compacted in 10 layers, each layer had a thickness of 10 mm. The height of the specimen was 100 mm. The diameter of the specimen was 50 mm. For the compaction of the first layer, ten disks were assembled and placed from the bottom into the compaction mould. The mass of the Kaolin mixture was put into the compaction mould from the top. The top plug consisted of 9 disks that were connected together. The top plug was placed on the top of the Kaolin mixture inside the compaction mould. The compaction mould with plug and Kaolin mixture were put into a triaxial frame. **Figure 4.3** shows the compaction machine for static compaction of the soil specimens.

Figure 4.3 Compaction machine for static compaction of specimens

The Kaolin mixture was compressed at a constant rate of 1 mm per minute in order to obtain a uniform density over the specimen and to avoid the raising up of excess pore-water pressure. The load was applied onto plugs until the soil layer was compressed to a thickness of 10 mm.

For the second layer, the mould was turned over and the top plug became the bottom one. One disk was removed from the top plug so the height of the top plug was reduced to 90 mm. The surface of the compacted soil was scratched in order to have good contact between adjacent layers. The Kaolin mixture for the second layer was then added to the mould. The top plug was put into the compaction mould and a load was applied onto the plugs in order to compress the soil at the same rate until a thickness of 20 mm was reached.

For the subsequent layers, the process consisting of turning over the compaction mould, removal one of the disks to reduce the length of the top plug, scratching the surface of the compacted soil and adding the next Kaolin mixture, was the same.

After the tenth layer was compacted, the soil specimen was extruded using the same compaction machine. Figure 4.4 shows the extrusion of a compacted Kaolin specimen.

Figure 4.4 Extrusion a of compacted Kaolin specimen.

4.4 Triaxial Setup and Its Development

In this study, the modified triaxial apparatus were used to obtain SWCC under different net confining stresses, the isotropic consolidation curve under different matric suctions, consolidated drained triaxial tests and to conduct the shearing-infiltration tests. The modified triaxial apparatus was similar to the modified triaxial apparatus described by Fredlund and Rahardjo (1993). This section describes the modified triaxial apparatus and test procedures for conducting SWCC tests, isotropic consolidation tests, CD tests and shearing-infiltration tests.

Figure 4.5 shows the layout of the modified triaxial apparatus for unsaturated soil testing. Both pore-air pressure, u_a , and pore-water pressure, u_w , could be controlled. NTU mini suction probes were installed along the specimen in order to measure the pore-water pressures during testing.

Figure 4.5 Modified triaxial apparatus for unsaturated soil testing (Modified from Fredlund and Rahardjo, 1993)

Figure 4.6 shows the schematic diagram for the plumbing of the modified triaxial apparatus. The confining pressure, pore-air and pore-water pressures were applied by opening valves D, A, and C, respectively. For the flushing process, valves B and C, were opened and the air bubbles accumulated in the water compartment were removed by the water pressure gradient.

Figure 4.6 The schematic diagram for the plumbing of the modified triaxial apparatus.

The modified triaxial apparatus for testing programme consisted of a triaxial compression cell; a force actuator for shearing and maintaining a constant axial force on soil specimen; a digital pressure and volume controller (DPVC-1) for cell pressure; a digital pressure and volume controller (DPVC-2) for back pressure; a linear variable differential transformer (LVDT); diffused-air volume indicator (DAVI); two pore pressure transducer; data acquisition unit and personal computer and two NTU mini suction probes for pore-water measurement.

The DPVC-1 controlled the cell pressure and recorded the cell volume change. The cell volume change was used to measure the volume change of the soil specimen during the test. The DPVC-2 controlled the water pressure at the base plate, u_w . The air pressure system controlled the air-pressure, u_a . All the pressure transducers and

NTU mini suction probes were calibrated before the tests started and for every 6-month period during the testing programme.

In order to account for the effect of pressure change and creep of the Perspex cell, the cell volume change, a correction had been applied. The correction was determined by running a test using a dummy sample (i.e., stainless steel). The dummy sample has the same size with that of soil specimen and it can be considered of having no volume change. The results from this test gave the correction for volume change measurement of soil specimen.

4.4.1 Triaxial Cell

In order to have enough space for two NTU mini probes to fit to the soil specimen, a large triaxial cell was used. The triaxial cell dimension is 380 mm in height, 201 mm in diameter and 16 mm in thickness. The capacity of the cell pressure is 1700 kPa.

4.4.2 Special Adapters at Triaxial Base

In order to test unsaturated soil specimens, the conventional triaxial apparatus requires modifications. To control the pore-water pressure, a high air-entry ceramic disk of 6.35 mm in thickness and 50 mm in diameter was used. This ceramic disk was connected to the triaxial base using a pedestal head with a water compartment. The ceramic disk was glued onto the water compartment by applying the slow setting epoxy resin around its circumference. The pedestal base has two ports, which were connected to two small holes in the water compartment. The ports were connected to valves. One port was connected to a pressure transducer for the measurement of pore-water pressure, the other port was connected to the DPVC-2 for controlling the pore-water pressure and measuring the water volume change during testing progress.

An extension ring with six ports was installed on the triaxial base. Cables for the two NTU mini probes were passed through the ports on the extension ring. In order to prevent water leakage, O-ring was used. **Figure 4.8** shows an extension ring and a typical wire passing through the extension ring

Figure 4.7 Water compartment in the pedestal with the high air entry disk (not installed)

Figure 4.8 A typical wire passing through the extension ring

4.4.3 Pore-water Pressure Control System

When water pressure decreases below -100 kPa, water will cavitate. In this case, the water compartment will be partially filled with air. As a result, wrong measurement of pore-water pressure will take place. In order to avoid cavitation in the measurements system, the axis-translation technique proposed by Hilf (1956) was adopted. The pore-air and pore-water pressures were controlled to obtain the desired matric suction. The applied air and water pressures were greater than zero and the cavitation would not occur.

A soil specimen was placed on top of the high air-entry disk in order to have continuity between the pore-water pressure in the soil specimen and the water pressure in the controlling system. The pore-water pressure in the soil specimen could then be controlled by the water pressure in the water compartment through DVPC-2. The water pressure in the water compartment was measured by a pressure transducer and this pressure transducer was connected to the water compartment at the base plate.

4.4.4 Pore-air Pressure Control System

On the top of the specimen, a filter paper, a coarse corundum disk and a perspex loading cap were placed in this order. A small hole was drilled in the loading cap and it was connected to a port. The port was connected to an air pressure line. The air pressure line was in contact with the soil specimen through the coarse corundum disk and filter paper. The coarse corundum disk has a very low air-entry value. Air will pass through the disk when the air pressure exceeds the air-entry value of the disk. So, the continuity between air void in the specimen and the air pressure control system could be established. The pore-air pressure can be controlled independently.

4.4.5 Flushing System

Air bubbles could be trapped in the water compartment during tests. In order to remove these air bubbles, a flushing system was required. Air bubble was resulted from air diffusing through water in the ceramic disk and accumulated in the water compartment. These air bubbles could impede the flow of water from and into the specimen and cause incorrect measurements of pore-water pressure and volume

change (Ho and Fredlund, 1982). The flushing system consists of an extra pore-water drainage line with a control valve connected to the base of the triaxial apparatus. When flushing, the control valve was opened over short period to allow the accumulated air bubbles to be flushed out through the additional drainage line.

4.4.6 Measurement of Matric Suction in the Soil Specimen

The NTU mini suction probe developed by Meilani et al. (2002) was used to measure pore-water pressure in the soil specimen. The NTU mini probe was developed from a pore-water pressure transducer PDCR 81, manufactured by Druck Ltd. The original ceramic disk of the PDCR 81 was removed and replaced with a new 5 bar high air entry ceramic disk.

Figure 4.9 NTU mini probe

In order to capture the pore-water pressure changes in the soil specimen during the tests, two NTU mini probes were installed along the soil specimen. The probes were installed at $\frac{1}{4}$ (i.e. “top”) and $\frac{3}{4}$ (i.e. “bottom”) heights of the specimen from its base and at opposite sides in the lateral direction.

Figure 4.10 Installation details of NTU mini suction probes (after Meilani et al. 2002)

The installation details of NTU mini suction probes are shown in **Figure 4.10**. In order to hold NTU mini probes and to prevent cell water from flowing into soil specimen, silicon rubber grommets were used. The internal diameter of the grommet was equal to the external diameter of NTU mini probe. Two O-rings were assembled to provide more protection from leakage. **Figure 4.11** shows details of NTU mini suction probe on specimen.

Figure 4.11 Details of NTU mini suction probe on specimen (after Meilani et al. 2002)

In order to place a soil specimen into a modified rubber membrane (i.e., the silicon rubber grommets were installed through the rubber membrane), a two-part split membrane stretcher was used. The membrane stretcher could be removed from the specimen. The original membrane stretcher was designed by Wong (2000) and later modified by Meilani et al. (2002). Two parts of the membrane stretcher with two rubber holders at different heights (i.e. $\frac{1}{4}$ and $\frac{3}{4}$ heights of the specimen) are shown in **Figure 4.12**. The rubber holders were placed on the location where the silicon rubber grommets were installed.

Figure 4.12 Two part of membrane stretcher with rubber holders

4.4.7 Force Actuator

Force actuator was used for applying an axial load during the shearing stage and maintaining the axial load during the subsequent stages (i.e., infiltration stage of the shearing-infiltration tests).

4.4.8 Thermometer

The thermometer was used to measure the temperature of water inside the triaxial cell during the tests. The temperatures of water were used to calculate the change in volume of water due to the change in temperature. The thermometer has a measurement range from -50°C to 80°C and accuracy of 0.2°C .

4.4.9 Linear Variable Differential Transformer (LVDT)

LVDT was used to measure the axial displacement of soil specimen during the tests by attaching to the loading ram. The LVDT has a measurement range of 25 mm and precision of 0.01 mm.

4.4.10 Data Acquisition System

The DPVC-1 was used to control the cell pressure and measure volume change of specimen during the tests. The pore-air pressure was controlled at a target value by an air pressure system. The pore-water pressure at the base plate was controlled by the DPVC-2. Therefore, matric suction (i.e., $(u_a - u_w)$) of the soil specimen could be controlled. The water volume change of the specimen during the test was recorded by the volume change in the DPVC-2 which indicated the amount of water that flowed into or out of the specimen. Data acquisition system provides channels for three pressure transducers, two NTU mini probes, a temperature sensor and a LVDT. A data acquisition program, Triax 4.0 developed by Toll (1999) was used to record all the data that were transferred to a personal computer. The data could be displayed on the monitor and then opened in the Microsoft Excel. The data could also be copied to a floppy disk or zip disk during the test.

4.5 Testing Procedure

The testing procedure for triaxial tests on saturated soils as proposed by Head (1986) was adopted. The testing procedure for triaxial tests on unsaturated soils as proposed by Fredlund and Rahardjo (1993) was followed. The axis-translation technique proposed by Hilf (1956) was used in order to control the initial matric suction of the specimen.

4.5.1 Testing Procedure for SWCC Tests

4.5.1.1 Saturation Stage

In order to have identical initial water content and degree of saturation, the soil specimen was initially saturated. In this stage, the water pressure line was connected to a digital pressure and volume controller to pump water into the specimen from the top. The specimen was saturated by applying a cell pressure, σ_3 , and a back pressure, u_w with the difference between cell pressure and back pressure being 10

kPa. The specimen was considered to be saturated when the pore-water pressure parameter, B , is close to 1. The specimen was assumed to be fully saturated when the value of B is greater than 0.97 (Head, 1986).

When the saturation stage was completed, the water line to the top loading cap was disconnected from DPVC-2 and the valve was closed. The DPVC-2 was then connected to the pore-water pressure system at the base plate.

4.5.1.2 Consolidation Stage

The soil specimen was isotropically consolidated to the required net confining stress, $(\sigma_3 - u_w)$ after the saturation stage. During the isotropic consolidation stage, the water valve was opened and the pressures were controlled at the required values. The amount of water flowing out from the soil specimen was measured by the DPVC-2 and the data were transferred to the personal computer. The total volume change of the specimen during the test was calculated based on the amount of water flowing in or out of the triaxial cell and the variation of temperature. The amount of water flowing in or out of the cell was measured by the DPVC-1, the variation of temperature was measured by the temperature sensor and the data were transferred to the personal computer. The consolidation stage was considered complete when the water volume change leveled off and the excess pore-water pressure in the specimen was fully dissipated. The matric suction was applied to the specimen when the consolidation stage had been completed.

4.5.1.3 Drying and Wetting Stages

The air line was connected to the air pressure system. The water pressure in the water compartment was measured by the pressure transducer that was connected to the water compartment at the base plate. The net confining stress was set to remain constant at the targeted value by controlling the cell pressure and the air pressure. During the drying stage, matric suction was increased by keeping the air pressure constant and decreasing the water pressure at the base plate. For the wetting stage, matric suction was decreased by increasing the water pressure at the base plate.

The amount of water that flowed out from or in to the specimen and the total volume changes of the specimen during the drying and wetting stages were

measured by the DPVC-1 and DVPC-2 and transferred to the personal computer. The matric suction was considered to be in equilibrium when the excess pore-water pressure had fully dissipated and the water volume change had decreased to 0.04 % per day (Sivakumar, 1993).

