

LB/DON/54/08

13



Consolidation Analysis of Sri Lankan Peaty Clay using Elasto-viscoplastic Theory



University of Moratuwa, Sri Lanka.
Electronic Theses & Dissertations
www.lib.libraryk
UNIVERSITY OF MORATUWA, SRI LANKA
MORATUWA

624 "07"
624.131.22 (043)

June 2007

University of Moratuwa



91211

91211

Wanigavitharana Asiri Karunawardena

91211

With the compliments of

Asiri
Asiri Karunawardena

20/12/2007



University of Moratuwa, Sri Lanka.
Electronic Theses & Dissertations
www.lib.mrt.ac.lk

UNIVERSITY OF MORATUWA, SRI LANKA LIBRARY	
ACCESSION No.	91211
CLASS No.	624 "07" 624.131.22(043)

Abstract

The consolidation of peat is complex due to the resultant large strain associated with the highly compressible nature of natural peat deposits and to the rapid changes in soil properties during the consolidation process. In addition, the consolidation process is further complicated by the occurrence of secondary compression which significantly contributes to the overall settlement of peaty soil. Therefore, it is necessary to take these properties into account in order to obtain better predictions from peat consolidation analyses. In the present study, the consolidation behavior of peaty clay found in Sri Lanka is extensively studied using a model based on the elasto-viscoplastic theory. The model can describe the prominent creep behavior of peaty soil as a continuous process. In addition, the model can accommodate the effect of structural degradation on the consolidation process. The analysis takes into account all the main features involved in the peat consolidation process, namely, finite strain, variable permeability, and the effect of secondary compression. Also, it considers the variable compressibility for stage-constructed embankments which exert high levels of pressure on the peaty subsoil.

The constitutive equations used in the model and the procedure adapted to account for the above-mentioned features of the analysis are described. The constitutive model is based on Perzyna's type viscoplastic theory and the Cambridge elasto-plastic theory combined with empirical evidence. In the finite element formulations, which are based on the finite deformation theory, an updated Lagrangian method is adopted. A description of the material parameters used in the model and the procedures applied to evaluate them, with standard laboratory and field tests, are explained. In addition, a performance of the model incorporating the original and the modified Cam-clay theory is evaluated by simulating triaxial test results. A comparison shows that with the present definition of the parameters, the original model yields more representative results than the model based on the modified Cam-clay theory.

Initially, the capability of the constitutive model to capture the consolidation behavior is verified using the consolidation model test data on peaty clay found in Sri Lanka. It is confirmed that the constitutive model is able to predict the observed creep characteristics and the effect of sample thickness on settlement predictions for the material under consideration.

The performance of the model in predicting the consolidation behavior under field conditions is studied using field data on instrumented earth fill constructed on peaty clay. One-dimensional compression is assumed for the peaty clay due to the large plane area of the fill. Separate analyses are carried out by the model considering the infinitesimal strain theory, the finite strain theory, and the finite strain theory together with the effect of structural degradation in order to explore how these features describe the observed field

behavior. Analyses reveal that it is necessary to consider finite deformation together with the effect of structural degradation in order to successfully simulate the resultant large strain and the stagnated pore water pressure observed in the field.

The construction of road embankments over peat deposits is quite problematic, and thus, it is often done after first improving the properties of the peaty soil through the utilization of appropriate ground-improvement techniques. Understanding the field response of peaty clay during this improvement process is naturally of great importance. A constitutive model is applied to predict the field performance of embankments constructed on peaty clay using different ground-improvement techniques. The back analysis of embankments constructed with the preloading method indicates that the model can be successfully applied to predict both the deformation and the stability of structures constructed on peaty clays. The stability of the embankment during and after construction is verified by investigating the stress-strain characteristics of the subsoil.

