GEOTECHNICAL, GEOLOGICAL AND EARTHQUAKE ENGINEERING

DEFORMATION ANALYSIS IN SOFT GROUND IMPROVEMENT

JINCHUN CHAI
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by

JINCHUN CHAI
Saga University, Japan

JOHN P. CARTER
The University of Newcastle, Callaghan, NSW, Australia

Springer

Jinchun Chai
Saga University
Department of Civil Engineering
and Architecture
Honjo 1
840-8502 Saga
Japan
chai@cc.saga-u.ac.jp

John P. Carter
The University of Newcastle
Faculty of Engineering and Built
Environment
University Drive
2308 Callaghan
Australia
John.Carter@newcastle.edu.au

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Preface

This book describes a variety of analytical and numerical models, based firmly on experimental evidence from observations made in both the laboratory and the field, which can be used in geotechnical engineering to make predictions of soil deformations arising from or associated with ground improvement techniques. The use of such techniques is becoming ever more popular in soft ground engineering. The deformations of the ground addressed in this work are often time-dependent and, given that soft soil behaviour can be highly non-linear, they are often difficult to estimate reliably.

This volume was inspired by the work of many researchers, practitioners and authors who have made significant contributions to this important field of geotechnical engineering. In writing the book it was the intention of the authors to provide a state of the art and practice in this field, at least as they stand at the time of writing. Ground improvement technology and our analytical and numerical capabilities are advancing at such a pace that there is a significant risk that such a volume may date quickly. Nevertheless, the fundamental mechanics of the soft soils to be treated will remain the same. It is our ability to describe and model them more accurately and reliably that may change with time, along with the evolving technologies being developed to improve their mechanical behaviour.

The book is directed towards students of geotechnical engineering as well as geotechnical practitioners. In the main it provides rather complete derivations of most of the important theoretical results, rather than just bland statements of each result. This is deliberate, as the intention was to write a book that could be used as both a teaching text and a reference work. In presenting such material the authors were also keenly aware of their obligation to present critical evaluation and validation of the various analytical and theoretical models presented in this book and therefore proposed for use in engineering practice. Accordingly, the book also contains numerous case histories and comparisons of the predictions of the proposed models with the results from high quality laboratory and field experimental work. The field case histories are from soft soil sites at various locations around the world.

The book starts with a chapter describing some of the various ground improvement technologies that are in current use in soft ground engineering. Not only does it provide brief descriptions of these technologies, but it also makes the argument that predicting the effects on subsequent ground performance of implementing these technologies is important in engineering practice. One important effect concerns the resulting ground deformations.

Since the development of predictive techniques for estimating soil displacements is underpinned by and critically dependent on the constitutive models adopted to represent the mechanical response of soil, it seems only natural to describe some of these fundamental tools that have been used successfully to date for this task. This is the rationale for including Chapter 2, which contains a description of some of the techniques that have been used successfully in modelling the mechanical response of soft clay subsoils. Included in this treatment are some of the fundamental stress-strain models used to describe the behaviour of clays as well as their implementation in the finite element method (FEM). Also presented in this chapter are example analyses to demonstrate the power and significance of these modern numerical techniques

Chapter 3 contains a description of the methods used for modelling soft ground improved by the installation of pre-fabricated vertical drains (PVDs) using plane strain finite element analyses. Also presented are some example analyses using specific case histories.

Chapter 4 presents the theory of vacuum consolidation and describes the major characteristics of vacuum consolidation of soft ground, the methods adopted for calculating the deformations induced by application of vacuum pressure, as well as some example analyses based on case histories.

Chapter 5 describes a consolidation theory for a clayey subsoil improved by the installation of soil-cement columns. It also describes methods for estimating ground settlement (mainly for the case of ground improved by the prior installation of 'floating' columns), a method for predicting the lateral displacement of the ground induced by the column installation, and the analyses of some case histories.

Finally, Chapter 6 presents suggestions and recommendations for further research and development in this area.

It is worth reiterating that the deliberate intent of the authors was that this book should provide up-to-date information on this important subject from both the theoretical and practical points of view.

While this book was inspired by the many developments already in the field of soft ground engineering, it is also our hope that it will in turn inspire others to focus their efforts on the important task of improving the mechanical behaviour of soft ground, striving to find better and more reliable ways to make such ground 'stiffer and stronger', and striving to develop more accurate and reliable models to be used in association with these techniques.