4.5.2 Testing Procedure for Isotropic Consolidation Tests

4.5.2.1 Saturation and Matric Suction Equalization Stages

The saturation stage was conducted in a similar manner as the tests for obtaining SWCCs. The matric suction stage was carried out after the saturation stage. At the end of the saturation stage, the net confining stress was 10 kPa. During the matric suction equalization stage the cell pressure and the air pressure remained constant at the same values as their respective values at the end of the saturation stage. The water pressure in the water compartment was set at the desired value. The amount of water flowing out of the soil specimen and the total volume change of the specimen were measured by the DPVC-2 and the DPVC-1, respectively, and the data were transferred to the personal computer.

4.5.2.2 Isotropic Consolidation Stage

The isotropic consolidation stage started when the matric suction equalization stage was completed. The air pressure and the water pressure remained constant. The net confining stress, $(\sigma_3 - u_a)$, was increased during the isotropic consolidation by increasing the cell pressure using the DPVC-1. The rate of stress loading / unloading during the isotropic consolidation was 2 kPa per hour.

The amount of water that flowed in or out of the soil specimen during the isotropic consolidation was measured by the DPVC-2 and the data were transferred to the personal computer. The total volume change of the soil specimen during this stage was also measured by the DPVC-1 and the data were transferred to the personal computer. The isotropic consolidation stage was considered complete when the water flow in or out of the soil specimen was negligible.

4.5.3 Testing Procedure for Consolidated Drained Triaxial Tests

4.5.3.1 Saturation, Consolidation and Matric Suction Equalization Stages

The saturation, consolidation and matric suction equalization stages in the CD triaxial tests were conducted in a similar manner as those the tests for obtaining SWCCs.

4.5.3.2 Shearing Stage

At the end of matric suction equalization stage, the axial load was applied at a constant strain rate in order to shear the specimen. The strain rate of 0.0009 mm/min. had been used for consolidated drained triaxial tests on compacted specimens of the residual soil from Jurong Sedimentary Formation of Singapore (Rahardjo et al., 2004) and compacted specimens of kaolin from Kaolin Malaysia SDN BHD. (Thu et al. 2006). The strain rate of 0.0009 mm/min. was used in this study because the selected soil had similar properties (i.e., plasticity index, coefficient of permeability) with the soil used by Rahardjo et al. 2004 and Thu et al. 2006. During the shearing stage under drained condition, the valves for the water pressure line and the air line were opened and maintained at the required pressure. The shearing stage was considered to finish when the shear plane in the soil specimen occurred or the deviator stress $q = \sigma_1 - \sigma_3$, reached a constant value.

4.5.4 Testing Procedure for Shearing-infiltration Tests

4.5.4.1 Saturation, Consolidation, Matric Suction Equalization and Shearing Stages

The shearing-infiltration test consisted of saturation, consolidation, matric suction equalization, shearing and infiltration stages. The first four stages were the same as those stages carried out in the CD triaxial test.

4.5.4.2 Infiltration Stage

The infiltration stage started when the shearing stage had been completed. The deviator stress, $q = (\sigma_1 - \sigma_3)$, was maintained by the force actuator. Water was then injected into the soil specimen from the base by DPVC2. The rate of water injection of 0.04 mm³/s had been used for the shearing-infiltration tests on compacted specimens of kaolin from Kaolin Malaysia SDN BHD (Meilani et al., 2005). In this study, the rate of water injection of 0.04 mm³/s was used because the selected soil

had similar properties (i.e., plasticity index, coefficient of permeability) with the soil used by Meilani et al., (2005). The infiltration stage was considered to be completed when the specimen had failed.

4.6 Testing Programme

4.6.1 SWCC Tests

A series of SWCC tests was conducted under different net mean stresses $p = (\sigma_1 + \sigma_2 + \sigma_3)/3 - u_a$. The net mean stress during the SWCC test was selected as follows: 10 kPa, 50 kPa, 100 kPa and 250 kPa. **Figure 4.13** shows the stress path for the SWCC tests under different net mean stresses. Each test was named as SWCC-x where the term x refer to net mean stress in kPa. The net mean stress and matric suction that were used in the SWCC-10 test are shown in **Table 4-1**

Figure 4.13 Stress paths for the SWCC under different net confining stresses.

Table 4-1 Net mean stresses and matric suctions that were used in SWCC-10 test.

σ_3 (kPa)	u_a (kPa)	u_w (kPa)	$p-u_a$ (kPa)	u_a-u_w (kPa)	σ_3 (kPa)	u_a (kPa)	u_w (kPa)	$p-u_a$ (kPa)	u_a-u_w (kPa)
Drying curve					Wetting curve				
560	550	550	10	0	560	550	100	10	450
560	550	530	10	20	560	550	180	10	370
560	550	500	10	50	560	550	270	10	280
560	550	450	10	100	560	550	360	10	190
560	550	360	10	190	560	550	450	10	100
560	550	270	10	280	560	550	500	10	50
560	550	180	10	370	560	550	530	10	20
560	550	100	10	450

Figure 4.14 Stress paths for the isotropic consolidation tests under different matric suction.

4.6.2 Isotropic Consolidation Tests

The matric suction for the isotropic consolidation tests were chosen as follows: zero kPa, 30 kPa, 150 kPa and 300 kPa. **Figure 4.14** shows the stress paths for the isotropic consolidation tests under different matric suctions. The isotropic consolidation test was named as IC-y where the term y refers to matric suction during the test in kPa.

4.6.3 Consolidated Drained Triaxial Tests

The single stage test was adopted for the CD triaxial tests. During the shearing stage matric suction was maintained at 0 kPa, 100 kPa or 200 kPa.

Table 4-2 shows the testing programme for CD triaxial tests. The CD triaxial test was named as CDxx-yy where the term xx refers to net confining stress and the term yy refers to matric suction during the shearing stage.

Figure 4.15 Stress paths for CD triaxial tests under difference net confining stress and matric suction.

Table 4-2 Programme for the CD triaxial tests

Net confining stress, ($\sigma_3 - u_a$) (kPa)	Matric suction, $s = (u_a - u_w)$ (kPa)		
	0	100	200
100	CD100-0	CD100-100	CD100-200
300	CD300-0	CD300-100	CD300-200

4.6.4 Shearing-infiltration Tests

The matric suction at the beginning of the shearing stage was 200 kPa or 300 kPa. The net confining stresses at the beginning of the shearing stage was 100 kPa, 200 kPa or 300 kPa. The specimen was sheared up to around 80% of the peak shear stress at failure. **Figure 4.16** shows the stress paths for the shearing-infiltration tests under different net confining stresses, matric suctions and deviator stresses.

Figure 4.16 Stress paths for the shearing-infiltration tests under different net confining stresses, matric suctions and deviator stresses.

Table 4-3 Programme for the shearing-infiltration tests

Net confining stress, ($\sigma_3 - u_a$) (kPa)	Matric suction, $s = (u_a - u_w)$ (kPa)		
	100	200	300
200		SI200-200	
300	SI300-100		SI300-300

4.7 Prediction from the Proposed Model

4.7.1 Prediction Programme

The proposed model presented in Chapter 3 was used to predict the behaviour of soil specimens tested in this study under consolidated drained and shearing-infiltration conditions. The soil parameters for the model prediction were obtained from SWCC tests under different net mean stresses and isotropic consolidation tests under different matric suctions (i.e., $N(s)$, $\lambda(s)$, $\kappa(s)$, $N_s(p)$, $\lambda_s(p)$ and $\kappa_s(p)$).

The proposed model presented in Chapter 3 was also used to predict the behaviour of soil specimens that were tested under consolidated drained and shearing-infiltration conditions as reported in the literature.

Meilani (2004) and Meilani et al. (2005) conducted a series of shearing-infiltration tests on compacted kaolin specimens using modified triaxial apparatuses. Thu (2005), Thu et al. (2006) and Thu et al. (2007) conducted a series of isotropic consolidation tests, a series of SWCC tests and a series of consolidated drained triaxial tests on compacted kaolin specimens using modified triaxial apparatuses. The compacted specimens that were used in Meilani (2004), Meilani et al. (2005), Thu (2005), Thu et al. (2006) and Thu et al. (2007) were identical since the same Kaolin and compaction method, dry density, water content were used to prepare those specimens. These test results provide input parameters for prediction using proposed model presented in Chapter 3 and verification of the proposed model.

The basic properties of kaolin were used in the tests by Meilani (2004), Meilani et al. (2005), Thu (2005) and Thu et al. (2007) are shown in **Table 4-4**. The specimens were compacted at a maximum dry density of 1.35 Mg/m^3 and an optimum water

content of 22%. The results from isotropic consolidation tests conducted by Thu (2005) and Thu et al. (2007) are shown in **Table 4-5**.

Table 4-4 Index properties of Kaolin used by Meilani (2004), Meilani et al. (2005), Thu (2005) and Thu et al. (2007) (after Meilani (2004), Meilani et al. (2005), Thu (2005) and Thu et al. (2007))

Soil properties	Value
Specific gravity, G_s	2.65
Liquid limit, LL (%)	51.0
Plastic limit, PL (%)	35.6
Plastic index, PI (%)	15.4
Clay (%)	15
Silt (%)	85
Unified soil classification system (USCS)	MH
Maximum dry density, $\rho_{\text{dry max}}$ (Mg/m^3)	1.35
Optimum water content, w_{opt} (%)	22

Table 4-5 Summary of parameters obtained from the isotropic consolidation tests on compacted kaolin conducted by Thu (2005) and Thu et al. (2007) (after Thu (2005) and Thu et al. (2007))

Matric suction (kPa)	p_o (kPa)	$\kappa(s)$	$\lambda(s)$	$N(s)$	$\lambda_w(s)$
0	22	0.027	0.063	1.937	0.063
50	72	0.024	0.057	1.929	0.038
100	160	0.015	0.047	1.914	0.011
150	210	0.013	0.043	1.907	0.008
200	235	0.011	0.040	1.902	0.004
300	244	0.010	0.039	1.897	0.001

The CD triaxial test conducted by Thu (2005) was named as CDxx-yy (Thu 2005) where the term xx refers to net mean stress and the term yy refers to matric suction during the shearing stage. Table 4-6 shows the consolidated drained triaxial tests conducted by Thu (2005).

Table 4-6 Consolidated drained triaxial tests conducted by Thu (2005)

Net confining stress, ($\sigma_3 - u_a$) (kPa)	Matric suction, $s = (u_a - u_w)$ (kPa)			
	0	100	200	300
100	CD100-0 (Thu 2005)	CD100-100 (Thu 2005)	CD100-200 (Thu 2005)	CD100-300 (Thu 2005)
200	CD200-0 (Thu 2005)	CD200-100 (Thu 2005)	CD200-200 (Thu 2005)	CD200-300 (Thu 2005)
300	CD300-0 (Thu 2005)	CD300-100 (Thu 2005)	CD300-200 (Thu 2005)	CD300-300 (Thu 2005)

The shearing-infiltration test conducted by Meilani (2004) was named as SIxx-yy (Meilani 2004) where the term xx refers to net mean stress and the term yy refers to matric suction during the shearing stage. Table 4-7 shows the consolidated drained tests conducted by Meilani (2004).

Table 4-7 Shearing-infiltration tests conducted by Meilani (2004)

Net confining stress, ($\sigma_3 - u_a$) (kPa)	Matric suction, $s = (u_a - u_w)$ (kPa)		
	50	100	200
25	SI25-50 (Meilani 2004)	SI25-100 (Meilani 2004)	SI25-100 (Meilani 2004)
100	SI25-100 (Meilani 2004)	SI25-100 (Meilani 2004)	SI25-100 (Meilani 2004)
200	SI25-100 (Meilani 2004)	SI25-100 (Meilani 2004)	SI25-100 (Meilani 2004)

4.7.2 Prediction Procedure

The initial point for all of the prediction process is at the end of the saturation stage. At this point, the predicted soil specimen has net mean stress, p , of 10 kPa, matric suction, s , of zero kPa, specific volume, v_i , yield stress at matric suction of zero kPa and initial yield suction at net mean stress of 10 kPa.

Prediction of initial loading-collapse yield curve and suction increase yield curve of SWCC tests and isotropic consolidation tests

For the cases of SWCC test, the elastic strain and plastic strain at the end of the consolidation stage are calculated based on the initial yield stress at matric suction of zero kPa, $p_{oi}(0)$, Equation (3.13) and Equation (3.19). The corresponding yield suction at net mean stress of 10 kPa at the end of consolidation stage is calculated based on Equation (3.31). Initial loading-collapse yield curve and suction increase yield curve of the SWCC tests are predicted using Equation (3.25) and Equation (3.29), respectively.

For the cases of isotropic consolidation test, the elastic strain and plastic strain at the end of the matric suction equalization stage are calculated based on the initial yield suction at net mean stress of 10 kPa, $s_{oi}(10)$, Equation (3.17) and Equation (3.21). The corresponding yield stress at the end of matric suction equalization stage is calculated based on Equation (3.30). The initial loading-collapse yield curve and the suction increase yield curve of the isotropic consolidation tests are predicted using Equation (3.25) and Equation (3.29).

Prediction of CD triaxial tests

The elastic strain and plastic strain at the end of the consolidation stage are calculated based on the initial yield stress at matric suction of zero kPa, $p_{oi}(0)$, Equation (3.13) and Equation (3.19). The corresponding yield suction at net mean stress of 10 kPa at the end of consolidation stage is calculated based on Equation (3.31).

The elastic strain and plastic strain at the end of the matric suction equalization stage are calculated based on the initial yield suction at net mean stress of 10 kPa,

$s_{oi}(10)$, Equation (3.17) and Equation (3.21). The corresponding yield stress at the end of matric suction equalization stage is calculated based on Equation (3.30).