The model applications used to predict the consolidation behavior of embankments constructed by the preloading method, combined with other ground-improvement techniques, are then discussed. Embankments constructed with prefabricated vertical drains (PVDs) and sand compaction piles (SCPs) are considered, and finite element analyses are carried out in all cases by converting the actual three-dimensional conditions that exist around the drains into simplified two-dimensional plane strain conditions. The field behavior when PVDs are installed in the peaty clay is simulated using the equivalent vertical permeability for the PVD-improved subsoil. In the case of SCPs, a conversion scheme is used to transform the axisymmetric nature of sand columns into equivalent plane strain conditions. A comparison of the predicted results with the field observations shows a reasonable agreement. An analysis of the PVD-improved foundation indicates that the installation of PVDs not only accelerates the rate of consolidation, but influences the deformation pattern of the subsoil due to embankment loading. The analysis also shows that the use of PVDs can significantly increase embankment stability. The model prediction for the SCP-improved foundation reveals that the stiffness and the area replacement ratio used in the conversion scheme play vital roles in predicting the behavior of SCP-improved soft grounds. The observed improvements in the bearing capacity of the subsoil and in the stability of the embankment, brought about by the installation of SCPs, can be simulated by the model.

Acknowledgements

The research described in this thesis was carried out at the Graduate School of Engineering, Kyoto University, Japan. I wish to express my profound gratitude to the members of the Dissertation Committee, namely, Professor Fusao Oka, Professor Takeshi Tamura, and Associate Professor Sayuri Kimoto, for their discussions, invaluable comments, and constructive suggestions in reviewing this work.

I am extremely grateful to my academic advisor, Professor Fusao Oka, for his enthusiastic guidance and his invaluable help and encouragement in all aspects of this research work. The numerous comments, criticisms, and suggestions he made, based on his deep insight and vast experience in the field of geotechnical engineering, contributed greatly to the success of this work. Also, I am deeply indebted to him for giving me the opportunity to follow the International Doctoral Program in Engineering organized by the Graduate School of Engineering, Kyoto University. I am sure that the confidence I gained being a student of Professor Oka will be a vital asset as I take my career to the next level.

I also owe many thanks to Associate Professor Sayuri Kimoto for her valuable suggestions, constant motivation, and constructive discussions during my doctoral research. Her continuous and ever-present support contributed significantly to my understanding of the constitutive model as well as to the outcome of this research work.

I would like to extend my thanks to Dr. Yosuke Higo, Research Associate in Geomechanics at the Graduate School of Engineering, for his kind help on various occasions. Also, the guidance and the advice on laboratory testing given by Associate Professor Takeshi Kodaka, now at Meiji University, Japan is gratefully acknowledged.

I offer my sincere thanks to all the students who graduated during my period of study. Among them, Dr. Boonlert Siribumrungwong, Dr. Md Rezaul Karim, Mr. Ryosuke Kato, Mr. Hirotaka Suzuki, and Mr. Ryota Asai deserve special recognition. The support given by Mr. Naoaki Takada, who now works for Japan Railways, and Mr. Hideki Kitahara, who now works for Kajima Corporation is highly appreciated and will never be forgotten.

Thanks are also due to Ms Chikako Itou, Secretary of Oka Laboratory, who has been so kind in assisting me with all the official formalities during the research period. I would like to thank all of my friends who are current members of the Oka Laboratory and have helped me in one way or another, including doctor course students Feng Huaiping, Nguen Huy Quoc, and master course students, Tinet Anne-Julie, Mai Sawada, Shinya Yamazaki, Tomohiko Fushita, Hirofumi Ohta, Junya Fukutani and Anna Paula Heitior, who all deserve to be mentioned.

The research presented in this dissertation was financially supported by the MONBUSHO Scholarship Program, provided by the Ministry of Education, Science and Culture, Government of Japan. This program deserves special acknowledgement.

I wish to express my deepest appreciation and gratitude to my supervisor in the Master Degree Program, Dr. S.A.S. Kulathilaka, Senior Lecturer, University of Moratuwa, Sri Lanka, for making arrangements to provide the relevant field and laboratory data used in this research work. Also, I am grateful to the Road Development Authority, University of Moratuwa and the National Building Research Organization of Sri Lanka for providing the necessary data for this research work.

I wish to express my gratitude to the Director General of the National Building Research Organization (NBRO) for granting me a study leave for this research work. The support and the encouragement given by Mr. D.L.C. Welikala, former Director of the Geotechnical Engineering Division of NBRO is highly acknowledged. Special thanks are due to all staff members at NBRO, including Mrs. Karuna De Silva, Mrs. Gayani Samaradiwakara, Mrs. Prashanthi Dissanayake, and Ms Sriyani Munashinghe.

Many thanks go to my friends in Sri Lanka for helping me in various ways to achieve this goal. A special note of sincere appreciation is also extended to Mr. Kishan Sugathapala and his family for their continuing support and good wishes.

