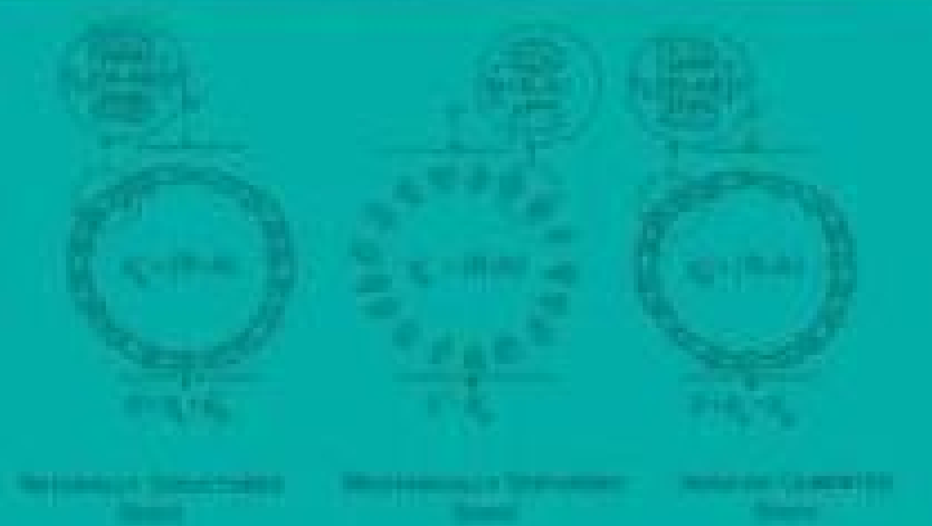
SOFT CLAY BEHAVIOUR

ANALYSIS AND ASSESSMENT



T.S. NAGARAJ and NORIHIKO MIURA

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Soft Clay Behaviour
Analysis and Assessment

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Preface

As a civil engineering material, soil is as important as steel or concrete. The unique feature of this material is that it largely comes engineered by nature, unlike other materials which have to be manufactured or processed to specific requirements with the product quality ensured. So geotechnical engineers concentrate on the cardinal task of characterizing the properties of this natural large volume material, before integrating the results with analysis to design contemplated structures to meet the strict performance requirements.

This book is primarily concerned with the analysis and assessment of the behaviour of soft clay in compression and shearing, both in their undisturbed and strengthened states. Soft clay deposits are widespread and cover many coastal regions of the world, such as in Japan, Eastern Canada, Norway, Sweden and other Scandinavian countries, India, and the South East Asian countries. Another feature of these coastal regions is that certain shore areas are lowlands, quite often due to subsidence on account of excessive withdrawal of ground water. The Central Plain (Chao Phraya) of Thailand and Saga Bay in Southern Japan are some examples of lowland areas. All these are inhabited zones, having undergone extensive urbanization and industrialization. Further extensive developmental activities are apace in many lowland areas to promote human activities, such as agriculture, industry, housing and such other infrastructure facilities.

Generally, soft clays exhibit low strength and high compressibility. Many are sensitive, in the sense that their strength is reduced by mechanical disturbance. Foundation failure is likely in structures erected in soft clays. Surface loading beyond yield stress levels due to embankments and shallow foundations inevitably results in large settlements. It is primarily because these soils respond in a spectacular manner to stress changes, that the geotechnical engineer has an obligation to examine to what extent soft clay behaviour can be analyzed within the framework of classical developments in soil mechanics.

This book is intended to meet the above requirement. It incorporates the results from extensive research into basic and applied aspects of soft clay behaviour and engineering conducted at the Indian Institute of Science, Bangalore, India and the Institute of Lowland Technology, Saga, Japan for over a decade. It has been possible to modify the approaches wherever needed and feasible. Apart from interpreting and generalizing the extensive data generated and published by many

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other institutions and organizations, this book presents simple working methods for the systematic analysis and parametric assessment of soft clays at the engineering level. This has been made possible by ingeniously advancing micro-mechanistic axioms concerning the fundamentals of soil behaviour to engineering levels. It is believed that attainment of this capability is commensurate with the computational tools and capabilities in use to solve practical problems involved with soft clays both in their natural and induced cemented states.