We are indebted to a number of people for their significant assistance in the task of producing this book, some of whom made direct contributions to its preparation and others whose efforts were more inadvertent but no less important. In particular, we would like to thank Ms M. Komoto and Mrs Y. Kanada for their patient and skillful assistance in preparing some of the figures contained in this book. Importantly,

we would also like to thank our life-long partners, Yaru Wang and Heather Carter, for their patience, support, love and understanding. It is to them that this contribution is dedicated.

Saga, Japan Jinchun Chai
Newcastle, New South Wales John P. Carter

Contents

Notation

A	cross-sectional area of a drainage channel (L^2)
B	measure of the proximity of the kinematic yield and bounding surfaces in the double surface 'bubble' model (dimensionless)
B	half width of a plane strain unit cell (L)
c	cohesion of soil ($ML^{-1}T^{-2}$)
c'	effective stress cohesion ($ML^{-1}T^{-2}$)
C_c	compression index (dimensionless)
C_d	a constant (dimensionless)
C_f	ratio of field hydraulic conductivity over corresponding laboratory value (dimensionless)
c_h	coefficient of consolidation in horizontal direction (L^2T^{-1})
C_k	a constant (dimensionless)
c_v	coefficient of consolidation in vertical direction (L^2T^{-1})
D	coefficient of dilatancy (dimensionless)
d_e	diameter of a unit cell of PVD (L)
d_m	area equivalent diameter of PVD installation mandrel (L)
d_s	diameter of smear zone (L)
d_w	diameter of vertical drain (L)
e	void ratio (dimensionless)
e_0	initial void ratio (dimensionless)
E	Young's modulus ($ML^{-1}T^{-2}$)
E_b	bulk modulus ($ML^{-1}T^{-2}$)
f	yield function
g	plastic potential function
g	gravitational acceleration (LT^{-2})
G'	elastic shear modulus ($ML^{-1}T^{-2}$)
H_c	optimum thickness of unimproved sub-layer
H_L	thickness of a soil layer (L)
i	hydraulic gradient (dimensionless)
$\boldsymbol{I}$	the second-rank identity tensor
I_r	rigidity index (dimensionless)
I_{rr}	reduced rigidity index (dimensionless)

k	a constant for tangential modulus (dimensionless)
K	stiffness matrix (MT^{-2})
k	hydraulic conductivity (LT^{-1})
k_0	initial permeability (LT^{-1})
K_a	the active earth pressure coefficient (dimensionless)
k_b	a constant for bulk modulus (dimensionless)
K_F	element stiffness matrix (MT^{-2})
k_h	hydraulic conductivity in horizontal direction (LT^{-1})
k_{hp}	matched horizontal hydraulic conductivity in plane strain condition (LT^{-1})
K_0	at-rest earth pressure coefficient (dimensionless)
k_s	hydraulic conductivity of smear zone (LT^{-1})
k_{sp}	hydraulic conductivity of the smear zone in the plane strain unit cell (LT^{-1})
k_v	hydraulic conductivity in vertical direction (LT^{-1})
k_{ve}	equivalent hydraulic conductivity in vertical direction (LT^{-1})
L	link (or coupling) matrix (L^2)
l	drainage length (L)
m	a constant for calculating bulk modulus (dimensionless)
M	slope of critical state line in q:p' plot (dimensionless)
n	a constant for calculating tangential modulus (dimensionless)
n	radius ratio = R/r_w (dimensionless)
N	specific volume of isotropic compression line at unit mean effective stress (dimensionless)
OCR	over consolidation ratio (dimensionless)
p	initial mean stress ($ML^{-1}T^{-2}$)
p'	effective overburden pressure ($ML^{-1}T^{-2}$)
p'	mean effective stress ($ML^{-1}T^{-2}$)
p'_0	size of yielding locus on mean effective stress axis ($ML^{-1}T^{-2}$)
p'_0	value of the mean effective stress at the intersection of the current swelling line with the isotropic compression line in the double surface 'bubble' model ($ML^{-1}T^{-2}$)
p'_i	initial mean effective stress ($ML^{-1}T^{-2}$)
p_a	atmospheric pressure ($ML^{-1}T^{-2}$)
p'_a	mean effective stress tensor at the centre of the kinematic yield surface ($ML^{-1}T^{-2}$)
q	deviator stress ($ML^{-1}T^{-2}$)
q	flow rate [L^3T^{-1}]
q_{VD}	flow rate from surrounding soil into a PVD (L^3T^{-1})
q_w	discharge capacity of prefabricated vertical drain (PVD) (L^3T^{-1})
q_{wp}	equivalent discharge capacity of a plane strain drain (L^3T^{-1})
r	radial distance (L)
R	the ratio of the size of the kinematic yield surface to that of the bounding surface (dimensionless)
r_e	radius of a unit cell (L)