I would like to thank all my Sri Lankan friends studying at Kyoto University for their help in many ways. I would also like to extend a huge thank you to Ms. Ryuko Yamaoka and her family, of Ibaraki City, Osaka, for making me feel at home during my stay in Japan.

I offer my heart-felt gratitude to my late father who passed away five months before I started this study program. I am sure that his principles and teachings contributed a great deal to the person I have become. I am indebted to my two brothers and their wives, namely, Sampath and Lali, and Dumindu and Nirosha, for their ever-loving support, patience, and encouragement during this work. Finally, I dedicate this dissertation to my dearest mother who has always inspired me to challenge myself. Her constant love, trust, understanding, and encouragement follow me everywhere.

Table of Contents

Abstract	i
Acknowledgement	iii
Table of Contents	v
List of Figures	ix
List of Tables	xiii
1 INTRODUCTION	1
1.1 Background and Objectives	1
1.2 Organization of the Dissertation	5
2 GENERAL FEATURES OF PEATY SOIL	7
2.1 Introduction	7
2.2 Physical Properties of Peaty Soil	8
2.3 Engineering Properties of Peaty Soil	11
2.4 Classification of Peat for Engineering Purposes	15
2.5 Properties of Sri Lankan Peat	16
2.5.1 Empirical Correlations for Sri Lankan Peaty Clay	17
2.6 Summary	20
3 CONSTITUTIVE MODEL AND ANALYSIS METHODS TO SIMULATE PEAT CONSOLIDATION	21
3.1 Introduction	21
3.2 Elasto-viscoplastic Constitutive Model	23
3.2.1 Constitutive Equation of Geomaterials	24
3.2.2 Elastic Strain Rate	24
3.2.3 Overconsolidation Boundary Surface	25
3.2.4 Static Yield Function and Viscoplastic Potential Function	26
3.2.5 Account of the Effect of Structural Degradation	28
3.2.6 Viscoplastic Flow Rule	29
3.3 Features of the Analysis Related to the Consolidation Process for Peat	31
3.3.1 Variable Permeability and Compressibility	31

3.3.2	Relationship between Viscoplastic Parameter m' and C_α / C_c	33
3.3.3	Account of Finite Deformation	35
3.4	Determination of the Model Parameters	43
3.5	Performance of the Elasto-viscoplastic Model with the Modified Cam-clay Theory	45
3.5.1	Derivation of Viscoplastic Volumetric and Deviatoric Strain Rates	46
3.5.2	Evaluation of the Model Performance	49
3.6	Summary and Remarks	52
4	MODEL VALIDATION USING LABORATORY CONSOLIDATION	
	TEST RESULTS	53
4.1	Introduction	53
4.2	Application of the Model to Predict Laboratory Consolidation Behavior	54
4.2.1	Determination of the Model Parameters for the Analysis	54
4.2.2	Comparison of Terzaghi's and Elasto-viscoplastic Models	56
4.3	Model Performance of Settlement Prediction Considering Different Layer Thicknesses	57
4.3.1	Importance of the Effect of Sample Thickness on the Consolidation Analysis	58
4.3.2	Numerical Simulation of the Effect of Sample Thickness on a Consolidation Analysis	59
4.3.3	Consolidation Data for Peaty Soil with Different Sample Heights	61
4.3.4	Prediction of the Observed Settlements	64
4.4	Experimental Verification of the Relation between the Parameter m' and C_α / C_c	68
4.5	Summary and Remarks	72
5	PREDICTION OF THE ONE-DIMENSIONAL CONSOLIDATION	
	BEHAVIOR OF PEATY CLAY IN THE FIELD	75
5.1	Introduction	75
5.2	Description of the Project	76
5.3	Determination of the Soil Parameters	78
5.4	Numerical Details for the Analysis	82
5.5	Prediction of the Field Behavior using the Infinitesimal Strain Theory	83
5.6	Prediction of the Field Behavior using the Finite Deformation Theory	86