During the shearing stage, an incremental in deviator stress, Δq , of 1 kPa is applied while the confining stress and matric suction are maintained constant. As a result, the increment in net mean stress is 1/3 kPa. For each loading step, the corresponding yield stress, the yield suction at isotropic stage (i.e., the corresponding yield stress and the yield suction on (p - s) plane) are calculated based on the consistency condition (i.e., Equation (3.41) and Equation (3.42)). The elastic strains are calculated based on Equation (3.13), Equation (3.15) and Equation (3.17). The plastic strains are calculated based on Equation (3.19), Equation (3.21) and Equation (3.35) or Equation (3.36). The prediction stops when axial strain increases rapidly, indicating failure condition.

The loading-collapse yield curve and the suction increase yield curve at the beginning of the shearing stage (i.e., at the end of matric suction equalization stage) and at the end of shearing stage can be predicted using Equation (3.25) and Equation (3.29). **Figure 4.17** shows the sketch of loading-collapse yield curve and suction increase yield curve at the beginning of the shearing stage and at the end of shearing stage together with the projection of the yield path on the (p - s) plane during the shearing stage (i.e., the locus of point B or point D in Figure 3.9).

Figure 4.17 Sketch of loading-collapse yield curve and suction increase yield curve at the beginning of the shearing stage and at the end of shearing stage together with the projection of yield path on the $(p-s)$ plane during the shearing stage

Prediction of shearing infiltration tests

The prediction of consolidation stage, matric suction equalization stage and shearing stage of the shearing infiltration tests are similar to the prediction of CD triaxial tests. For the infiltration stage, an increment in matric suction, Δs , of 0.5 kPa is applied while net mean stress and deviator stress are maintained constant. For each loading step, the corresponding yield stress, yield suction at isotropic stage (i.e., the corresponding yield stress and yield suction on $(p-s)$ plane) are calculated based on the consistency condition (i.e., Equation (3.41) and Equation (3.42)). The elastic strains are calculated based on Equation (3.13), Equation (3.15) and Equation (3.17). The plastic strains are calculated based on Equation (3.19), Equation (3.21) and Equation (3.35) or Equation (3.36). The prediction stops when axial strain increases rapidly, indicating failure condition.

The loading-collapse yield curve and suction increase yield curve at the beginning of the shearing stage (i.e., at the end of matric suction equalization stage), at the end of shearing stage and at the end of infiltration stage can be predicted using Equation (3.25) and Equation (3.29). **Figure 4.18** shows the sketch of loading-collapse yield

curve and suction increase yield curve at the beginning of the shearing stage, at the end of the shearing stage and at the end of the infiltration stage together with the projection of yield paths on the $(p-s)$ plane during the shearing and infiltration stage (i.e., the locus of point B or point D in Figure 3.9).

SI₁ - SI yieldcurve at beginning of shearing stage LC₁ - LC yieldcurve at beginning of shearing stage
 SI₂ - SI yield curve at end of shearing stage LC₂ - LC yield curve at end of shearing stage
 SI₃ - SI yield curve at end of infiltration stage LC₃ - LC yield curve at end of infiltration stage

Figure 4.18 Sketch of loading-collapse yield curve and suction increase yield curve at the beginning of the shearing stage, at the end of the shearing stage and at the end of the infiltration stage together with the projection of yield paths on the $(p-s)$ plane during the shearing stage and infiltration stage

CHAPTER 5 EXPERIMENTAL RESULTS

5.1 Introduction

In this chapter, firstly the results of basic soil properties (i.e., index properties and compaction test) of coarse Kaolin made by Kaolin Malaysia SDN BHD, the SWCC tests under different net mean stresses and the isotropic consolidation tests under different matric suctions are presented. Secondly, the results of consolidated drained triaxial tests are presented. Subsequently, the extended Mohr-Coulomb envelope obtained from the consolidated drained triaxial tests is presented. The final section presents the results of shearing-infiltration tests.

5.2 Index Properties

5.2.1 Index Properties

The specific gravity of coarse Kaolin of 2.68 was determined according to ASTM D 854-00. The coarse Kaolin has a liquid limit of 56.1 %, a plastic limit of 37.9 % and a plasticity index of 18.2 %. The liquid limit and plastic limit were determined according to BS 1377-2:1990 and ASTM D 4318-00, respectively. Based on the plasticity chart of the Unified Soil Classification System (ASTM D 2487-00), the Kaolin was classified as silt with high plasticity (MH). **Figure 5.1** shows position of the Kaolin on the Unified Soil Classification System plasticity chart (ASTM D 2487-00)

The grain size distribution curve of the coarse Kaolin was determined from a hydrometer test according to ASTM D 422-63 (2002). **Figure 5.2** shows the grain size distribution curve of Kaolin. Based on **Figure 5.2**, Kaolin consists of 83.7 % of silt and 16.3 % of clay (finer than 2 μm).

Figure 5.1 Classification of Kaolin in accordance with the Unified Soil Classification System plasticity chart (ASTM D 2487-00)

Figure 5.2 Grain size distribution curve of Kaolin

The standard Proctor compaction test was conducted according to ASTM D 698-00. The compaction curve (i.e., the relationship between dry density, ρ_d , and water content, w) was presented in **Figure 5.3**. The Kaolin has a maximum dry density of 1.37 Mg/m^3 (i.e., $\gamma_{\max}=13.44 \text{ kN/m}^3$) and an optimum water content of 26.5%.

Figure 5.3 Compaction curve of Kaolin obtained from standard Proctor tests

Table 5.1 Index properties of Kaolin

Soil properties	Value
Specific gravity, G_s	2.68
Liquid limit, LL (%)	56.1
Plastic limit, PL (%)	37.9
Plastic index, PI (%)	18.2
Silt (%)	83.7
Clay (%)	16.3
Unified soil classification system (USCS)	MH
Maximum dry density, ρ_{drymax} (Mg/m³)	1.37
Optimum water content, w_{otp} (%)	26.5

In order to have a relatively high saturated permeability and to avoid the rising up of excess pore-water pressure during the static compaction process, the dry density, ρ_d of 1.35 Mg/m³ and water content of 21.7 % (i.e., at 4.8% dry of optimum) were chosen as the initial condition for all the specimens used in the triaxial testing programme (i.e., SWCC tests, isotropic consolidation tests, consolidated drained triaxial tests and shearing infiltration tests).

5.2.2 Soil Water Characteristic Curves

The influence of net mean stress on SWCC of compacted Kaolin was investigated by using the modified triaxial apparatus to conduct SWCC tests at different net mean stresses.

During the drying stage, water flowed out from the specimen as a result total volume and water volume of soil specimen decreased. **Figure 5.4** shows the relationships between total volume change; water volume change and elapsed time of specimen SWCC-250 for the case of matric suction increased from 100 kPa to 190 kPa. In contrast, during the wetting stage, water flowed into the specimen and therefore, total volume and water volume of soil specimen increased. The relationships between total volume change; water volume change and elapsed time of specimen SWCC-100 for the case of matric suction decrease from 190 kPa to 100 kPa are shown in **Figure 5.5**. The total volume change and water volume change reached equilibrium after about 65 hours for the drying stage and about 50 hours for the wetting stage.

Figure 5.4 Total volume change and water volume change versus elapsed time of specimen SWCC-250 for the case of matric suction increase from 100 kPa to 190 kPa.

Figure 5.5 Total volume change and water volume change versus elapsed time of specimen SWCC-100 for the case of matric suction decrease from 190 kPa to 100 kPa.

Water flowed out from the specimen during the drying stage and as a result pore-water pressure in soil specimen decreased to the controlled value at the base plate. The change of pore-water pressure during the drying state of specimen SWCC-250 is shown in **Figure 5.6**. As the matric suction increased the time require for pore-water pressure in the specimen to reach equilibrium increased. When matric suction increased from 370 kPa to 450 kPa, the measurement of pore-water pressure by NTU mini suction probe showed the same trend as the previous stage i.e., decreased and then increased instead of reaching equilibrium pressure with the controlled pressure at the base plate.

Figure 5.6 Pore-water pressure versus elapsed time during drying stage of specimen SWCC-250.

The variations of specific volume, specific water volume and volumetric water content with respect to matric suction of specimen SWCC-10 are presented in **Figure 5.7**. The specific volume, $v=1+e$, decreased as matric suction increased. Matric suction at the point when specific volume, v , starts to increase drastically (i.e. inflection point) is called yield suction, s_o , of soil specimen (Alonso et al. 1990). The volumetric water content (i.e., the ratio between volume of water and total volume of soil, $\theta_w=V_w/V$) also decreased as matric suction increased. The value of matric suction at which air first enters the pore of soil is called air-entry value (AEV).

Figure 5.7 Variations of volumetric water content, specific volume and specific water volume with respect to matric suction of specimen SWCC-10

Figure 5.8 Variations of volumetric water content, specific volume and specific water volume with respect to matric suction of specimen SWCC-50

Figure 5.9 Variations of volumetric water content, specific volume and specific water volume with respect to matric suction of specimen SWCC-100

Figure 5.10 Variations of volumetric water content, specific volume and specific water volume with respect to matric suction of specimen SWCC-250

Similar to the case of specimen SWCC-10, **Figure 5.8** to **Figure 5.10** present the variations of specific volume, specific water volume and volumetric water content with respect to matric suction of specimens SWCC-50, SWCC-100 and SWCC-250, respectively.

The summaries of specific volume, specific water volume and volumetric water content with respect to matric suction under different net mean stresses are shown in **Figure 5.11** to **Figure 5.13**, respectively. The SWCCs in **Figure 5.11** show that the AEV increase as net mean stress increase. The results from **Figure 5.12** show that yield suction, s_o , also increases as net mean stress increases. Soil parameters obtained from SWCC tests are summarized in **Table 5.2**.

Table 5.2 Summary of soil parameter obtained from SWCC tests

Net mean stress, p (kPa)	AEV (kPa)	$N_s(p)$	$\lambda_s(p)$	$\kappa_s(p)$	s_o (kPa)
10	55	2.075	0.044	0.009	18
50	60	2.056	0.026	0.007	18.5
100	71	2.031	0.020	0.004	21.8
250	91	1.976	0.020	0.004	32.5

Figure 5.11 Soil water characteristic curves at different net mean stresses

Figure 5.12 Specific volume, v , versus matric suction at different net mean stresses

Figure 5.13 Specific water volume, v_w , versus matric suction at different net mean stresses

Figure 5.7 to **Figure 5.10** show that the yield suction and the air-entry value increases as the net mean stress increases (i.e., the stiffness of soil increases with the increase in net mean stress). The yield suction at net mean stress of 10 kPa is 18.5 kPa. The results of the tests are in agreement with the results from Thu et al. (2007). The variations of yield suction with respect to net mean stress from SWCC tests are shown in **Figure 5.14**.

Figure 5.14 Air-entry value, AEV and yield suction, $s_o(p)$ at different net mean stresses as obtained from isotropic suction consolidation curves

Figure 5.15 Measured values of $N_s(p)$ at various net mean stresses as obtained from SWCCs

The specific volume, v , on the virgin curve of the isotropic suction consolidation curve (i.e., the specific volume versus matric suction curve obtained from SWCC test) at reference matric suction of 100 kPa (i.e., $N_s(p)$) also varies with net mean stress as shown in **Figure 5.15**. At net mean stress of 10 kPa, the $N_s(p)$ has a value of 2.075. The $N_s(p)$ decreases with the increase in net mean stress.

The change in slope of the virgin curve, $\lambda(s)$, of the isotropic suction consolidation curve with respect to net mean stress of compacted kaolin is shown in **Figure 5.16**. At net mean stress of 10 kPa, the slope of virgin curve, $\lambda_s(10)$ is 0.044. With the increase in net mean stress, the $\lambda_s(p)$ decreases. The slopes of unloading/reloading curves, $\kappa_s(s)$, of the isotropic suction consolidation curves are also shown in **Figure 5.16**. Similar to the case of $N_s(p)$ and $\lambda_s(p)$, the $\kappa_s(p)$ decreases with the increase in net mean stress.

Figure 5.16 Slope of virgin curve and unloading/reloading curve at various net mean stresses as obtained from isotropic suction consolidation curves

5.2.3 Isotropic Consolidation Curves

The variation of specific volume, $v=1+e$, and specific water volume, $v_w=1+Se$, with respect to net mean stress at various matric suctions as obtained from isotropic consolidation tests are shown in **Figure 5.17** and **Figure 5.18**, respectively. **Figure 5.17** shows specific volume decreases rapidly when the net mean stress reaches yield stress. The results of the tests show that the yield stress increases as the matric suction increases (i.e., the stiffness of soil increase as the increase in matric suction). The yield stress at saturated condition (i.e., matric suction, $s = 0$ kPa) is 25 kPa. The results of the tests are in agreement with the results from Burland (1965), Alonso et al. (1990), Rampino et al. (2000) and Thu et al. (2007). The variations of

yield stress with respect to matric suction from isotropic consolidation tests are shown in **Figure 5.19**.

Figure 5.17 Specific volume, v , versus net mean stresses at different matric suctions

Figure 5.18 Specific water volume, v_w , versus net mean stresses at different matric suctions

Figure 5.19 Yield stress, $p_o(s)$ at different matric suctions as obtained from isotropic consolidation curves

The specific volume, v , on normal consolidation curve at reference net mean stress of 100 kPa (i.e., $N(s)$) also varies with matric suction as shown in **Figure 5.20**. The reference mean net stress of 100 kPa was used by Wheeler and Sivakumar (1995) and Thu et al. (2007). At saturated condition, the $N(s)$ has value of 2.052. The $N(s)$ decreases with the increase in matric suction. The specific water volumes at reference net mean stress of 100 kPa, $N_w(s)$ are also shown in **Figure 5.20**. The $N_w(0)$ (i.e., $N_w(s)$ at saturated condition) has value of 2.052. Similar to $N(s)$, $N_w(s)$ decreases with the increase in matric suction.