R_f	failure ratio of hyperbolic model (dimensionless)
R_q	discharge capacity ratio (dimensionless)
r_s	radius of smear zone (L)
r_w	radius of vertical drain (L)
$\boldsymbol{s}$	deviator stress tensor ($ML^{-1}T^{-2}$)
s	ratio of the diameter of smear zone over the diameter of a drain
S	spacing of PVDs (L)
$\boldsymbol{s}_a$	deviatoric stress tensor at the centre of the kinematic yield surface ($ML^{-1}T^{-2}$)
$\boldsymbol{s}_{ij}$	the (i, j) component of the deviator stress tensor ($ML^{-1}T^{-2}$)
$\boldsymbol{s}_{ij0}$	initial value of $\boldsymbol{s}_{ij}$ at the end of anisotropic consolidation ($ML^{-1}T^{-2}$)
S_u	undrained shear strength ($ML^{-1}T^{-2}$)
S_x	length of an element in x direction (L)
S_y	length of an element in y direction (L)
S_z	length of an element in z direction (L)
t	thickness of a prefabricated vertical drain (PVD) (L)
t	time (T)
T_h	time factor in horizontal direction (dimensionless)
T_v	time factor in vertical direction (dimensionless)
u	excess pore water pressure ($ML^{-1}T^{-2}$)
$\bar{u}$	average excess pore water pressure ($ML^{-1}T^{-2}$)
u_i	excess pore pressure at the center of ith element ($ML^{-1}T^{-2}$)
U_h	average degree of consolidation due to radial drainage (dimensionless)
$\overline{U}_h$	average degree of consolidation in horizontal direction (dimensionless)
U_{kp}	average degree of consolidation in horizontal direction under plane strain condition (dimensionless)
U_v	average degree of consolidation due to vertical drainage (dimensionless)
U_{vh}	average degree of consolidation of PVD-improved subsoil (dimensionless)
v	specific volume (dimensionless)
v_d	dilatancy (dimensionless)
w	width of a PVD (L)
W_L	liquid limit (dimensionless)
W_n	natural water content (dimensionless)
W_P	plastic limit (dimensionless)
X	material property in double surface 'bubble' model (dimensionless)
X_p	material property in double surface 'bubble' model for deviatoric shape of plastic potential (dimensionless)
Y	material property in double surface 'bubble' model (dimensionless)
Y_p	material property in double surface 'bubble' model for deviatoric shape of plastic potential (dimensionless)
x	Cartesian co-ordinate
y	Cartesian co-ordinate
z	depth (L)
Z	material property in double surface 'bubble' model (dimensionless)

Z_p	material property in double surface 'bubble' model for deviatoric shape of plastic potential (dimensionless)
α	secondary compression index (dimensionless)
β	constant of proportionality for viscoplastic strain (dimensionless)
β_i	coefficient of water flow ($M^{-1}L^3T^2$)
β_{VD}	coefficient of water flow of a PVD per unit length ($M^{-1}L^3T^2$)
Γ	specific volume on the critical state line in the $p' - q$ plane at unit p'
γ'	the effective unit weight of soil ($ML^{-2}T^{-2}$)
γ_t	the total unit weight of soil ($ML^{-2}T^{-2}$)
γ_w	the unit weight of pore water ($ML^{-2}T^{-2}$)
δ_i	lateral displacement (L)
δ_{ij}	Kronecker delta (dimensionless)
Δ	average volumetric strain (dimensionless)
Δ_d	incremental nodal displacement (L)
ΔF_1	incremental nodal force (MLT^{-2})
ΔF_2	incremental nodal flow (L^3T^{-1})
Δt	time interval or time increment (T)
Δu	incremental nodal excess pore water pressure ($ML^{-1}T^{-2}$)
$\delta\varepsilon_q{}^e$	elastic shear strain increment (dimensionless)
$\delta\varepsilon_p{}^e$	elastic volumetric strain increment (dimensionless)
$\delta\varepsilon_q{}^p$	plastic shear strain increment (dimensionless)
$\delta\varepsilon_p{}^p$	plastic volumetric strain increment (dimensionless)
$\Delta\varepsilon_v$	incremental volumetric strain (dimensionless)
$\Delta\sigma_{vac}$	the incremental vacuum pressure ($ML^{-1}T^{-2}$)
ε_v	volumetric strain (dimensionless)
$\dot{\boldsymbol{\varepsilon}}_{ij}^{e}$	the (*i, j*) component of elastic strain rate tensor (T^{-1})
$\dot{\boldsymbol{\varepsilon}}_{ij}$	the (*i, j*) component of strain rate tensor (T^{-1})
$\dot{\boldsymbol{\varepsilon}}_{ij}^{vp}$	the (*i, j*) component of viscoplastic strain rate tensor (T^{-1})
$\dot{\varepsilon}_v$	the volumetric strain rate (T^{-1})
η	coefficient of viscosity of water ($ML^{-1}T^{-3}$)
η	stress ratio q/p' in modified Cam clay model (dimensionless)
η^*	stress parameter in Sekiguchi and Ohta model (dimensionless)
θ	interface transmissivity (L^2T^{-1})
θ_b	the Lode angle (°)
θ_y	Lode angle for the kinematic yield surface (°)
κ	slope of unloading-reloading line in $e - \ln p'$ plot (dimensionless)
κ^*	slope of unloading-reloading line in $\ln v - \ln p'$ plot (dimensionless)
Λ	$1-\kappa/\lambda$
λ	slope of virgin compression curve in plot $e - \ln p'$ (dimensionless)
λ^*	the slope of isotropic normal compression line in $\ln v - \ln p'$ space (dimensionless)
μ_p	parameter representing effects of spacing, smear and well resistance in consolidation theory of a plane strain unit cell of a vertical drain (dimensionless)