Chapters 1 and 2 introduce the subject, describe practical methods for handling soft clays in engineering practice and justify the need for analysis and assessment of the engineering properties of soft clays. Chapter 3 presents the basic framework for analysis. Chapter 4 deals with the analysis of soft clays in their uncemented state, which is a necessary prelude for analysis of the behaviour of soft clays in their naturally and induced cemented states, to which Chapters 5 and 6 are devoted. The discussion in Chapter 6 is timely since induced cemented states realized through in-situ deep mixing methods are extensively used in Japan as one of the prominent means of soft ground improvement in soft clay engineering.

We are aware that we have drawn a bold picture with a broad brush and that a few of our interpretations and generalizations may not be based on extensive data. Some of the inferences might be controversial. Some other aspects of soft clay behaviour are not dealt with in this book. Nevertheless, we decided, to present some phenomenological approaches for the analysis of soft clay behaviour, with the hope that it would perhaps draw the attention of geotechnical engineers and lead to more rigorous studies. What we present at this juncture itself offers a simple framework and a working approach with which the engineering properties of soft clays can be assessed in day to day routine investigations.

This book will serve the interests of post-graduate students, researchers and practising engineers. It may also serve as course material, on a selective basis; for example, Chapter 2 on soft clay engineering could be a part of a course on ground engineering, and Chapters, on uncemented saturated soft clays – stress and time effects and naturally cemented soft clays, could form part of the coverage under basic geomechanics.

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We would like to acknowledge all those who have contributed to this book both at the Indian Institute of Science, Bangalore, lndia and at the Institute of Lowland Technology, Saga University, Saga, Japan. This book was planned while the first author was a visiting Professor at the Institute of Lowland Technology in 1996. Our research and teaching efforts in this broad area have further helped to attempt an in-depth treatment of the subject and the coverage provided in this treatise.

At all stages of the preparation of the draft of this book, we have received a great deal of assistance from colleagues and research students. Thanks are due to our esteemed colleagues for their help at various stages of the preparation of the manuscript. Specific mention is made of the painstaking efforts at critical review of the manuscript by Dr Nagendra Prasad, Associate Professor, Department of Civil Engineering, SV University at Tirupati, India. The manuscript was checked at various stages by Dr G.L. Sivakumar Babu, Assistant Professor, Department of Civil Engineering, Indian Institute of Science, Bangalore and by Dr R.G. Robinson, Research Associate at the Indian Institute of Science, Bangalore who is presently Post Doctoral Fellow, National University of Singapore.

We place on record our sincere thanks to Mr A.V. Narayan, Senior Draughtsman, Department of Metallurgy of the lndian Institute of Science, who meticulously prepared all illustrations needed for the book, and Mr P. Raghuveera Rao who rendered a helping hand at various stages during formatting of tables and references.

The first author places on record his sincere thanks to the Curriculum Development Cell of the Centre for Continuing Education, Indian Institute of Science, Bangalore, India, for providing financial support for the preparation of this book and All India Council for Technical Education, India (AICTE) for Emeritus Fellowship during which period major portion of the draft was prepared.

Finally, we alone remain responsible for any lapses and errors that might have crept into the text inadvertently despite our best efforts to be free of them. We welcome comments and suggestions from the readers.

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Introduction

'Analysis related to science is meaningful
Assessment explained by basics is sound'

In addition to analysis and design, engineering involves decision making on the basis of available information. It is primarily an exercise in judgment regarding
- the technical feasibility of the project,
- the economic implications and the time requirement for completion,
- the acceptability of the solution anticipated, and
- the performance of the final outcome.

In all these the engineer is constantly required to predict. In fact, prediction is the cornerstone of all innovations and hence an integral component of advanced engineering practice. Geotechnical engineering is no exception.

All structures, except those which float or fly, rely upon soil deposits and/or rock formations for their support. Geo-materials and water exert pressure on or are utilized in construction, thus affecting the safety of all structures. Despite this significance, the word 'soil' generally implies that it makes up the ground on which we live and makes us dirty. Most people take soils for granted and are not overly concerned. However geotechnical engineers are one group who are deeply concerned with soil, the others being geomorphologists, geologists, hydrologists, agronomists and more.