Figure 5.20 Measured values of $N(s)$ at various matric suctions as obtained from isotropic consolidation curves

Figure 5.21 Slope of normal consolidation curve and unloading/reloading curve at various matric suctions as obtained from isotropic consolidation curves

The change in slope of normal consolidation curve, $\lambda(s)$, with respect to matric suction of compacted kaolin is shown in **Figure 5.21**. At saturated condition, the slope of consolidation curve, $\lambda(0)$ is 0.071. With the increase in matric suction, the $\lambda(s)$ decreases. The slopes of unloading/reloading curves, $\kappa(s)$, of the isotropic consolidation curves are also shown in **Figure 5.21**. Similar to the case of $N(s)$ and $\lambda(s)$, the $\kappa(s)$ decreases with the increase in matric suction.

Figure 5.22 Measured values of $\kappa_w(s)$ and $\lambda_w(s)$ at various matric suctions as obtained from isotropic consolidation tests

Figure 5.22 shows the variation of the measurement values of $\kappa_w(s)$ and $\lambda_w(s)$ with respect to matric suction. Similar to the case of $\kappa(s)$ and $\lambda(s)$, $\kappa_w(s)$ and $\lambda_w(s)$ decrease with the increase in matric suction.

The parameters obtained from the isotropic consolidation tests on compacted kaolin are summarized in **Table 5.3**

Table 5.3 Summary of parameters obtained from the isotropic consolidation tests on compacted kaolin

Matric suction (kPa)	p_o (kPa)	$\kappa(s)$	$\lambda(s)$	$N(s)$	$\kappa_w(s)$	$\lambda_w(s)$	$N_w(s)$
0	25	0.025	0.071	2.052	0.025	0.071	2.052
30	39	0.018	0.068	2.045	0.018	0.068	2.045
150	91.5	0.009	0.046	2.041	0.006	0.036	1.685
300	120	0.007	0.044	2.034	0.004	0.033	1.507

The variations of matric suction during the IC-30 and IC-300 tests are shown in **Figure 5.23**. During the loading stage, matric suction decreases first then increases but it is still lower than the initial value (i.e., the matric suction at beginning of the loading stage). During the unloading stage, the matric suction increases monotonically. The pore-air pressure is maintained at a constant value during the loading-unloading stage. In other words, the change in matric suction is due to the change in pore-water pressure. During the loading stage of the IC-30 test, matric suction decreases down to 10 kPa, during the unloading stage matric suction increases up to 8 kPa. For the IC-300 test, matric suction decrease down to 5 kPa during the loading stage.

Figure 5.23 Matric suction, $s=u_a-u_w$, versus net mean stress during isotropic consolidation tests, (a) for the case of initial matric suction of 30 kPa, (b) for the case of initial matric suction of 300 kPa

5.3 Consolidated Drained (CD) Triaxial Test Results

This section presents the results of CD triaxial tests conducted on compacted kaolin specimens. The CD triaxial tests were conducted by using the modified triaxial apparatus at a matric suction of 0 kPa, 100 kPa, 200 kPa and a net confining stress of 100 kPa or 300 kPa. The results of the CD triaxial tests are presented for the

same matric suction but under different net confining stresses and the same net confining stress but under different matric suctions.

5.3.1 Shear Strength Behaviour

The variation of deviator stress, $q = \sigma_1 - \sigma_3$ with respect to axial strain, ε_a , during the shearing stage of compacted kaolin specimens under the same matric suction of 0 kPa, 100 kPa or 200 kPa and different net confining stresses (i.e., 100 kPa or 300 kPa) are shown in **Figure 5.24**, **Figure 5.25** and **Figure 5.26**, respectively. **Figure 5.24**, **Figure 5.25** and **Figure 5.26** show that the stiffness of the compacted kaolin specimen increases with the increase in net confining stress. **Figure 5.25** and **Figure 5.26** indicate that the deviator stress versus axial strain curves for the case of net confining stress of 100 kPa show a rapid decrease in deviator stress after reaching the peak value.

Figure 5.24 Variation of deviator stresses with respect to axial strain from CD triaxial tests under the same matric suction of zero kPa and net confining stress of 100 kPa or 300 kPa

Figure 5.25 Variation of deviator stresses with respect to axial strain from CD triaxial tests under the same matrix suction of 100 kPa and net confining stress of 100 kPa or 300 kPa

Figure 5.26 Variation of deviator stresses with respect to axial strain from CD triaxial tests under the same matrix suction of 200 kPa and net confining stress of 100 kPa or 300 kPa

Figure 5.27 Variation of deviator stresses with respect to axial strain from CD triaxial tests under the same net confining stresses of 100 kPa and matric suction of 0 kPa, 100 kPa or 200 kPa

Figure 5.28 Variation of deviator stresses with respect to axial strain from CD triaxial tests under the same net confining stresses of 300 kPa and matric suction of 0 kPa, 100 kPa or 200 kPa

Figure 5.27 and **Figure 5.28** show the variation of deviator stress, $q = \sigma_1 - \sigma_3$ with respect to axial strain, ϵ_a , during the shearing stage under the same net confining

stress of 100 kPa or 300 kPa but different matric suction (i.e., 0 kPa, 100 kPa or 300 kPa) on compacted kaolin specimens. **Figure 5.27** and **Figure 5.28** also point out that the stiffness of compacted kaolin specimen increased with the increase of matric suction.

The void ratio, e , water content, w , and the degree of saturation, S , at each stage of the CD triaxial tests on compacted kaolin are summarized in **Table 5.4**.

Table 5.4 Void ratio, e , water content, w , and degree of saturation, S , of the CD triaxial tests on compacted Kaolin

State	End of consolidation			End of matric suction equalization			After shearing		
Specimen	e	w (%)	S (%)	e	w (%)	S (%)	e	w (%)	S (%)
CD100-0	1.066	39.65	100				1.052	39.73	100
CD300-0	0.985	36.40	99				0.934	36.58	100
CD100-100	1.073	40.05	100	1.023	33.69	88.26	1.014	31.57	83.42
CD300-100	0.981	36.60	100	0.959	35.07	98.05	0.890	32.67	98.37
CD100-200	1.081	40.34	100	1.032	23.45	60.86	1.039	21.33	55.04
CD300-200	0.995	37.14	100	0.964	22.27	61.88	0.923	20.04	58.19

5.3.2 Total Volume Change Characteristics

The variation of volumetric strain, ε_v , with respect to axial strain, ε_a , during the shearing stage of compacted kaolin specimens under the same matric suction of 0 kPa, 100 kPa or 200 kPa and different net confining stress (i.e., 100 kPa or 300 kPa) are shown in **Figure 5.29**, **Figure 5.30** and **Figure 5.31**, respectively. **Figure 5.29**, **Figure 5.30** and **Figure 5.31** show that the higher the net confining stress, the higher the compression will be.

Figure 5.29 Variation of volumetric strain with respect to axial strain from CD triaxial tests under the same matrix suction of zero kPa and net confining stress of 100 kPa or 300 kPa

Figure 5.30 Variation of volumetric strain with respect to axial strain from CD triaxial tests under the same matrix suction of 100 kPa and net confining stress of 100 kPa or 300 kPa

Figure 5.31 Variation of volumetric strain with respect to axial strain from CD triaxial tests under the same matric suction of 200 kPa and net confining stress of 100 kPa or 300 kPa

Figure 5.32 Variation of volumetric strain with respect to axial strain from CD triaxial tests under the same net confining stresses of 100 kPa and matric suction of 0 kPa, 100 kPa or 200 kPa

Figure 5.33 Variation of volumetric strain with respect to axial strain from CD triaxial tests under the same net confining stresses of 300 kPa and matric suction of 0 kPa, 100 kPa or 200 kPa

Figure 5.32 and **Figure 5.33** show the variation of volumetric strain, ϵ_v , with respect to axial strain, ϵ_a , during the shearing stage of compacted kaolin specimens under the same net confining stress of 100 kPa or 300 kPa and different matric suction (i.e., 0 kPa, 100 kPa or 300 kPa). **Figure 5.32** illustrates that specimens CD 100-0, CD 100-100 and CD 100-200 compresses first and then dilates afterwards. On the other hand, **Figure 5.33** shows that specimens CD 300-0, CD 300-100 and CD 300-200 compresses all the way through the whole shearing stage.

5.3.3 Water Volume Change Characteristics

The variation of water volumetric strain, ϵ_w , with respect to axial strain, ϵ_a , during the shearing stage of compacted kaolin specimens under the same matric suction of 0 kPa, 100 kPa or 200 kPa and different net confining stresses (i.e., 100 kPa or 300 kPa) are shown in **Figure 5.34**, **Figure 5.35** and **Figure 5.36**, respectively. **Figure 5.34** shows that the water volumetric strain of specimens CD 100-0 and CD 300-0 decreases first then increases. However, **Figure 5.35** and **Figure 5.36** depict that the water volumetric strain of specimens CD 100-100, CD 100-200, CD 300-100 and CD 300-200 decreases all the way through the entire shearing stage.

Figure 5.34 Variation of water volumetric strain with respect to axial strain from CD triaxial tests under the same matrix suction of zero kPa and net confining stress of 100 kPa or 300 kPa

Figure 5.35 Variation of water volumetric strain with respect to axial strain from CD triaxial tests under the same matrix suction of 100 kPa and net confining stress of 100 kPa or 300 kPa

Figure 5.36 Variation of water volumetric strain with respect to axial strain from CD triaxial tests under the same matric suction of 200 kPa and net confining stress of 100 kPa or 300 kPa

5.4 Extended Mohr-Coulomb Failure Envelope from Consolidated Drained Tests

5.4.1 Failure Criterion

Figure 5.24, **Figure 5.25** and **Figure 5.26** show the variation of deviator stress, $q = \sigma_1 - \sigma_3$ with respect to axial strain, ϵ_a , during the shearing stage of compacted kaolin specimens under the same matric suction of 0 kPa, 100 kPa or 200 kPa but different net confining stresses (i.e., 100 kPa or 300 kPa). The peak deviator stress was used as failure criterion. **Figure 5.37** demonstrates the use of peak deviator stress as a failure criterion for the CD triaxial tests. The values of axial strain at failure are also shown in **Figure 5.37**. The values of axial strains at failure are summarized in **Table 5.5**.

Figure 5.37 Peak deviator stresses of the consolidation drained triaxial tests under matric suction of 0 kPa, 100 kPa or 200 kPa and net confining stress of 100 kPa or 300 kPa.

Table 5.5 Summary of deviator stresses and axial strains at failure for the CD triaxial tests under matric suction of 0 kPa, 100 kPa or 200 kPa and net confining stress of 100 kPa or 300 kPa

Specimen	Deviator stress, q_f (kPa) at failure	Axial strain, ε_{af} (%) at failure
CD 100-0	242.05	9.15
CD 300-0	601.75	12.27
CD 100-100	373.31	8.74
CD 300-100	648.65	11.75
CD 100-200	534.19	8.92
CD 300-200	793.56	10.81

5.4.2 Mohr-Coulomb Failure Envelope from Consolidated Drained Tests

The extended Mohr-Coulomb failure envelope of the compacted kaolin from the CD triaxial tests at matric suction of zero kPa, 100 kPa or 200 kPa and at net confining stress of 100 kPa or 300 kPa are presented in **Figure 5.39**.

Figure 5.38 Extended Mohr-Coulomb failure envelope for compacted kaolin obtained from CD triaxial tests under matric suction of 0 kPa, 100 kPa or 200 kPa and at net confining stress of 100 kPa or 300 kPa

Figure 5.39 Extended Mohr-Coulomb failure envelope for compacted kaolin obtained from CD triaxial tests under zero matric suction

Figure 5.40 Extended Mohr-Coulomb failure envelope for compacted kaolin obtained from CD triaxial tests under matric suction of 100 kPa

Figure 5.41 Extended Mohr-Coulomb failure envelope for compacted kaolin obtained from CD triaxial tests under matric suction of 200 kPa

The failure envelopes of the compacted kaolin from the CD triaxial tests at matric suction of zero kPa, 100 kPa or 200 kPa are also presented in **Figure 5.39**, **Figure 5.40** and **Figure 5.41**, respectively. **Figure 5.42** shows the extended Mohr-Coulomb failure envelope for the compacted kaolin as obtained from CD triaxial tests at the net normal stress of zero kPa plane. Based on **Figure 5.39**, an effective

angle of internal friction, ϕ' , of 25.6° and effective cohesion, c' , of 33.3 kPa can be obtained. The angle indicating the rate of change in shear strength relative to matric suction, ϕ^b , of 21.1° can be obtained from **Figure 5.42**.

Figure 5.42 Extended Mohr-Coulomb failure envelope for compacted kaolin as obtained from CD triaxial tests at the net normal stress of zero kPa plane

5.5 Shearing-infiltration Tests Results

This section presents the results of three shearing-infiltration triaxial tests conducted on compacted kaolin specimens. The first shearing-infiltration triaxial test was conducted by using the modified triaxial apparatus at matric suction of 200 kPa and net confining stress of 200 kPa. The second and the third tests were conducted by using the modified triaxial apparatus at matric suction of 300 kPa and net confining stress of 100 kPa and 300 kPa, respectively.