ν	Poisson's ratio (dimensionless)
ν	angle of dilation (°)
σ	confining pressure ($ML^{-1}T^{-2}$)
$\boldsymbol{\sigma}'$	effective stress tensor (ML^{-1} T^{-2})
σ_3	minimum principle stress or confining stress for triaxial compression ($ML^{-1}T^{-2}$)
σ'_h	in situ horizontal effective stress ($ML^{-1}T^{-2}$)
σ'_v	in situ vertical effective stress ($ML^{-1}T^{-2}$)
σ'_{h0}	initial value of in situ horizontal effective stress ($ML^{-1}T^{-2}$)
σ'_{v0}	initial value of in situ vertical effective stress ($ML^{-1}T^{-2}$)
σ'_x	normal effective stress in *x*-direction ($ML^{-1}T^{-2}$)
σ'_y	normal effective stress in *y*-direction ($ML^{-1}T^{-2}$)
σ'_z	normal effective stress in *z*-direction ($ML^{-1}T^{-2}$)
Φ	hydraulic conductivity matrix ($M^{-1}L^4T$)
ϕ	friction angle of soil (°)
ϕ'	effective stress friction angle of soil (°)
Ψ	parameter in hardening function (dimensionless)

Chapter 1
Introduction

Abstract The book begins with a chapter describing some of the various ground improvement technologies that are in current use in soft ground engineering. Not only does it provide brief descriptions of these technologies, but it also makes the argument that predicting the effects on subsequent ground performance of implementing these technologies is important in engineering practice. Clearly, one important effect concerns the ground deformations resulting from their use. This particular issue is examined in detail in the subsequent chapters of this book. The obligation to validate and calibrate the various analytical and numerical solutions presented in this book by comparing their predictions with experimental data is acknowledged. Indeed, the book includes a large number of case studies and field trials used for this very purpose.

1.1 What is Ground Improvement and When and Why Is It Necessary?

The selection of a suitable site for construction is one of the most important considerations in almost all civil engineering projects. From the point of view of geotechnical engineering, a suitable site usually means one where the foundation soil (or rock) has sufficient strength and stiffness to carry the loads that will be imposed on the ground without causing unacceptably large deformations or stability problems, or else the foundation material has drainage properties suitable for the particular construction application.

However, there are many cases in practice where the most suitable site has been determined by social or economic requirements, rather than purely geotechnical or other engineering considerations. For example, for ease of access to sea transport some of the world's largest cities have been located throughout the history of civilization in coastal areas, many of which are typically underlain by soft clay or clay-like deposits. Obvious examples of this type of city include Shanghai in China and Bangkok in Thailand, but there are many others that could also be listed. In these cases the soft soil deposits underlying these cities often have relatively low strength and high compressibility, sometimes so weak or so compressible that major infrastructure cannot be built directly on the ground as it exists in nature. In many cases, the mechanical properties of these soft deposits, e.g., their strength, stiffness and

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hydraulic conductivity, need to be improved by selected engineering methods, i.e., ‘ground improvement’ techniques, before any major components of infrastructure are constructed.