Soft clays in the geo-material spectrum

The inherent nature and diversity of the geological processes involved in soil formation are responsible for the wide variability in the in-situ state of soils. In the continuous geo-material spectrum, very soft and soft clays form one end of the spectrum with the other end being extremely hard rock. This book deals primarily with soft clays behaviour, their analysis and assessment to meet the specific needs of soft clay engineering.

Study of field situations, analysis and design with the incorporation of appropriate material properties and, finally, construction, are different chain of events in the practice of geo-engineering. In the realistic assessment of soft clay engineering properties, the frequent questions for which answers sought are:

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1. What is the state of in-situ soft clay? Is it normally consolidated, overconsolidated, or sensitive to quick clays?
2. Is sample disturbance a critical factor? If yes, how can the degree of mechanical disturbance be assessed and accounted for in property characterization?
3. How to assess the yield stress and account for it in compression and recompression indices at different stress levels?
4. Can the stress-strain-pore water pressure/volumetric strain response of naturally cemented soft clays be fitted into the response of stress-dependent overconsolidated clays?
5. What are the stress-state-permeability relations in cemented soft clays?
6. Can the response of soft clays be altered to desired levels, e.g., reduced compressibility and enhanced shear strength, without necessarily altering their in-situ state, i.e., their in-situ water content?

Practical requirements

It has been the ardent desire of geotechnical engineers to find simple and quick methods to assess the engineering properties of soft clays both on-shore and off-shore, quite often to significant depths as dictated by the specific needs. The need for simple and reliable means of testing is even more desired, since only a minute fraction of soil can be tested because of practical and economic constraints. For example, even if the spacing of the bore hole in natural deposits is as close as 10 m and a 50 mm diameter sample is procured and tested at every alternate meter, only one millionth of the total volume would have been explored. Further, speed and accuracy of assessment of the properties are prime considerations in the current engineering practice. So any attempt to assess, interpret and generalize soil behaviour will provide the opportunity to (Nagaraj et al. 1994).

- form an independent evaluation of the laboratory investigations,
- increase the level of confidence in handling extensive data,
- optimize time and cost,
- monitor the progressive attainment of desired changes in soft ground while ground improvement techniques are employed, and
- employ observational approaches in geotechnical engineering more effectively.

Until now, engineers and researchers have adopted the following two approaches in the analysis and assessment of soil behaviour.

1. The engineering approach in which empirical methods of correlation essentially based on experimental observations are developed. These are often designated as rules of thumb, whose use is restricted to specific regions.
2. Micromechanistic approaches which lean towards basic sciences. The nature and equilibrium of forces are given due consideration at micro-scopic and sub-microscopic levels along with interactions between different phases of multi-phase systems.

A large number of technical publications of the recent past testify to the intensity of the study pursued by these approaches. Despite the great strides made inde-

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pendently, they have not always been complementary to each other. In the first approach of empirical property correlation, a rational basis for interpretations and interrelations cannot always be discerned. In the second approach, micro-parameters identified through micro-mechanistic means are not always amenable for easy measurement. Further micro-mechanistic axioms cannot always be translated into commonly used macro-parameters for analysis and assessment. Perhaps, the most desirable approach would be one that:

- identifies the basic mechanisms underlying the observed response of the material under different stimuli,
- integrates the empirical and basic approaches, in both behavioral and parametric, through easily measurable engineering parameters, and
- formulates phenomenological models for the analysis and assessment of the engineering properties of soils to meet time constraints without sacrificing the accuracy needed at engineering level.

The relevance of such an approach has been amply elaborated by Calladine (1969) as early as 1969. The possibility of arriving at such models capable of both analysis and prediction has been recently demonstrated by Nagaraj et al. (1994).

The cardinal aim of this treatise is to present how the observed experimental findings can be synthesized to establish new and simple approaches in a holistic manner for the analysis and assessment of soft clay behaviour. This would be in tune with the phenomenal changes taking place in geotechnical engineering at present. Ground improvement techniques in general and use of geosynthetics in particular provide satisfactory means to solve most of the problems associated with soft clays. So the emphasis has shifted to fast and efficient methods of property characterization. In fact this is going to be the trend in geomechanics in the 21st century (Zaman et al. 1992).