5.6.1	Comparison of the Results from Finite Strain with Infinitesimal Deformation Analysis	88
5.7	Prediction of the Field Behavior using the Finite Deformation Theory and Considering the Effect of Degradation	88
5.8	Comparison of the Results and a Discussion	91
5.8.1	Importance of Considering the Effect of Degradation	92
5.9	Summary and Remarks	93
6	SIMULATION OF FIELD BEHAVIOR DUE TO EMBANKMENT CONSTRUCTION ON NATURAL SUBSOIL	95
6.1	Introduction	95
6.2	Project Description	97
6.2.1	Subsoil Condition and Embankment Construction	97
6.3	Material Parameters	99
6.4	Finite Element Analysis of the Embankment Foundation	100
6.5	Prediction of Field Behavior	102
6.5.1	Deformation Behavior of Subsoil	102
6.5.2	Excess Pore Water Pressure	105
6.6	Verification of the Embankment Stability	106
6.6.1	Results and Discussion	109
6.7	Summary and Concluding Remarks	119
7	SIMULATION OF FIELD BEHAVIOR DUE TO EMBANKMENT CONSTRUCTION ON IMPROVED SUBSOIL	121
7.1	Introduction	121
7.2	Modeling of PVD-improved Subsoil under Plane Strain Conditions	122
7.2.1	Verification of the Method using Large-scale Model Test Data	125
7.3	Analysis of an Embankment Constructed on PVD-improved Subsoil	128
7.3.1	Details of the Embankment and the Subsoil Profile	128
7.3.2	Finite Element Analysis	129
7.3.3	Results of the Finite Element Analysis	132
7.4	Stability of the Embankment	136
7.4.1	Diagram for the Construction Control of Embankments	136
7.4.2	Effect of PVDs on Embankment Stability	137
7.5	Modeling of Sand Compaction Piles in the Finite Element Analysis	138

7.6	Prediction of the Consolidation Behavior of an Embankment Constructed using SCPs	141
7.6.1	Results and Discussion	144
7.7	Prediction of the Consolidation Behavior of an Embankment Constructed with both SCPs and PVDs	147
7.8	Summary and Remarks	150
8	CONCLUSIONS AND RECOMMENDATIONS	153
8.1	Summary and Conclusions	153
8.2	Recommendations for Future Work	156
	References	159
	Appendix	169
A1	Finite Element Formulation for Infinitesimal Strain Analysis	169
A2	Evaluation of Equivalent Vertical Permeability for PVD-improved Subsoil	175



University of Moratuwa, Sri Lanka.
Electronic Theses & Dissertations
www.lib.mrt.ac.lk

List of Figures

Fig. 1.1:	Prediction of excess pore water pressure using Terzaghi's model (Karunawardena 2002)	2
Fig. 1.2:	Prediction of settlements using Terzaghi's model (Karunawardena 2002).....	2
Fig. 1.3:	Variation in coefficient of consolidation during the consolidation process.....	3
Fig. 2.1:	Void ratio vs. organic content for some foreign peats (Hobbs 1987)	8
Fig. 2.2:	Bulk density vs. water content in some UK peats (Hobbs 1986).....	9
Fig. 2.3:	Specific gravity vs. water content relationship in peat soils around the world (Hobbs 1986).....	10
Fig. 2.4:	Liquid limit vs. water content relationship for peats (Hobbs 1986).....	10
Fig. 2.5:	Organic content vs. liquid limit relationship for peats (Hobbs 1986)	11
Fig. 2.6:	Values of the compression index and the natural water content for peats in comparison to those for soft clay and silt deposits (Mesri et al. 1997)	12
Fig. 2.7:	C_α / C_c relationship for Middleton peat, UK (Mesri et al. 1997).....	12
Fig. 2.8:	Values of C_k for peats compared to those for soft clay and silt deposits (Mesri et al. 1997).....	13
Fig. 2.9:	Distribution of peaty areas around Colombo.....	16
Fig. 2.10:	C_c vs. natural water content (%) relationship for Sri Lankan peats	18
Fig. 2.11:	C_c vs. natural water content (%) relationship for foreign peats	18
Fig. 2.12:	C_c vs. e_0 relationship for Sri Lankan peats	18
Fig. 2.13:	C_c vs. O.M.C. (%) relationship for Sri Lankan peats	19
Fig. 2.14:	C_α vs. O.M.C. (%) relationship for Sri Lankan peats	19
Fig. 2.15:	C_α vs. C_c relationship for Sri Lankan peats	20
Fig. 2.16:	C_r vs. C_c relationship for Sri Lankan peats	20
Fig. 3.1:	Stress-strain relationship used in the model	24
Fig. 3.2:	Overconsolidated boundary surface under triaxial conditions	26
Fig. 3.3:	OC boundary surface, static yield function, and viscoplastic potential function	27
Fig. 3.4:	Different C_c values for Olga clay (Mesri 1985).....	32
Fig. 3.5:	Typical shape of $e - \log \sigma'_v$ curve for Sri Lankan peaty clay	32
Fig. 3.6:	Finite elements and Gauss integration points	35