Furthermore, due to increased urbanization there is now a growing shortage of available land in many cities, so that new facilities such as airports and sea ports have had to be developed on land reclaimed from the sea. Well known examples include Kansai International airport in Japan, Changi International airport in Singapore, Hong Kong International airport and Incheon International airport in Korea. Since the newly reclaimed land of this type is often too weak to adequately support runways and terminal buildings, some form of ground improvement is normally required in these situations.

Another example of applications for which ground improvement is almost inevitably required occurs in railway and highway construction. Because of existing pressures on land use these transportation routes often pass through regions underlain by deposits of soft or weak soils. The mechanical properties of these softer soils often have to be improved in order to adequately support the associated earth structures and to reduce the residual settlements that occur during the useful life of the associated transport infrastructure.

1.2 Techniques of Ground Improvement

Numerous ground improvement techniques have been developed throughout the course of human history. Bergado et al. (1996) classified the various ground improvement methods according to whether or not additives are used to directly enhance the strength and stiffness of the ground. A slightly modified version of their classification scheme for these methods is as follows:

1. Work on the soil only (primarily to reduce the voids content)
   (a) Densification by applying external forces to coarse-grained soils
      (i) Surface compaction
      (ii) Vibro-flotation and vibro-compaction
      (iii) Dynamic compaction
      (iv) Resonance compaction
   (b) Improvement of the drainage of fine-grained soils (often combined with preloading)
      (i) Sand drains
      (ii) Prefabricated vertical drains (PVDs)
      (iii) Horizontal drains
2. Addition of other materials into the soil deposit
   (a) Soil reinforcement

(i) Mechanically stabilized earth (MSE)
(ii) Sand compaction piles (SCP)
(iii) Geo-piers (stone columns and granular piles)

(b) Use of chemical admixtures

(i) Deep mixing method (DMM) using lime and cement
(ii) Chemical piles

Of these methods, preloading in combination with the installation of prefabricated PVDs, and cement deep mixing (CDM), normally in order to form soil-cement columns in the ground, are widely used to improve the mechanical performance of soft clayey deposits (Bergado et al. 1996). Where preloading is used, the loads can be applied either directly to the soft ground as a surcharge (e.g., by the placement of embankment fill) or by applying a vacuum pressure to the soil (Chu et al. 2000; Tang and Shang 2000; Indraratna and Chu 2005; Indraratna et al. 2004; Chai et al. 2006).

1.3 Why Do We Need to Estimate Ground Deformations?

Almost all geotechnical designs require consideration of at least one or possibly a combination of the following three factors:

1. Strength – such as in the bearing capacity of foundations, slope stability, earthquake resistance of the ground, etc.;
2. Deformation – such as surface settlement, differential settlement, and lateral displacement of the ground and any associated geotechnical structures; and
3. Permeability or hydraulic conductivity – which is significant for water retaining structures like dams, and drainage systems, etc.

There are situations where deformations induced in the ground, or the structures constructed on the ground, become the controlling factors in a design, such as, for example, in problems involving the design of shallow foundations on sand, and also in the design and construction of pile foundations. Clearly, ground deformation has a direct influence on the serviceability, or in-service performance, of the affected structures. In the case of road embankments constructed over soft ground, settlement of the embankment, due mainly to vertical but also lateral deformations of the soft underlying ground, may become excessive. If these settlements become so pronounced that the elevation of the road surface is reduced to the point where the roadway becomes flooded during heavy rainfall, then clearly the serviceability and functionality of the roadway itself becomes compromised. Furthermore, differential settlements of embankments and foundations may induce cracks or inclination (tilting) of the supported structures, which may have adverse effects on the functioning of these structures. Therefore, predicting the settlement and lateral movement of soft ground under the influence of applied loads is often an essential design requirement.

Furthermore, under working conditions the deformation of an earth structure can be measured, at least in principle, but also often in practice. Such measurements can then be compared with the original design predictions. The advantage of conducting such comparisons should be obvious. Clearly, one important advantage is that they allow the predictive models and their underlying assumptions to be refined. Such refinement is common and often essential in geotechnical practice and reflects the fact that the calculation of deformations for real engineering projects is often an iterative process.

1.4 What Is This Book All About?

This book deals with a particular class of deformation problems and the methods of analysis that may be applied to such problems. The focus is on the deformation of soft clay deposits or clay-like deposits whose mechanical properties and performance have been enhanced by various forms of ground improvement. Particular emphasis has been placed on predicting the deformation response of ground improved by the installation of prefabricated vertical drains (PVDs) and soil-cement columns formed by deep cement mixing (DCM). The behaviour of ground treated in this way and then subjected to embankment loading and/or vacuum pressure loading is specifically addressed. Various theories and numerical modelling techniques that can be applied successfully to the analysis of this class of geotechnical problem in soft ground are described.