5.5.1 Shear Strength Behaviours

The variation of deviator stress, $q = \sigma_1 - \sigma_3$ with respect to axial strain, ϵ_a , during the shearing stage and the infiltration stage on compacted kaolin specimens, SI 100-300, SI 200-200 and SI 300-300 are shown in **Figure 5.43**, **Figure 5.44** and **Figure 5.45**, respectively.

Figure 5.43 Variation of deviator stresses with respect to axial strain from SI 100-300 triaxial test

Figure 5.44 Variation of deviator stresses with respect to axial strain from SI 200-200 triaxial test

Figure 5.45 Variation of deviator stresses with respect to axial strain from SI 300-300 triaxial test

Figure 5.43, **Figure 5.44** and **Figure 5.45** show the reduction in deviator stress in between the shearing stage and the infiltration stage. During the infiltration stage the deviator stress also decreases. **Figure 5.45** show the change in deviator stress during the infiltration stage of specimen SI 300-300 at axial strain of 6.75 %. This change was caused by the interruption of the electricity in the laboratory.

The void ratio, e , water content, w , and the degree of saturation, S , at each stage of the CD triaxial tests on compacted kaolin are summarized in **Table 5.6**.

Table 5.6 Void ratio, e , water content, w , and degree of saturation, S , of the shearing-infiltration triaxial tests on compacted kaolin

State	End of consolidation			End of matric suction equalization			End of shearing			End of infiltration		
	e	w (%)	S (%)	e	w (%)	S (%)	e	w (%)	S (%)	e	w (%)	S (%)
Specimen												
SI100-300	1.055	39.36	100	0.988	18.59	50.41	0.955	17.45	48.95	1.016	22.84	60.27
SI200-200	1.020	38.08	100	0.978	22.61	61.99	0.934	21.12	60.60	1.032	31.01	80.50
SI300-300	0.989	36.97	100	0.956	19.61	54.97	0.909	18.13	53.46	0.926	28.53	82.60

5.5.2 Total Volume Change Characteristics

The variation of volumetric strain, ε_v , with respect to axial strain, ε_a , during the shearing stage and the infiltration stage of compacted kaolin specimens, SI 100-300, SI 200-200 and SI 300-300 are shown in **Figure 5.46**, **Figure 5.47** and **Figure 5.48**, respectively. **Figure 5.46** to **Figure 5.48** show the decrease in volumetric strain during the shearing stage and its increase during the infiltration stage.

Figure 5.46 Variation of volumetric strain with respect to axial strain from SI 100-300 triaxial test

Figure 5.47 Variation of volumetric strain with respect to axial strain from SI 200-200 triaxial test

Figure 5.48 Variation of volumetric strain with respect to axial strain from SI 300-300 triaxial test

5.5.3 Matric Suction Change Characteristics

The variations of matric suction, s , with respect to axial strain, ϵ_a , from the shearing-infiltration tests on compacted kaolin specimens (i.e., SI 100-300, SI 200-200 and SI 300-300) are shown in **Figure 5.49** to **Figure 5.51**.

Figure 5.49 Variation of matric suction with respect to axial strain from SI 100-300 triaxial test

Figure 5.50 Variation of matric suction with respect to axial strain from SI 200-200 triaxial test

Figure 5.51 Variation of matric suction with respect to axial strain from SI 300-300 triaxial test

Figure 5.49 to **Figure 5.51** and **Figure 5.55** to **Figure 5.57** show that, during the shearing stage matric suction decreases and increases at the end of shearing stage. **Figure 5.49** to **Figure 5.51** also show that at the beginning of the infiltration stage matric suction decreases without the change in axial strain, then the rate of change in the axial strain with respect to matric suction increases exponentially. Failure is

deemed to occur when the rate of change in the axial strain with respect to matric suction increases rapidly.

5.5.4 Stress Path from Shearing-infiltration Tests

Three shearing-infiltration tests on compacted kaolin specimens had been conducted in this study. The stress paths of the shearing-infiltration stage of these three specimens and the failure envelope obtained from CD triaxial tests are shown in **Figure 5.52** to **Figure 5.54**. The stress paths of the shearing-infiltration stage of these three specimens and the failure envelope obtained from CD triaxial tests on q - s plane are also shown in **Figure 5.55**, **Figure 5.56** and **Figure 5.57**. **Figure 5.52**, **Figure 5.53**, **Figure 5.55** and **Figure 5.56** show that the stress path of specimens SI 100-300 and SI 200-200 reached the failure envelope. However, **Figure 5.54** and **Figure 5.57** show that stress path of specimen SI 300-300 passes above the failure envelope. **Figure 5.55** to **Figure 5.57** also show the variation of matric suction during the shearing stage (i.e., the reduction of matric suction during the shearing stage).

Figure 5.52 Failure envelope and stress path of shearing-infiltration stage of specimen SI 100-300

Figure 5.53 Failure envelope and stress path of shearing-infiltration stage of specimen SI 200-200

Figure 5.54 Failure envelope and stress path of shearing-infiltration stage of specimen SI 300-300

Figure 5.55 Failure envelope and stress path of shearing-infiltration stage of specimen SI 100-300 on the q - s plane

Figure 5.56 Failure envelope and stress path of shearing-infiltration stage of specimen SI 200-200 on the q - s plane

Figure 5.57 Failure envelope and stress path of shearing-infiltration stage of specimen SI 300-300 on the q - s plane

CHAPTER 6

DISCUSSION OF THE EXPERIMENTAL RESULTS AND COMPARISONS WITH PREDICTION FROM PROPOSED FORMULATIONS

6.1 Introduction

This chapter presents the discussion on the results that were obtained from this study (i.e., presented in Chapter 5). The comparisons between experimental results and proposed model predictions are also present and discussed.

6.2 Soil-Water Characteristics Curves

In this section, the experimental results of SWCC tests are discussed. The changes of matric suctions during the drying stage are discussed first. Then, the variations of air-entry value (*AEV*) and yield suction with respect to net mean stress are discussed.

6.2.1 Matric Suction Changes during Drying Stage

The variations of matric suction during drying stages are shown in Figure 5.6. Figure 5.6 shows that the rate of change in pore-water pressure varies with time (i.e., the transient process of water flow along the soil specimen). It could be explained as follows: at the beginning of each step of the drying stage water pressure in the water compartment is reduced to a required value. As a result, a hydraulic head gradient develops in the ceramic disk first and then in the soil specimen and water flows out of the soil specimen. Since water flows out of the soil specimen, the pore-water pressure and the volume of water in the soil specimen decrease, causing the hydraulic head gradient and the volumetric water content to decrease. The reduction of the volumetric water content leads to the reduction in the coefficient of permeability of the soil specimen. Therefore, the rate of change in pore-water pressure decreases because of the reduction in the hydraulic head gradient and in the permeability coefficient of the soil specimen. Figure 5.6 also shows that the rate of change in pore-water pressure is dependent on the matric suction at the beginning of each step. The higher the matric suction at the beginning of the step, the lower the rate of change in pore-water pressure. This is due to the

fact that the coefficient of permeability is dependent on the matric suction (i.e., permeability coefficient decreases with the decrease in matric suction).

Figure 5.6 also shows that the cavitation in the NTU mini suction probe takes place when matric suction increases from 370 kPa to 450 kPa. It could be due to the diffusion of air through the high air-entry ceramic disk. As a result, the reading from the NTU mini suction probe after the cavitation takes place is not the pore-water pressure any more.

6.2.2 Variations of Air-entry Value (AEV) and Yield Suction

The air-entry value increases with the increase in net mean stresses as shown in Figure 5.7 to Figure 5.10. It could be explained as follows: the soil specimen under the high net mean stress has a lower void ratio, e , and the pore size of the soil specimen under the high net mean stress is smaller. A smaller pore size leads to a higher surface tension. As a result, the air-entry of the soil specimen under a higher net mean stress is higher than the AEV of the soil specimen under the lower net mean stress.

Similar to the case of AEV , the yield suction, s_o , also increases with the increase in net mean stresses as shown in Figure 5.7 to 5.10. It could be explained as follows: the high net mean stress in the soil specimen leads to the low void ratio, e , (i.e., high density). In other words, the soil specimen is more compacted, and the soil specimen becomes harder resulting in a higher yield suction.

6.3 Isotropic Consolidation Tests

In this study, the isotropic consolidation tests were conducted under different matric suction, s , (i.e., 0 kPa, 30 kPa, 150 kPa and 300 kPa) on compacted kaolin specimens. The isotropic consolidation curves obtained from the isotropic consolidation tests are presented in Figure 5.17. The parameters obtained from the isotropic consolidation curves are presented in Figure 5.19 to Figure 5.21 and Table 5.3. Figure 5.17 shows that the compressibility of the soil specimen decreases with the increase in matric suction. The parameters obtained from isotropic consolidation curve also show the reduction in the compression index with the increase in matric suction. It could be attributed to the decreasing amount of water in the soil specimen with the increase in matric suction. As a result, the thickness of water

layer around the soil particle inside soil specimen reduces with the increase in matric suction. On the other hand, when matric suction was applied to the specimen, the void ratio, e , decreases (i.e., soil specimen becomes denser), the inter-force between soil particles increases, preventing the soil particles from movement and orientation into a denser configuration.

Figure 5.23 shows the variation of matric suction during the isotropic consolidation tests. During the loading stage, matric suction decreases first and then increases but its magnitude is still lower than the initial matric suction (i.e., the matric suction at beginning of the loading stage). During the unloading stage, pore-water pressure decreases monotonically. In other words, during the loading stage matric suction is lower than the initial matric suction and during the unloading stage matric suction is greater than the initial matric suction. During the loading stage, the soil specimen is compressed. As a result, the void volume of soil specimen decreases, and the degree of saturation, S , increases. The increment of degree of saturation leads to decrease in matric suction. Since the pore-air pressure is maintained at a constant value, the change in matric suction is due to the change in pore-water pressure, and water flows out of the soil specimen. In the opposite direction, void ratio increases during the unloading stage, and the degree of saturation, S , decreases. As a result, matric suction increases.

Figure 5.20b also shows the cavitation in the NTU mini suction probe that takes place when the net mean stress increases to 420 kPa. It could be due to the diffusion of air through the ceramic disk. As a result, the reading from NTU mini suction probe when the net mean stress is greater than 420 kPa is not the pore-water pressure any more.

6.4 Combination of Yield Stress and Predicted Yield Curve on the p - s Plane

In this section, the yield suctions obtained from SWCC tests and the yield stresses obtained from isotropic consolidation tests are discussed. The corresponding yield curve on the (p - s) plane at the end of the consolidation stage for the SWCC tests and at the end of matric suction equalization stage for the isotropic consolidation tests are predicted by using the proposed equations in this study (i.e., Equation 3.25 and Equation 3.29) and the equations proposed by Alonso et al. (1990).

The stress paths of SWCC tests on compacted kaolin specimens are shown in Figure 4.13. The isotropic consolidation test on compacted kaolin at matric suction of zero kPa (i.e., saturated condition) shows that the yield stress, $p_o(0)$, has a value of 25 kPa. Therefore, plastic strain developed during the consolidation stage of the SWCC-50, SWCC-100 and SWCC-250 tests. As a result, the yield suctions, s_o , obtained from SWCC tests do not belong to the same suction increase yield curve.

The stress paths for the isotropic consolidation tests on compacted kaolin specimens are shown in Figure 4.14. The SWCC test at net mean stress of 10 kPa (i.e., SWCC-10) gives the yield suction of 18 kPa. Therefore, plastic strains developed during the matric suction equalization stage of the IC-30, IC-150 and IC-300 tests. As a result, yield stresses, p_o , obtained from the isotropic consolidation tests do not belong to the same loading collapse yield curve.

The proposed equations for yield curves on (p - s) plane (i.e., suction increase yield curve and loading collapse yield curve) are used to simulate the corresponding yield curves at the end of consolidation stage of the SWCC tests. These yield curves are shown in **Figure 6.1** together with the yield suctions, s_o , obtained from SWCC tests. On the other hand, the yield curves on (p - s) plane proposed by Alonso et al. (1990) are also used to simulate the corresponding yield curve at the end of consolidation stage of the SWCC tests. These yield curves are shown in **Figure 6.2** together with the yield suction, s_o , obtained from SWCC tests.

Figure 6.1 Yield suctions obtained from SWCC tests and predicted yield curves based on the equation proposed in this study

Figure 6.2 Yield suctions obtained from SWCC tests and predicted yield curves based on the equation proposed by Alonso et al. (1990)

Similar to the SWCC tests, the yield curves at the end of matric suction stage of the isotropic consolidation tests are predicted by using the equations proposed in this study and the equations proposed by Alonso et al. (1990). The yield stresses obtained from isotropic consolidation tests and the predicted yield curves are shown in **Figure 6.3** and **Figure 6.4**, respectively.

Figure 6.5 shows the corresponding yield curves at the end of consolidation stage of the SWCC tests and the corresponding yield curves at the end of matric suction stage of the isotropic consolidation tests that were obtained from prediction using the equations proposed in this study, together with the yield suction obtained from SWCC tests and the yield stress obtained from isotropic consolidation tests.

Figure 6.6 shows the corresponding yield curves at the end of consolidation stage of the SWCC tests and the corresponding yield curves at the end of matric suction stage of the isotropic consolidation tests that were obtained from prediction using the equations proposed by Alonso et al. (1990), together with the yield suction obtained from SWCC tests and the yield stress obtained from isotropic consolidation tests.

Figure 6.1, Figure 6.3 and Figure 6.5 show that the predictions based on the equations proposed in this study are in closer agreement with the experimental data as compared with the predictions based on the Alonso et al. (1990) equations as shown in **Figure 6.2, Figure 6.4 and Figure 6.6**.