To achieve rapidity in the assessment of the engineering properties of soils in-situ testing methods have received due attention. Penetration testing is one of the key methods which takes care of site exploration as well as enabling us to determine the soil properties. This is a distinct deviation from the elemental laboratory testing of undisturbed samples to assess the engineering properties for geotechnical design purposes. The cone penetration test and its variant the piezocone test are the most common penetration tests performed throughout the world. Lunne et al. (1997) have published an excellent and exhaustive treatise on cone penetration testing in geotechnical practice. The authors (Lunne et al. 1997) provide their recommended guidelines to interpret the entire range of geotechnical parameters from cone penetration data.

Most of the present day conventional portions in soil mechanics will not be covered in this book. Hence a general background in the principles of soil mechanics is a prerequisite to appreciate and use this book. However, Chapter 1, covering soils as engineering materials, could be considered as a recapitulation of the essentials of soil mechanics.

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CHAPTER 1

Soils as engineering materials

1.1 INTRODUCTION

In geotechnical engineering, soil comprehends the entire thickness of the earth's crust which is accessible and feasible for practical utilization in tackling practical problems. In general, soils are encountered in at least two ways.
1. in their in-situ state from which structures derive their support.
2. as construction materials for dams, embankments and other earth structures.
In many other engineering activities, one would normally state the requirements and specifications of materials and achieve these by appropriate processing technology. In contrast to this, geotechnical engineers usually have to adjust their design in accordance with the prevailing properties of large volumes of in-situ materials. One redeeming feature is that with the development of a wide spectrum of ground improvement techniques, it is possible to approach the first situation to a limited extent. It is unnecessary to stress that innovative designs and adoption of appropriate ground engineering methods can effect great flexibility and economy.

1.2 SOIL FORMATION

The word 'soil' originates from the Latin works 'Solum' which means the same as that of the present terminology. Broadly, in-situ deposits may be regarded as uncemented to partly cemented material differentiated into horizons dependent upon the mode of their formation. Soils are primarily formed by the natural cycle of weathering of earth's crust by mechanical or chemical agents of erosion, transportation, deposition and compression by later sediments. Such phenomena have been recognized by geologists for a long period (e.g., Holmes 1965). That part of the geological cycle up to the recompression stage leads to the formation of engineering soils.

In a general sense the atmosphere, biosphere and hydrosphere are the reactors in the weathering process, with the surface of the lithosphere producing the earth's crust. Weathering is a continuous reaction involving geological material and the environment with a change towards a decrease in the free energy of the system (Carroll 1970). If rocks are only physically degraded by wind, water and

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ice, the composition of soils are the same as that of the parent rocks. It is also the case that the shape and texture of the mineral grains reflect the history of degradation, transportation and deposition. When chemical changes occur, the basic rock forming minerals get altered to clay minerals. This fraction of soil most decisively controls the formation of soft deposits and their engineering behaviour, even if the percentage of clay in soil is far less than that of sand and silt.

1.3 SOIL COMPOSITION AND STATE

At the first level of formation, soils are made up of smaller units identified as individual grains or their aggregated units. The determination of size, shape and specific surface forms the first level of characterization. The material in bulk could be either single component or multicomponent. Nevertheless the range of particle sizes in engineering soils is very large. The smallest unit is of 10 Angstrom (1 nano-meter) size and extends up to 2 mm in size with a ratio of one to one fiftieth billion. The tremendous magnitude of this size range in soils can better be grasped by considering this analogy: when the smallest particle is compared to a pea of 3 mm in size, the size of the Earth corresponds to sand particle, the ratio of which is also close to one fiftieth billion. There is no other engineering particulate material with this wide range of particle sizes in bulk. The distribution of particle sizes is determined by combined mechanical and sedimentation methods. Brief descriptions of different methods of determining particle sizes and their critical reappraisal are provided elsewhere (Nagaraj 1993). The distribution of particle size as a grading curve is the percentage finer than that of a given size plotted against the size in mm. The grading curve is flat when the soil contains a wide spectrum of particle sizes. This is a well graded situation. If the curve is steep and one size dominates then the soil is poorly graded. Principal particle size classifications are cited in Figure 1.1.