Since the development of predictive techniques for estimating soil displacements is underpinned by, and critically dependent on, the constitutive models adopted to represent the soil response, it seems only natural to describe some of these fundamental tools that have been used successfully to date for this task. This is the rationale for including Chapter 2, which contains a description of some of the techniques that have been used successfully in modelling the mechanical response of soft clay subsoils. Included in this treatment are some of the fundamental stress-strain models used to describe the behaviour of clays as well as their implementation in the finite element method (FEM). Also presented in this chapter are example analyses to demonstrate the power and significance of these modern numerical techniques

Chapter 3 contains a description of the methods used for modelling soft subsoils whose mechanical performance, particularly their drainage characteristics, have been improved by the installation of pre-fabricated vertical drains (PVDs). Included is a description of how a typical problem may be dealt with using an equivalent plane strain finite element analysis. Also presented are some example analyses using specific case histories, and comparisons of the model predictions with the field performance of PVD systems.

Chapter 4 presents the theory of vacuum consolidation and describes the major characteristics of vacuum consolidation of soft ground, the methods adopted for calculating the deformations induced by application of vacuum pressure, as well as some example analyses based on case histories.

Chapter 5 describes a consolidation theory for clayey subsoil improved by the installation of soil-cement columns. It also describes methods for estimating ground settlements, mainly for the case of ground improved by the prior installation of ‘floating’ columns, a method for predicting the lateral displacement of the ground induced by column installation, and the analyses of some relevant case histories.

Many of the techniques for deformation analysis presented in Chapters 2, 3, 4, and 5 involve approximations of reality, sometimes quite crude approximations. These have been clearly and deliberately identified. In many cases it has been demonstrated, by presenting comparisons of predictions with field data, that these approximations provide a reasonable balance between tractability of the problem at hand and the accuracy and reliability of the resulting deformation predictions. However, there is still a need for considerable further research in this area of geotechnology. Much still needs to be done to increase our predictive capabilities and particularly the accuracy of our predictions of ground deformations associated with common ground improvement techniques. It is anticipated that this need will only continue as further advances are made in the technology of ground improvement. Chapter 6 provides some suggestions for future research directions, which hopefully will address some of the present shortcomings or gaps in our current predictive abilities.

It is the deliberate intent of the authors that this book should provide up-to-date information on this important subject from both the theoretical and practical points of view.

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Chapter 2
Modelling Soft Clay Behaviour

Abstract Since the development of predictive techniques for estimating soil displacements is underpinned by, and critically dependent on, the constitutive models adopted to represent the soil response, it seems only natural to describe some of these fundamental tools that have been used successfully to date for this task. This is the rationale for including Chapter 2, which contains a description of some of the techniques that have been used successfully in modelling the mechanical response of soft clay subsoils. Included in this treatment are some of the fundamental stress-strain models used to describe the mechanical behaviour of clays as well as their implementation in the finite element method (FEM). Also presented in this chapter are example analyses to demonstrate the power and significance of these modern numerical techniques. Case histories from Malaysia and Japan are discussed in relation to this form of numerical modelling.

2.1 Introduction

In order to simulate accurately the mechanical behaviour of a clay deposit using mathematical techniques, it is most important to select an appropriate constitutive model and correct values of the model parameters, as well as an accurate assessment of the initial effective stress state in the soil deposit. In order to do so, an objective method should be established for first checking whether the chosen soil model together with the corresponding soil parameters and assumed in situ stress state can simulate, to acceptable accuracy, the stress-strain relationships of the deposit.

An important quantity to determine in most soft ground engineering problems is the undrained shear strength of the clay or clay-like soil. In many geotechnical investigations of soft clay sites, the undrained shear strength (S_u) of the soil deposit is often measured using the field vane shear test, or estimated from the results of cone penetration tests, or else measured in an unconfined compression test or a triaxial compression test using undisturbed soil samples. These measurements to determine the profile of undrained shear strength with depth below the ground surface provide a simple way to check whether the adopted stress-strain model, as well as the corresponding parameters and initial effective stress, are appropriate. In particular, they

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provide a means of checking whether together the assumed soil model, parameter values and initial in situ stress state can provide accurate predictions of the profile of S_u. This process may also be adopted to assess whether it is necessary to make allowance for strain rate effects on the undrained shear strength (Bjerrum 1972, 1973).