Figure 6.3 Yield suctions obtained from isotropic consolidation tests and predicted yield curves based on the equation proposed in this study

Figure 6.4 Yield suctions obtained from isotropic consolidation tests and predicted yield curves based on the equation proposed by Alonso et al. (1990)

Measured(1) – Measured from SWCC tests,

Measured(2) – Measured from isotropic consolidation tests.

Figure 6.5 Yield stresses obtained from isotropic consolidation tests, yield suctions obtained from isotropic consolidation tests and predicted yield curves based on the equations proposed in this study

Measured(1) – Measured from SWCC tests,

Measured(2) – Measured from isotropic consolidation tests.

Figure 6.6 Yield stresses obtained from isotropic consolidation tests, yield suctions obtained from isotropic consolidation tests and predicted yield curves based on the equations proposed by Alonso et al. (1990)

6.5 Results of Triaxial Shearing Tests

In this section, the results of consolidation drained tests are discussed first. Then, the results from the shearing-infiltration tests are discussed.

6.5.1 Consolidated Drained Triaxial Tests

The variations of deviator stress, q , with respect to axial strain during the shearing stage under consolidated drained condition are shown in Figure 5.24 to Figure 5.28. Figure 5.27 and Figure 5.28 show that under the same net confining stress, the peak deviator stress increases with the increase in matric suction. In other words, the shear strength increases with the increase in matric suction. It can be seen from Figure 5.37, the axial strain increases to 6% when the deviator stress, q , reaches around 90% of the peak deviator stress. After that, the deviator stresses increases slowly to reach the peak deviator stress and then decreases slowly, except for the case of CD 100-100 and CD 100-200 triaxial tests. The rapidly decrease in deviator stress with respect to axial strain for the CD 100-100 and CD 100-200 could be due to the brittle failure mode occurred in those specimens.

Figure 6.7 Variation of matric suction with respect to axial strain during shearing stage of specimens CD 100-100 and CD 300-100

Figure 6.8 Variation of matric suction with respect to axial strain during shearing stage of specimens CD 100-200 and CD 300-200

The variations of matric suction with respect to axial strain during the shearing stage of specimens tested under an initial matric suction of 100 kPa and at different net confining stresses are shown in **Figure 6.7**. It could be seen from **Figure 6.7** that matric suction decreases up to 3 kPa when the axial strain increases to 0.8% and afterward matric suction increases and finally reaches the initial matric suction at the end of shearing stage. Similar to **Figure 6.7**, **Figure 6.8** shows the variation of matric suction with respect to axial strain during the shearing stage of specimens tested under an initial matric suction of 200 kPa and at different net confining stresses. Matric suction also decreases up to 6 kPa when the axial strain increases to 0.8% and afterward matric suction increases and finally reaches the initial matric suction at the end of shearing stage. The variations of matric suction during the shearing stage are similar to the variation of matric suction during the loading stage of isotropic consolidation tests. Therefore, the variations of volumetric strain during the shearing stage should be the same as variations of volumetric strain during the loading stage of isotropic consolidation tests.

Figure 5.32 and Figure 5.33 present the water volumetric strain of specimens CD 100-100, CD 100-200, CD 300-100 and CD 300-200 during the shearing stage. It can be seen that water volumetric strain decreases during the shearing stage. The

variations of water volumetric strain agree well with the variations of matric suction during the shearing stage.

The results from isotropic consolidation tests show the decrease in water volume when the soil volume decreases and vice versa. Figure 5.35 shows water volumetric strain of specimens CD 100-100 and CD 100-200 that continues to decrease and reaches a constant value when axial strain increases to a value greater than 6%. The matric suctions are smaller than the initial matric suction when the axial strain increase is greater than 6%. It can be said that the pore size is reduced when the axial strain becomes higher than 6%.

Figure 5.33 shows that the specimens tested under the same net confining stress of 300 kPa and at different matric suctions compress and reach a constant volume (i.e., reach the critical state). Figure 5.32 shows that the specimens tested under the same net confining stress of 100 kPa and at different matric suctions compress first until the axial strain reaches to around 4% then remain constant until the axial strain reaches 6% and then dilates afterwards.

The stress paths of consolidated drained triaxial tests are shown in Figure 4.15. The results of the SWCC tests show that plastic strain occurred during the consolidation stage and the matric suction equalization stage. As a result, the soil specimens at the beginning of the shearing stage were normally consolidated. Therefore, the behaviour of soil specimens during the shearing stage could resemble that of the normally consolidated soil behavior. The results from specimens tested under a net confining stress of 300 kPa and at different matric suctions show good agreement with the behavior of normally consolidated soil. However, there is a small difference with the normally consolidated soil behavior for the specimens tested under a net confining stress of 100 kPa and at different matric suctions. The difference is that the soil specimens dilate when the axial strain becomes greater than 6%. The reason for this difference can be explained as follows: when the axial strain reaches 6%, the deviator stress reaches around 90% of the peak deviator stress. Due to the low net confining stress, the movement of soil particle could lead to the opening of micro cracks that start to develop inside the specimen. As a result, the total volume of the specimen increases.

For the case of CD 100-100 test and CD 100-200 test, it can be said that the volumetric strain versus axial strain curve are valid only before the axial strain reaches 6% because of the pore size continues to decrease during the shearing stage and the opening of micro cracks continues to develop inside the specimen. In general, the opening and development of micro cracks occur inside the specimen tested under a low net confining stress when the deviator stress increases to a magnitude greater than 90% of the peak deviator stress.

6.5.2 Shearing-infiltration Tests

The variations of matric suction during the shearing stage of SI 100-300 and SI 300-300 tests are shown in **Figure 6.9**. It can be seen that matric suction decreases to 6 kPa when the axial strain increases to 0.8%. Afterward, matric suction increases but its value is still smaller than the applied matric suction (i.e., matric suction at the initial of shearing stage). The variations of matric suction during the shearing stage of SI 100-300 and SI 300-300 tests have the same form with the variations of matric suction during the shearing stage of CD 100-200 and CD 300-200 tests. **Figure 6.7**, **Figure 6.8** and **Figure 6.9** illustrate that the variation of matric suction during shearing stage does not seem to depend on net confining stress.

Figure 6.9 Variation of matric suction with respect to axial strain during shearing stage of specimens SI 100-300 and SI 300-300

Since, the matric suctions at the end of shearing stage of shearing infiltration test are smaller than the applied matric suction as shown in Figure 5.49 to Figure 5.51, Figure 5.55 to Figure 5.57 and **Figure 6.9**, matric suction increases to the applied matric suction. Therefore, volumetric strain occurs due to the increase in matric suction. On the other hand, the cell pressure fluctuated around the applied pressure due to the change in temperature. As a result, the friction force between the load ram and the triaxial cell is changed, causing a reduction in deviator stress between the shearing stage and the infiltration stage.

The stress paths of specimens SI 100-300 and SI 200-200 reach the failure envelope and the failure points are also close to the failure envelope as shown in Figure 5.55 and Figure 5.56. However, the stress path of specimen SI 300-300 exceeds the failure envelope and the failure point lies above the failure envelope. This could be due to the interruption of the electricity in the laboratory.

6.6 Comparison of Proposed Model Predictions with the Consolidated Drained Triaxial Test Results

In this section, the proposed model presented in Chapter 3 is used to predict the shear strength and volumetric strain during shearing stage of the consolidated drained triaxial tests. The CD triaxial tests conducted in this study are predicted first. Then, the CD triaxial tests obtained from the literature are predicted.

6.6.1 Comparison of Proposed Model Predictions with the Results from CD Triaxial Tests Conducted in this Study

The predicted and measured results of shear strength are presented in the same figure for comparison. Similar to shear strength, predicted volumetric strain and measured volumetric strain also presented in the same figure. The comparison between predicted and measured results for CD tests conducted in this study at a net confining stress of 300 kPa and different matric suctions (i.e., CD300-0, CD300-100 and CD300-200) are presented in this section. The comparison between predicted and measured results for CD tests conducted at a net confining stress of 100 kPa and different matric suctions (i.e., CD100-0, CD100-100 and CD100-200) are presented in Appendix B.

Figure 6.16 shows the yield curve on (p - s) plane (i.e., loading-collapse yield curve and suction increase yield curve) at the beginning of the shearing stage and at the end of shearing stage together with the projection of yield path on the (p - s) plane during shearing stage for CD 300-100 test. The yield curves and failure envelope on the constant s/p plane at the beginning of the shearing stage and at the end of shearing stage for CD 300-100 test are shown in **Figure 6.17** and **Figure 6.18**, respectively. Similar yield curves on (p - s) plane, s/p =constant plane and the projection of the yield path on (p - s) plane for CD100-100, CD 100-200 and CD 300-200 are presented in Appendix B.

The comparison between the predicted and measured results of shear strength during the shearing stage of specimens CD300-0, CD300-100 and CD300-200 are presented in **Figure 6.10**, **Figure 6.12** and **Figure 6.14**, respectively. The predicted shear strengths during the shearing stage of CD tests show a good agreement with the experimental results.

Figure 6.11, Figure 6.13 and Figure 6.15 show the comparison between the predicted and measured results of volumetric strain during the shearing stage of specimens CD300-0, CD300-100 and CD300-200, respectively. The predicted volumetric strain during the shearing stage in the CD tests at net mean stress of 300 kPa (i.e., CD300-0, CD300-100 and CD300-200) shows a good agreement with the experimental results.

The comparison between the predicted and measured results of volumetric strain during the shearing stage of specimens CD100-0, CD100-100 and CD100-200 as presented in Figure B.2, Figure B.4 and Figure B.6 show a good agreement when the axial strain is smaller than 6%. There are discrepancies between the predicted and measured results of volumetric strain during the shearing stage of these specimens when the axial strain is higher than 6%. However, the measured volumetric strain versus axial strain for these specimens are valid only for axial strain before reaches 6% as discussed in Section 6.5.1. Therefore, it can be said that the predicted volumetric strain during the shearing in the CD tests at net mean stress of 100 kPa (i.e., CD100-0, CD100-100 and CD100-200) shows a good agreement with the experimental results.

Figure 6.10 Variation of deviator stresses with respect to axial strain for CD300-0 test

Figure 6.11 Variation of volumetric strain with respect to axial strain for CD300-0 test

Figure 6.12 Variation of deviator stresses with respect to axial strain for CD300-100 test

Figure 6.13 Variation of volumetric strain with respect to axial strain for CD300-100 test

Figure 6.14 Variation of deviator stresses with respect to axial strain for CD300-200 test

Figure 6.15 Variation of volumetric strain with respect to axial strain for CD300-200 test

Figure 6.16 Yield curve at the beginning of the shearing stage and at the end of shearing stage together with the projection of yield path on the $(p-s)$ plane during the shearing stage for CD 300-100 test

Figure 6.17 Yield curve and failure envelope at the beginning of the shearing stage on the $s/p = 1/3$ plane for CD 300-100 test

Figure 6.18 Yield curve and failure envelope at the end of the shearing stage on the $s/p = 0.19$ plane for CD 300-100 test

6.6.2 Comparison of Proposed Model Predictions with the Results of CD Triaxial Tests from Literature

Similar to the previous section, the predicted and measured results of shear strength are presented in the same figure for comparison. Predicted volumetric strain and measured volumetric strain are also presented in the same figure. The comparison between predicted results of the proposed model presented in Chapter 3 and measured results for CD tests conducted at a net confining stress of 300 kPa and at different matric suctions (i.e., CD300-0 (Thu 2005), CD300-100 (Thu 2005) and CD300-200 (Thu 2005)) as conducted by Thu (2005) are presented in this section. The comparison between the predicted results of the proposed model presented in Chapter 3 and measured results for CD tests conducted at net confining stress of 100 kPa and 200 kPa and at different matric suctions (i.e., CD100-0 (Thu 2005), CD100-100 (Thu 2005), CD100-200 (Thu 2005), CD200-0 (Thu 2005), CD200-100 (Thu 2005) and CD200-200 (Thu 2005)) are presented in Appendix B.

The comparison between the predicted and measured results of shear strength during the shearing stage of specimens CD300-0 (Thu 2005), CD300-100 (Thu 2005), CD300-200 (Thu 2005) and CD300-300 (Thu 2005) are presented in **Figure 6.19**, **Figure 6.21**, **Figure 6.23** and **Figure 6.25**, respectively. The predicted shear strength as obtained from the proposed model presented in Chapter 3 during the shearing of CD tests conducted by Thu (2005) shows a good agreement with the experimental results.

Figure 6.20, **Figure 6.22**, **Figure 6.24** and **Figure 6.26** show the comparison between predicted and measured results of volumetric strain during the shearing stage of specimens CD300-0 (Thu 2005), CD300-100 (Thu 2005), CD300-200 (Thu 2005) and CD300-300 (Thu 2005), respectively. The predicted volumetric strain during the shearing of CD tests shows an agreement with the experimental results.

Figure 6.19 Variation of deviator stresses with respect to axial strain for CD300-0 (Thu 2005) test

Figure 6.20 Variation of volumetric strain with respect to axial strain for CD300-0 (Thu 2005) test

Figure 6.21 Variation of deviator stresses with respect to axial strain for CD300-100 (Thu 2005) test

Figure 6.22 Variation of volumetric strain with respect to axial strain for CD300-100 (Thu 2005) test

Figure 6.23 Variation of deviator stresses with respect to axial strain for CD300-200 (Thu 2005) test

Figure 6.24 Variation of volumetric strain with respect to axial strain for CD300-200 (Thu 2005) test

Figure 6.25 Variation of deviator stresses with respect to axial strain for CD300-300 (Thu 2005) test

Figure 6.26 Variation of volumetric strain with respect to axial strain for CD300-300 (Thu 2005) test

6.7 Comparison of Proposed Model Predictions with the Shearing-infiltration Tests

In this section, the proposed model presented in Chapter 3 is used to predict the shear strength and volumetric strain during the shearing stage of shearing-infiltration tests. The shearing-infiltration tests conducted in this study are predicted first. Then, the shearing-infiltration tests obtained from the literature are predicted.