Fig. 3.7:	Modified Cam-clay yield locus	46
Fig. 3.8:	Stress paths	49
Fig. 3.9:	q/σ'_m vs. $\dot{\epsilon}_{11}$ relationship	49
Fig. 3.10:	Comparison of the stress-strain behavior	51
Fig. 3.11:	Comparison of the stress paths	51
Fig. 4.1:	Void ratio vs. $\log \sigma'_v$ relationship	54
Fig. 4.2:	Void ratio vs. $\log(\text{time})$ relationship	54
Fig. 4.3:	Vertical strain vs. time profile	56
Fig. 4.4:	Rate of change in void ratio vs. time profile	56
Fig. 4.5:	Predicted excess pore water pressure behavior	57
Fig. 4.6:	Consolidation of clay layers of different thicknesses according to Hypotheses A and B	59
Fig. 4.7:	Numerical simulation of the effect of sample thickness on settlement predictions.....	61
Fig. 4.8:	Schematic diagram of the large scale model	62
Fig. 4.9:	Importance of controlling the preparatory consolidation time	63
Fig. 4.10:	Finite element meshes used in the analysis	64
Fig. 4.11:	Strain predicted by considering the difference in initial strain rates	66
Fig. 4.12:	Strain predicted by considering the same initial strain rates for the thick and the thin samples.....	66
Fig. 4.13:	Predicted settlement for the oedometer test considering high initial strain rate	67
Fig. 4.14:	Model performance of the settlement prediction for different layer thicknesses	68
Fig. 4.15:	Experimental results for soft marine clay found around Aji River, Osaka, Japan	70
Fig. 4.16:	Experimental results for soft marine clay found around Higashi Osaka, Osaka, Japan	71
Fig. 5.1:	Plan view of the fill and the instrumentation location	76
Fig. 5.2:	Construction history of the fill	77
Fig. 5.3:	Locations of the field instruments	77
Fig. 5.4:	Subsurface profile at the site	78
Fig. 5.5:	Observed relationship for the permeability vs. void ratio obtained through consolidation test.....	79
Fig. 5.6:	Void ratio vs. vertical effective stress relationship obtained through stage-loading consolidation test	79

Fig. 5.7:	Void ratio-logarithmic time curves from stage-loading consolidation test	79
Fig. 5.8:	Simulation of the stress-strain behavior	81
Fig. 5.9:	Simulation of the stress paths	81
Fig. 5.10:	Finite element mesh and boundary conditions	83
Fig. 5.11:	Four-node isoparametric element for displacement interpolation	84
Fig. 5.12:	Excess pore water prediction assuming infinitesimal strain	85
Fig. 5.13:	Settlement prediction assuming infinitesimal strain	85
Fig. 5.14:	Prediction of excess pore water pressure based on finite strain, infinitesimal strain, and corresponding field data	87
Fig. 5.15:	Prediction of settlements based on finite strain, infinitesimal strain, and corresponding field data	87
Fig. 5.16:	Excess pore water pressure prediction based on finite strain, considering the effect of degradation	90
Fig. 5.17:	Settlement prediction based on finite strain, considering the effect of degradation	90
Fig. 5.18:	Excess pore water pressure-time profile	91
Fig. 5.19:	Settlement-time profile beneath the fill	92
Fig. 6.1:	Subsurface profile beneath the embankment	98
Fig. 6.2:	Loading curve	98
Fig. 6.3:	Finite element configuration	101
Fig. 6.4:	Initial vertical effective stress profile	101
Fig. 6.5:	Time increment used in the analysis	101
Fig. 6.6:	Comparison of settlement predictions	102
Fig. 6.7:	Deformation patterns of the subsoil	104
Fig. 6.8:	Predicted excess pore water pressure beneath the embankment	105
Fig. 6.9:	Distribution of excess pore water pressure beneath the embankment	106
Fig. 6.10:	Soil element behavior under the embankment - silty clay/peat: 5.75 m depth	112
Fig. 6.11:	Soil element behavior under the embankment - silty clay/peat: 4.25 m depth	113
Fig. 6.12:	Soil element behavior under the embankment - silty clay/peat: 2.75 m depth	114
Fig. 6.13:	Soil element behavior under the embankment - silt: 1.5 m depth	115
Fig. 6.14:	Soil element behavior under the embankment - peat: 0.75 m depth	116
Fig. 6.15:	Soil element behavior under the embankment - peat: 0.25 m depth	117
Fig. 6.16:	Variation in accumulated viscoplastic shear strain during construction	118
Fig. 6.17:	Variation in accumulated viscoplastic shear strain after construction	119