Embankments are often constructed as major components of road, railway and dike projects, and therefore they are normally regarded as important geotechnical structures. It is now quite common for embankments constructed on soft clay soils to incorporate some form of ground improvement, such as the installation of prefabricated vertical drains to accelerate the rate of consolidation, or deep cement mixing to reduce the settlement and increase the stability of the embankment.

A theory for analyzing the stresses and deformations induced in the ground under an embankment load is available for the special case where it is assumed that the ground is semi-infinite, homogeneous and elastic (Gray 1936, Giroud 1968). However, in most practical cases the subsoil is neither homogeneous nor elastic, especially if ground improvement techniques have been used to treat the soft clayey subsoil. Most practical problems are much more complicated than the ideal cases for which closed-form solutions exist for the stresses and deformations of the embankment and underlying ground.

The finite element method (FEM) can take into account many of the complicated subsoil and boundary conditions. In particular, it can include elastoplastic and elasto-visco-plastic models of the behaviour of soil, provided an appropriate constitutive model is selected. The FEM has been widely used in the analysis of geotechnical boundary and initial value problems including the simulation of embankments constructed on soft subsoils. In addition to selecting a suitable constitutive model to represent the mechanical behaviour of the soil, there are several other factors that may influence the results of a FEM analysis of an embankment on soft subsoil. These include the methods adopted to simulate application of the embankment load and indeed the entire construction process including any ground improvement, as well as the possibility of including large deformations in the analysis.

In the following sections some of the constitutive soil models more commonly used to represent the mechanical behavior of soft clayey soils are described, as well as their corresponding predictions of the values of undrained shear strength, S_u. Some example analyses are also presented to illustrate the importance of checking the simulated S_u profile before launching in to a complex non-linear analysis of embankment construction. Methods used in FEM analysis to apply embankment load, simulating the embankment construction process and techniques to take into account the effects of large deformation phenomena are also described. Finally, further complications such as the effect of the load application method on the foundation response, the effect of foundation deformations during construction on the factor of safety of the embankment and large-deformation-induced buoyancy effects are discussed and illustrated though the use of example analyses.

2.2 Initial Stiffness and Undrained Shear Strength

2.2.1 Constitutive Models for Clay Soils

It is not intended in this section to provide a comprehensive review of all constitutive models for soft clay soils nor to suggest the "best" soil models for clay soils. Rather, the intention is to describe a selection of the constitutive models that are believed to be of use in representing the mechanical response of soft clays and to demonstrate the importance of particular numerical modelling techniques when simulating the behaviour of a soft clayey subsoil. The Modified Cam Clay model (Roscoe and Burland 1968), the Sekiguchi-Ohta model (Sekiguchi and Ohta 1977) and a generalized two-surface "bubble" model (Grammatikopoulou 2004, Grammatikopoulou et al. 2006) are described in some detail, as well as the equations they provide for calculating the undrained shear strength S_u.

2.2.1.1 Modified Cam Clay (MCC)

The MCC model (Roscoe and Burland 1968) is one of the most widely used models for soft clay soils. As with any elasto-plastic model, MCC has four major components and these are:

1. an elastic response and associated elastic properties;
2. a yield surface defining the boundary between elastic and elastoplastic soil behaviour;
3. a plastic potential function which is used to define the relative magnitudes of the plastic strain increments; and
4. a hardening rule which allows the magnitudes of the plastic strain to be calculated.

MCC assumes that the recoverable volumetric strain ($\delta\varepsilon_p^e$) can be expressed as follows:

$$\delta\varepsilon_p^e = \kappa \frac{\delta p'}{(1+e)\,p'} \tag{2.1}$$

where κ = the slope of unloading-reloading curve in e–ln(p') plot, e = voids ratio, p' = the mean effective stress and $\delta p'$ = the mean effective stress increment. It is also assumed that the recoverable shear strain ($\delta\varepsilon_q^e$) is a linear function of deviator stress (q) increment δq:

$$\delta\varepsilon_q^e = \frac{\delta q}{3G'} \tag{2.2}$$

where G' = the elastic shear modulus. Equations (2.1) and (2.2) imply a variation of Poisson's ratio with mean effective stress, but the alternative assumption of a constant value of Poisson's ratio is also possible and often adopted in practice.