6.7.1 Comparison of Proposed Model Predictions with the Results from Shearing-infiltration Tests Conducted in this Study

The deviator stress, q , versus axial strain, ε_a , obtained from shearing-infiltration tests (i.e., experimental results) and prediction results are presented in the same figure for comparison. Similar to the deviator stress, matric suction, s , versus axial strain, ε_a , and volumetric strain, ε_v , versus axial strain, ε_a , are also presented in the same figure.

The variation of deviator stress, q , with respect to axial strain, ε_a , obtained from shearing-infiltration tests (i.e., SI200-200, SI100-300 and SI300-300) and the predicted results are presented in **Figure 6.27**, **Figure 6.30** and **Figure 6.33**, respectively. The predicted deviator stress, q , during the shearing-infiltration tests shows a good agreement with the experimental results.

Figure 6.29, Figure 6.32 and Figure 6.35 present the variation of volumetric strain, ε_v , with respect to axial strain, ε_a , as obtained from the prediction and shearing-infiltration tests. The predicted volumetric strain, ε_v , during the shearing-infiltration tests shows a good agreement with the experimental results when the axial strain is smaller than 5%. There are discrepancies between the predicted and measured results of volumetric strain of these specimens when the axial strain is higher than 5%. Examination of Figure 6.29, Figure 6.32 and Figure 6.35 reveals that the discrepancies between the predicted and measured results of volumetric strain depend on net confining stress. Based on the discussion in Section 6.5.1, it can be deduced that the discrepancies between the predicted and measured results of volumetric strain could be attributed to the development of open cracks in the soil specimens.

The variation of matric suction, s , with respect to axial strain, ε_a , obtained from the shearing-infiltration tests and the predicted ones are presented in **Figure 6.28**, **Figure 6.31** and **Figure 6.34**, respectively. **Figure 6.28**, **Figure 6.31** and **Figure 6.34** show that the axial strain starts to increase rapidly at the same matric suction as observed in the experiment and predicted by the proposed model. In other words, failure occurs at the same matric suction during the experiment and as predicted by the proposed model.

Figure 6.36 shows the yield curve on (p - s) plane (i.e., loading-collapse yield curve and suction increase yield curve) at the beginning of the shearing stage, at the end of the shearing stage and at the end of the infiltration stage together with the projection of yield paths on the (p - s) plane during shearing stage and infiltration stage for SI 200-200 test. The yield curves and failure envelope on the constant s/p plane at the beginning of the shearing stage, at the end of the shearing stage and at the end of the infiltration stage for SI 200-200 test are shown in **Figure 6.37**,

Figure 6.38 and **Figure 6.39**, respectively. Similar yield curves on $(p-s)$ plane, $s/p=\text{constant}$ plane and the projection of the yield paths on $(p-s)$ plane for SI 100-300 and SI 300-300 are presented in Appendix B.

Figure 6.27 Variation of deviator stresses with respect to axial strain for SI200-200 test

Figure 6.28 Variation of matric suction with respect to axial strain for SI200-200 test

Figure 6.29 Variation of volumetric strain with respect to axial strain for SI200-200 test

Figure 6.30 Variation of deviator stresses with respect to axial strain for SI100-300 test

Figure 6.31 Variation of matric suction with respect to axial strain for SI100-300 test

Figure 6.32 Variation of volumetric strain with respect to axial strain for SI100-300 test

Figure 6.33 Variation of deviator stresses with respect to axial strain for SI300-300 test

Figure 6.34 Variation of matric suction with respect to axial strain for SI300-300 test

Figure 6.35 Variation of volumetric strain with respect to axial strain for SI300-300 test

Figure 6.36 Yield curve at the beginning of the shearing stage, at the end of the shearing stage and at the end of infiltration stage together with the projection of yield paths on the $(p-s)$ plane during the shearing stage and infiltration stage for SI 200-200 test

Figure 6.37 Yield curve and failure envelope at the beginning of the shearing stage on the $s/p = 1$ plane for SI 200-200 test

Figure 6.38 Yield curve and failure envelope at the end of the shearing stage on the $s/p = 0.55$ plane for SI 200-200 test

Figure 6.39 Yield curve and failure envelope at the end of the infiltration stage on the $s/p = 0.07$ plane for SI 200-200 test

6.7.2 Comparison of Proposed Model Predictions with the Results of Shearing-infiltration Tests from Literature

Similar to the previous section, the deviator stress, q , versus axial strain, ε_a , obtained from shearing-infiltration tests (i.e., experimental results) and predicted ones are also presented in the same figure for comparison. Similar to the deviator stress, matric suction, s , versus axial strain, ε_a , and volumetric strain, ε_v , versus axial strain, ε_a , are also presented in the same figure. The comparison between the predicted values and the measurement results for shearing-infiltration tests conducted at matric suction of 100 kPa and at different net confining stresses (i.e., SI25-100 (Meilani 2004), SI100-100 (Meilani 2004) and SI200-100 (Meilani 2004)) by Meilani (2004) are presented in this section. The comparison between predicted and measured results for shearing-infiltration tests conducted at matric suction of 50 kPa and 200 kPa and at different net confining stresses (i.e., SI25-50 (Meilani 2004), SI100-50 (Meilani 2004), SI200-50 (Meilani 2004), SI25-200 (Meilani 2004),

SI100-200 (Thu Meilani 2004) and SI200-200 (Meilani 2004)) by Meilani (2004) are presented in Appendix B.

The variation of deviator stress, q , with respect to axial strain, ε_a , obtained from shearing-infiltration tests (i.e., SI25-100 (Meilani 2004), SI100-100 (Meilani 2004) and SI200-100 (Meilani 2004)) and the predicted ones are presented in Figure 6.40, Figure 6.43 and Figure 6.46, respectively. The predicted deviator stresses, q , during the shearing-infiltration tests show a good agreement with the experimental results. Figure 6.42, Figure 6.45 and Figure 6.48 present the variation of volumetric strain, ε_v , with respect to axial strain, ε_a , as obtained from prediction and shearing-infiltration tests. The predicted volumetric strain, ε_v , during the shearing-infiltration tests shows an agreement with the experimental results.

The variation of matric suction, s , with respect to axial strain, ε_a , as obtained from shearing-infiltration tests and the predicted one are presented in **Figure 6.41**, **Figure 6.44** and **Figure 6.47**, respectively. **Figure 6.41**, **Figure 6.44** and **Figure 6.47** show that the axial strain starts to increase gradually at almost the same matric suction as observed in the experiment and predicted by the proposed model. In other words, failure occurs at almost the same matric suction during the experiment and as predicted by the proposed model.

Figure 6.40 Variation of deviator stresses with respect to axial strain for SI25-100 (Meilani 2004) test

Figure 6.41 Variation of matric suction with respect to axial strain for SI25-100 (Meilani 2004) test

Figure 6.42 Variation of volumetric strain with respect to axial strain for SI25-100 (Meilani 2004) test

Figure 6.43 Variation of deviator stresses with respect to axial strain for SI100-100 (Meilani 2004) test

Figure 6.44 Variation of matric suction with respect to axial strain for SI100-100 (Meilani 2004) test

Figure 6.45 Variation of volumetric strain with respect to axial strain for SI100-100 (Meilani 2004) test

Figure 6.46 Variation of deviator stresses with respect to axial strain for SI200-100 (Meilani 2004) test

Figure 6.47 Variation of matric suction with respect to axial strain for SI200-100 (Meilani 2004) test

Figure 6.48 Variation of volumetric strain with respect to axial strain for SI100-100 (Meilani 2004) test

CHAPTER 7

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

This chapter presents the summary and the conclusions that can be drawn from this study. After that, the recommendations for future study are also presented.

7.1 Summary

The works have been accomplished in this research can be summarized as follows:

1. The development of elasto-plastic constitutive relations in this study was based on the general framework proposed by Allonso et al. (1987, 1990), Wheeler and Sivakumar (1995), Wheeler (1996). An improved formulation for the suction increase yield curve was developed where the yield curves in (q-p-s) space were defined in radial direction and the current SWCC was introduced. In addition, the variation of water phase constitutive equation was also proposed in this study.
2. The coupled formulations developed in this study were based on the approach used by Dakshanamurthy et al.'s (1984) and Fredlund and Rahardjo's (1993). The proposed coupled formulations consider the change in specific water volume as a result of the change in net normal stress, deviator stress and matric suction. The influence of hysteresis is also taken into account in the coupled formulations proposed in this study.
3. The results from SWCC tests on compacted soil specimens show that the yield suction, s_o , obtained from specific volume, v , versus matric suction, s , curve (i.e., isotropic suction consolidation curve) increases with the increase in net mean stress. These yield suctions do not belong to the same yield curve. Dissimilar to yield suction, the slopes of the isotropic suction consolidation curve (i.e., slope of the unloading/reloading curve and slope of the virgin curve) decrease with the increase in net mean stress.
4. The SWCCs of compacted soil show that the air-entry value increases with the increase in net mean stress.

5. The results from isotropic consolidation tests on compacted soil specimens show that yield stress, p_o , obtained from specific volume, v , versus net mean stress, p , curve, increases with the increase in matric suction, s . Similar to the yield suction, these yield stresses do not belong to the same yield curve. The slopes of isotropic consolidation curve decrease with the increase in matric suction.
6. During the loading stage of the isotropic consolidation tests, the specific volume of compacted soil specimens decreased, matric suction was smaller than the initially applied matric suction and water flowed out of the specimens. During the unloading stage of the isotropic consolidation tests, the specific volume of compacted soil specimens increased, matric suction was higher than the initially applied matric suction and water flowed into the specimens.
7. The prediction of yield curve on (p-s) plane (i.e., SI yield curve and LC yield curve) based on the proposed equations developed in this study show a closer agreement with the experimental data as compared with the predictions based on Alonso et al. (1990) equations.
8. Results from the consolidated drained triaxial tests show that matric suction during shearing was smaller than the initially applied matric suction. This was in agreement with the variation of water volume change during the shearing stage (i.e., water volume change decreased throughout the shearing stage). The behaviour of compacted soil specimens is similar to the normal consolidation soil. In other words, the total volume change of specimens should decrease throughout the shearing stage.
9. Results from the shearing infiltration tests show that most stress paths of the shearing-infiltration tests reached the failure envelope and the failure points were also close to the failure envelope. The occurrence of stress path that exceeded the failure envelope and the failure point being located above the failure envelope could be due to the interruption of electricity in the laboratory during these particular tests.

10. The predicted shear strength and total volume change characteristics under the consolidated drained triaxial condition as obtained from the elasto-plastic constitutive relations proposed in this study show that the predicted and the measured shear strengths were in a good agreement and the predicted total volume change was partly in agreement with the measured volume change.
11. The predicted shear strength, total volume change and matric suction change characteristics under the shearing-infiltration condition as obtained from the elasto-plastic constitutive relations proposed in this study show that the predicted and measured shear strengths were in a good agreement and the predicted volume change was partly in agreement with the measured volume change. In addition, the measured matric suction change and the predicted matric suction change started to increase rapidly at the same matric suction (i.e., failure occurred at the same matric suction during the experiment and the simulation using the proposed elasto-plastic constitutive relations).

7.2 Conclusion

In this research, an elasto-plastic model for unsaturated soil has been developed and verified using the experimental results from consolidated drained and shearing infiltration tests (i.e., with reasonably close agreement). The model is now ready to be used for solving geotechnical problems involving unsaturated soils.

7.3 Recommendations

1. The NTU mini suction probe for the measurement of pore-water pressure in soil specimen has been shown to provide useful information on the behaviour of unsaturated soil during the tests. However, it would be useful to improve NTU mini suction probe in order to measure higher suction values over a long period of testing in order to be used for tests involving high matric suction.
2. It is worth to develop equipments and incorporate them into triaxial apparatus to measure the total volume change and to investigate the development of the micro cracks in soil specimen during the test.

3. The characteristic of yield curve on the (p-s) plane (i.e., the SI yield curve and the LC yield curve) should be investigated using the independent tests and for other types of soil.
4. The elasto-plastic constitutive relations should be improved in order to have better agreement with the experiment results, especially on the prediction of volumetric strain.
5. The numerical solution of coupled formulation could be developed for analyzing complex problems involving unsaturated soils with similar properties as Kaolin used in this study.