Fig. 7.1:	Arrangement of the test apparatus	125
Fig. 7.2:	Comparison of excess pore water pressure	127
Fig. 7.3:	Comparison of the settlements	127
Fig. 7.4:	Subsurface profile at the site	128
Fig. 7.5:	Loading curve	129
Fig. 7.6:	Finite element mesh with boundary conditions	130
Fig. 7.7:	Comparison of settlements	132
Fig. 7.8:	Comparison of the excess pore water pressure	133
Fig. 7.9:	Predicted excess pore water pressure under the center of the embankment as a function of the embankment load	134
Fig. 7.10:	Predicted lateral displacement profiles for the PVD-improved subsoil	135
Fig. 7.11:	Graph showing the influence of PVDs on lateral displacement	135
Fig. 7.12:	Modified Matsuo stability plot	136
Fig. 7.13:	Improvement in embankment stability due to the PVD installation	137
Fig. 7.14:	Conversion from an axisymmetric unit cell into an equivalent plane strain.....	139
Fig. 7.15:	Subsurface profile at the site	141
Fig. 7.16:	Loading curve	142
Fig. 7.17:	Finite element mesh showing the locations of the SCPs	142
Fig. 7.18:	Predicted settlement under the center of the embankment	144
Fig. 7.19:	Predicted settlement under the crest of the embankment	144
Fig. 7.20:	Mean effective stress distribution in the foundation due to embankment loading	145
Fig. 7.21:	Comparison of excess pore water pressure	146
Fig. 7.22:	Predicted deformation pattern for the embankment at the end of construction	146
Fig. 7.23:	Finite element mesh with a PVD-improved zone and SCP locations	147
Fig. 7.24:	Comparison of settlements under the embankment center.....	148
Fig. 7.25:	Comparison of excess pore water pressure	148
Fig. 7.26:	Distribution of excess pore water pressure in the foundation	149
Fig. 7.27:	Distribution of mean effective stress for embankment foundation	149
Fig. 7.28:	Deformation pattern of the embankment foundation without considering SCP improvement under the slope	149
Fig. A1.1:	Drainage distance used to evaluate the rate of flow	173

List of Tables

Table 2.1:	Values of C_{α} / C_c for natural soil deposits	12
Table 2.2:	Values of natural water content, initial vertical coefficient of permeability, and C_{α} / C_c for peat deposits	13
Table 2.3:	Basic properties of Sri Lankan peat	16
Table 3.1:	Comparison between original Cam-clay and Modified Cam-clay	48
Table 3.2:	m' calculated from the experimental results	49
Table 3.3:	Parameters used in the simulation.....	50
Table 4.1:	Parameters used in the model validation	55
Table 4.2:	Parameters used in the numerical simulation of the effect of sample thickness.....	60
Table 4.3:	Description of the test setup.....	62
Table 4.4:	Preparatory consolidation data	64
Table 4.5:	Parameters used in the settlement prediction of peat sample with different thicknesses	65
Table 4.6:	Summary of the test results and the estimated m' values	72
Table 5.1:	Parameters used in the triaxial simulation	80
Table 5.2:	Parameters used in the analysis of a one-dimensional field consolidation problem	82
Table 6.1:	Parameters used in the embankment analysis on natural soil	99
Table 7.1:	Material parameters for the large-scale peaty soil model with PVD	126
Table 7.2:	Drain parameters used in the analysis	127
Table 7.3:	Parameters used in the analysis of PVD-improved embankment foundation analysis	130
Table 7.4:	Drain parameters used in the analysis	131
Table 7.5:	Parameters used for the SCP-improved peaty clay subsoil	143
Table 7.6:	Parameters for the SCPs	143