For MCC the yield locus in the $p'-q$ stress plane is an ellipse which is described by Eq. (2.3):

$$\frac{p'}{p'_0} = \frac{M^2}{M^2 + \eta^2} \tag{2.3a}$$

where $\eta = q/p'$. The form of this yield surface (*f*) can also be written as:

$$f = q^2 - M^2 \left[p' \left(p'_0 - p'\right)\right] = 0 \tag{2.3b}$$

where $M =$ the slope of critical state line (CSL) in $p'-q$ stress plane, and $p'_0 =$ the size of yield focus on the mean stress axis. Under a state of triaxial compression, M is related to the friction angle (ϕ') as follows:

$$M = \frac{6\sin\phi'}{3 - \sin\phi'} \tag{2.4}$$

Equation (2.3) describes a set of ellipses, all having the same shape (controlled by the value of M), all passing through the origin, and having sizes controlled by the value of p'_0. When the soil is yielding, the change in size of the yield locus, p'_0, is linked with the changes in effective stresses p' and $q = \eta p'$, through the differential form of Eq. (2.3):

$$\frac{\delta p'}{p'} + \frac{2\eta\delta\eta}{M^2 + \eta^2} - \frac{\delta p'_0}{p'_0} = 0 \tag{2.5a}$$

or

$$\left(\frac{M^2 - \eta^2}{M^2 + \eta^2}\right)\frac{\delta p'}{p'} + \left(\frac{2\eta}{M^2 + \eta^2}\right)\frac{\delta q}{p'} - \frac{\delta p'_0}{p'_0} = 0 \tag{2.5b}$$

It is assumed that the soil obeys the normality condition, and the plastic potentials (g) are the same as the yield functions (f) in the $p'-q$ plane:

$$g = f = q^2 - M^2 \left[p' \left(p'_0 - p'\right)\right] = 0 \tag{2.6}$$

In the MCC model, a linear relationship is assumed between the specific volume ($v = 1 + e$) and the logarithm of mean effective stress p'_0 during isotropic normal compression of the soil:

$$v = N - \lambda \ln p'_0 \tag{2.7}$$

where $\lambda =$ the slope of the virgin loading curve in the $e - \ln(p')$ plot, $N =$ the specific volume on the isotropic compression line at unit mean stress, i.e., $p' = 1$ kPa (Fig. 2.1). It follows then that the magnitude of the plastic volumetric strain is given by:

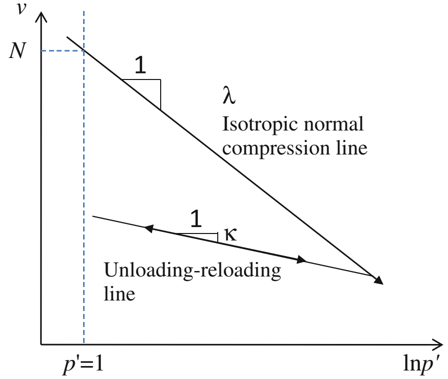

Fig. 2.1 Specific volume and mean effective stress relationship

$$\delta\varepsilon_p^{\mathrm{P}} = \left[(\lambda - \kappa)/v\right]\frac{\delta p'_0}{p'_0} \tag{2.8}$$

and the hardening relationship becomes:

$$\frac{\partial p'_0}{\partial \varepsilon_p^{\mathrm{P}}} = \frac{v p'_0}{\lambda - \kappa} \tag{2.9}$$

In particular, the MCC model predicts the undrained shear strength (S_u) of the soil as follows:

$$S_u = \frac{p'}{2^{1+\Lambda}} M \left(\frac{M^2 + \eta^2}{M^2}\right)^{\Lambda} (OCR)^{\Lambda} \tag{2.10}$$

where $OCR =$ overconsolidation ratio, and $\Lambda = 1 - \kappa/\lambda$.

2.2.1.2 Sekiguchi–Ohta Model

This is an anisotropic Cam clay-type model with an associated flow rule, which also allows for viscosity in the formulation. Sekiguchi and Ohta (1977) introduced a new stress parameter (η^*) to model the shear stress-induced dilatancy of clays as follows:

$$\eta^* = \sqrt{\frac{3}{2}\left(\boldsymbol{\eta}_{ij} - \boldsymbol{\eta}_{ij0}\right)\left(\boldsymbol{\eta}_{ij} - \boldsymbol{\eta}_{ij0}\right)} \tag{2.11}$$

in which

$$\boldsymbol{\eta}_{ij} = \frac{\boldsymbol{s}_{ij}}{p'},\ \boldsymbol{\eta}_{ij0} = \frac{\boldsymbol{s}_{ij0}}{p'_i} \tag{2.12}$$