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APPENDIX A

Calibration data of modified triaxial apparatus

Table A.1 Calibration data of pore-air pressure transducer

Pressure (kPa)	Transducer reading (V)
9.469	5.0852E-02
49.063	5.2052E-02
99.063	5.3568E-02
149.906	5.5110E-02
199.094	5.6609E-02
249.188	5.8128E-02
299.188	5.9640E-02
349.438	6.1161E-02
399.406	6.2677E-02
450.563	6.4238E-02
499.656	6.5726E-02
549.5	6.7240E-02
600.563	6.8794E-02
649.25	7.0271E-02

Figure A.1 Calibration of pore-air pressure transducer

Table A.2 Calibration data of pore-water pressure transducer

Pressure (kPa)	Transducer reading (V)
9.469	5.0822E-02
49.063	5.2791E-02
99.063	5.5281E-02
149.906	5.7805E-02
199.094	6.0242E-02
249.188	6.2736E-02
299.188	6.5213E-02
349.438	6.7702E-02
399.406	7.0189E-02
450.563	7.2739E-02
499.656	7.5179E-02
549.5	7.7662E-02
600.563	8.0214E-02
649.25	8.2642E-02

Figure A.2 Calibration of pore-water pressure transducer

Table A.3 Calibration data of “Top” NTU mini suction probe

Pressure (kPa)	Transducer reading (V)
9.469	5.0668E-02
49.063	5.2625E-02
99.063	5.5103E-02
149.906	5.7617E-02
199.094	6.0062E-02
249.188	6.2540E-02
299.188	6.5020E-02
349.438	6.7513E-02
399.406	6.9992E-02
450.563	7.2545E-02
499.656	7.4981E-02
549.5	7.7464E-02
600.563	8.0000E-02
649.25	8.2416E-02

Figure A.3 Calibration of “Top” NTU mini suction probe

Table A.4 Calibration data of “Bottom” NTU mini suction probe

Pressure (kPa)	Transducer reading (V)
9.469	3.4685E-02
49.063	3.6644E-02
99.063	3.9116E-02
149.906	4.1628E-02
199.094	4.4065E-02
249.188	4.6548E-02
299.188	4.9019E-02
349.438	5.1499E-02
399.406	5.3979E-02
450.563	5.6539E-02
499.656	5.8975E-02
549.5	6.1434E-02
600.563	6.3983E-02
649.25	6.6398E-02

Figure A.4 Calibration of “Bottom” NTU mini suction probe

Table A.5 Calibration data of LVDT

Micrometer reading (mm)	LVDT reading (V)
0	5.0486E-02
5	5.7884E-02
10	6.5182E-02
15	7.2494E-02
20	7.9772E-02
25	8.7046E-02

Figure A.5 Calibration of LVDT

Table A.6 Calibration data of load cell

Load (N)	Load cell reading (V)
0	5.0046E-02
139.14	5.0611E-02
277.59	5.1154E-02
414.38	5.1698E-02
553.66	5.2262E-02
688.8	5.2813E-02
827.25	5.3352E-02
964.32	5.3904E-02
1100.7	5.4457E-02
1239.84	5.5005E-02
1376.91	5.5543E-02
1518.12	5.6110E-02
1653.12	5.6633E-02
1791.98	5.7175E-02

Figure A.6 Calibration of load cell

Table A.7 Calibration data of thermometer

Temperature (°C)	Thermometer reading (V)
20.3	5.5416E-01
21.8	5.5444E-01
22.2	5.5449E-01
22.8	5.5466E-01
23.3	5.5471E-01
25	5.5503E-01
26.2	5.5525E-01
27.7	5.5553E-01
29	5.5574E-01
30.6	5.5602E-01
31.2	5.5616E-01

Figure A.7 Calibration of thermometer

APPENDIX B

Prediction of the consolidated drained triaxial tests and shearing-infiltration tests using the proposed model presented in Chapter 3

Figure B.1 Variation of deviator stresses with respect to axial strain for CD100-0 test

Figure B.2 Variation of volumetric strain with respect to axial strain for CD100-0 test

Figure B.3 Variation of deviator stresses with respect to axial strain for CD100-100 test

Figure B.4 Variation of volumetric strain with respect to axial strain for CD100-100 test

Figure B.5 Variation of deviator stresses with respect to axial strain for CD100-200 test

Figure B.6 Variation of volumetric strain with respect to axial strain for CD100-200 test

Figure B.7 Variation of deviator stresses with respect to axial strain for CD300-0 test

Figure B.8 Variation of volumetric strain with respect to axial strain for CD300-0 test

Figure B.9 Variation of deviator stresses with respect to axial strain for CD300-100 test

Figure B.10 Variation of volumetric strain with respect to axial strain for CD300-100 test

Figure B.11 Variation of deviator stresses with respect to axial strain for CD300-200 test

Figure B.12 Variation of volumetric strain with respect to axial strain for CD300-200 test

Figure B.13 Yield curve at the beginning of the shearing stage and at the end of shearing stage together with the projection of yield path on the $(p-s)$ plane during the shearing stage for CD 100-100 test

Figure B.14 Yield curve and failure envelope at the beginning of the shearing stage on the $s/p = 1$ plane for CD 100-100 test

Figure B.15 Yield curve and failure envelope at the end of the shearing stage on the $s/p = 0.44$ plane for CD 100-100 test

Figure B.16 Yield curve at the beginning of the shearing stage and at the end of shearing stage together with the projection of yield path on the $(p-s)$ plane during the shearing stage for CD 100-200 test

Figure B.17 Yield curve and failure envelope at the beginning of the shearing stage on the $s/p = 2$ plane for CD 100-200 test

Figure B.18 Yield curve and failure envelope at the end of the shearing stage on the $s/p = 0.75$ plane for CD 100-200 test

Figure B.19 Yield curve at the beginning of the shearing stage and at the end of shearing stage together with the projection of yield path on the $(p-s)$ plane during the shearing stage for CD 300-100 test

Figure B.20 Yield curve and failure envelope at the beginning of the shearing stage on the $s/p = 1/3$ plane for CD 300-100 test

Figure B.21 Yield curve and failure envelope at the end of the shearing stage on the $s/p = 0.19$ plane for CD 300-100 test

Figure B.22 Yield curve at the beginning of the shearing stage and at the end of shearing stage together with the projection of yield path on the $(p-s)$ plane during the shearing stage for CD 300-200 test

Figure B.23 Yield curve and failure envelope at the beginning of the shearing stage on the $s/p = 2/3$ plane for CD 300-200 test

Figure B.24 Yield curve and failure envelope at the end of the shearing stage on the $s/p = 0.35$ plane for CD 300-200 test

Figure B.25 Variation of deviator stresses with respect to axial strain for CD100-0 (Thu 2005) test

Figure B.26 Variation of volumetric strain with respect to axial strain for CD100-0 (Thu 2005) test

Figure B.27 Variation of deviator stresses with respect to axial strain for CD100-100 (Thu 2005) test

Figure B.28 Variation of volumetric strain with respect to axial strain for CD100-100 (Thu 2005) test

Figure B.29 Variation of deviator stresses with respect to axial strain for CD100-200 (Thu 2005) test

Figure B.30 Variation of volumetric strain with respect to axial strain for CD100-200 (Thu 2005) test

Figure B.31 Variation of deviator stresses with respect to axial strain for CD100-300 (Thu 2005) test

Figure B.32 Variation of volumetric strain with respect to axial strain for CD100-300 (Thu 2005) test

Figure B.33 Variation of deviator stresses with respect to axial strain for CD200-0 (Thu 2005) test

Figure B.34 Variation of volumetric strain with respect to axial strain for CD200-0 (Thu 2005) test

Figure B.35 Variation of deviator stresses with respect to axial strain for CD200-100 (Thu 2005) test

Figure B.36 Variation of volumetric strain with respect to axial strain for CD200-100 (Thu 2005) test

Figure B.37 Variation of deviator stresses with respect to axial strain for CD200-200 (Thu 2005) test

Figure B.38 Variation of volumetric strain with respect to axial strain for CD200-200 (Thu 2005) test

Figure B.39 Variation of deviator stresses with respect to axial strain for CD200-300 (Thu 2005) test

Figure B.40 Variation of volumetric strain with respect to axial strain for CD200-300 (Thu 2005) test

Figure B.41 Variation of deviator stresses with respect to axial strain for CD300-0 (Thu 2005) test

Figure B.42 Variation of volumetric strain with respect to axial strain for CD300-0 (Thu 2005) test

Figure B.43 Variation of deviator stresses with respect to axial strain for CD300-100 (Thu 2005) test

Figure B.44 Variation of volumetric strain with respect to axial strain for CD300-100 (Thu 2005) test

Figure B.45 Variation of deviator stresses with respect to axial strain for CD300-200 (Thu 2005) test

Figure B.46 Variation of volumetric strain with respect to axial strain for CD300-200 (Thu 2005) test

Figure B.47 Variation of deviator stresses with respect to axial strain for CD300-300 (Thu 2005) test

Figure B.48 Variation of volumetric strain with respect to axial strain for CD300-300 (Thu 2005) test

Figure B.49 Variation of deviator stresses with respect to axial strain for SI200-200 test

Figure B.50 Variation of matric suction with respect to axial strain for SI200-200 test

Figure B.51 Variation of volumetric strain with respect to axial strain for SI200-200 test

Figure B.52 Variation of deviator stresses with respect to axial strain for SI100-300 test

Figure B.53 Variation of matric suction with respect to axial strain for SI100-300 test

Figure B.54 Variation of volumetric strain with respect to axial strain for SI100-300 test

Figure B.55 Variation of deviator stresses with respect to axial strain for SI300-300 test

Figure B.56 Variation of matric suction with respect to axial strain for SI300-300 test

Figure B.57 Variation of volumetric strain with respect to axial strain for SI300-300 test

Figure B.58 Yield curve at the beginning of the shearing stage, at the end of the shearing stage and at the end of infiltration stage together with the projection of yield paths on the $(p-s)$ plane during the shearing stage and infiltration stage for SI 200-200 test

Figure B.59 Yield curve and failure envelope at the beginning of the shearing stage on the $s/p = 1$ plane for SI 200-200 test

Figure B.60 Yield curve and failure envelope at the end of the shearing stage on the $s/p = 0.55$ plane for SI 200-200 test

Figure B.61 Yield curve and failure envelope at the end of the infiltration stage on the $s/p = 0.07$ plane for SI 200-200 test

Figure B.62 Yield curve at the beginning of the shearing stage, at the end of the shearing stage and at the end of infiltration stage together with the projection of yield paths on the $(p-s)$ plane during the shearing stage and infiltration stage for SI 100-300 test

Figure B.63 Yield curve and failure envelope at the beginning of the shearing stage on the $s/p = 3$ plane for SI 100-300 test

Figure B.64 Yield curve and failure envelope at the end of the shearing stage on the $s/p = 1.36$ plane for SI 100-300 test

Figure B.65 Yield curve and failure envelope at the end of the infiltration stage on the $s/p = 0.38$ plane for SI 100-300 test

Figure B.66 Yield curve at the beginning of the shearing stage, at the end of the shearing stage and at the end of infiltration stage together with the projection of yield paths on the $(p-s)$ plane during the shearing stage and infiltration stage for SI 300-300 test

Figure B.67 Yield curve and failure envelope at the beginning of the shearing stage on the $s/p = 1$ plane for SI 300-300 test

Figure B.68 Yield curve and failure envelope at the end of the shearing stage on the $s/p = 0.57$ plane for SI 300-300 test

Figure B.69 Yield curve and failure envelope at the end of the infiltration stage on the $s/p = 0.09$ plane for SI 300-300 test

Figure B.70 Variation of deviator stresses with respect to axial strain for SI25-50 (Meilani 2004) test

Figure B.71 Variation of matric suction with respect to axial strain for SI25-50 (Meilani 2004) test

Figure B.72 Variation of volumetric strain with respect to axial strain for SI25-50 (Meilani 2004) test

Figure B.73 Variation of deviator stresses with respect to axial strain for SI100-50 (Meilani 2004) test

Figure B.74 Variation of matric suction with respect to axial strain for SI100-50 (Meilani 2004) test

Figure B.75 Variation of volumetric strain with respect to axial strain for SI100-50 (Meilani 2004) test

Figure B.76 Variation of deviator stresses with respect to axial strain for SI200-50 (Meilani 2004) test

Figure B.77 Variation of matric suction with respect to axial strain for SI200-50 (Meilani 2004) test

Figure B.78 Variation of volumetric strain with respect to axial strain for SI200-50 (Meilani 2004) test

Figure B.79 Variation of deviator stresses with respect to axial strain for SI25-100 (Meilani 2004) test

Figure B.80 Variation of matric suction with respect to axial strain for SI25-100 (Meilani 2004) test

Figure B.81 Variation of volumetric strain with respect to axial strain for SI25-100 (Meilani 2004) test

Figure B.82 Variation of deviator stresses with respect to axial strain for SI100-100 (Meilani 2004) test

Figure B.83 Variation of matric suction with respect to axial strain for SI100-100 (Meilani 2004) test

Figure B.84 Variation of volumetric strain with respect to axial strain for SI100-100 (Meilani 2004) test

Figure B.85 Variation of deviator stresses with respect to axial strain for SI200-100 (Meilani 2004) test

Figure B.86 Variation of matric suction with respect to axial strain for SI200-100 (Meilani 2004) test

Figure B.87 Variation of volumetric strain with respect to axial strain for SI200-100 (Meilani 2004) test

Figure B.88 Variation of deviator stresses with respect to axial strain for SI25-200 (Meilani 2004) test

Figure B.89 Variation of matric suction with respect to axial strain for SI25-200 (Meilani 2004) test

Figure B.90 Variation of volumetric strain with respect to axial strain for SI25-200 (Meilani 2004) test

Figure B.91 Variation of deviator stresses with respect to axial strain for SI100-200 (Meilani 2004) test

Figure B.92 Variation of matric suction with respect to axial strain for SI100-200 (Meilani 2004) test

Figure B.93 Variation of volumetric strain with respect to axial strain for SI100-200 (Meilani 2004) test

Figure B.94 Variation of deviator stresses with respect to axial strain for SI200-200 (Meilani 2004) test

Figure B.95 Variation of matric suction with respect to axial strain for SI200-200 (Meilani 2004) test

Figure B.96 Variation of volumetric strain with respect to axial strain for SI200-200 (Meilani 2004) test