The various subdivisions or fractions chosen for the classification as depicted in Figure 1.1 are purely arbitrary. The term clay is intended to reflect only clay sized particles and not with any associated plasticity characteristics. Since it is not possible to distinguish between a fine rock flour in these size ranges and active clays, many engineers avoid the use of the term clay to indicate that fraction of soil falling in the size range of 0.005 to 0.002 mm. The term clay size avoids this possible confusion. An experienced engineer can obtain valuable indications of the properties from grain size distribution data in conjunction with other information such as the bulk density, natural water content, consistency limits and the geological origin of soils.

The uniformity coefficient C_u, which is the ratio of D_{60} to D_{10} (D_{60} and D_{10} are diameters at 60% and 10% finer by weight respectively determined from particle size distribution curve of the soil), is an index which indicates the degree to which the particles are all of the same diameter. C_u, is unity, if all particles are of the

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same size with the general rule being that increasing values represent an increasingly wide range of size differences. The coefficient of curvature, C_{cur}.

$$C_{cur} = \frac{D_{30}^2}{D_{60}\,D_{10}} \tag{1.1}$$

describes the smoothness and shape of the gradation curvature. D_{30} is the diameter at 30% finer by weight on the particle size distribution curve. Very high or very low values indicate that the curves are irregular.

Particle shape: Closely related to particle size is particle shape since size measurements can hardly be complete without shape considerations. When the shape deviates from spherical it is expressed as axial ratio. The longest intercept is a, the widest part in the direction perpendicular to the long axis, b, and the short diameter from the position where b is measured is c (Fig. 1.2). These dimensions are in mutually perpendicular directions but do not necessarily intercept at the same

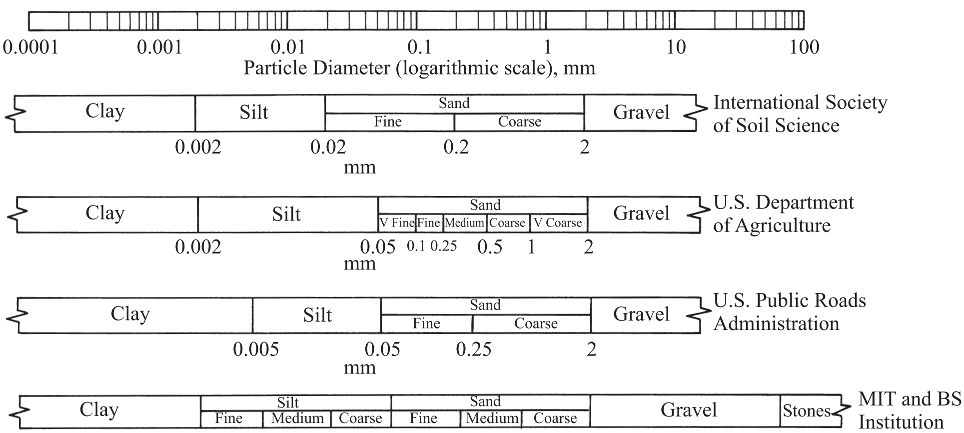

Figure 1.1. Principal particle size classifications.

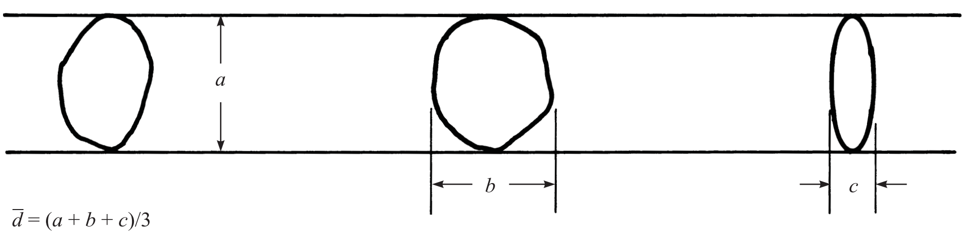

Figure 1.2. Characteristic dimension of the particle.

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point. The characteristic size is the mean of these values i.e., $d = (a + b + c)/3$. Soil particle shapes differ considerably. Clay particles occur as very thin plates while silt and sand are more round.

The shapes of the individual units markedly influence the potential of the particulate dispersal systems with respect to their orientation and contacts under the action of external stresses and lateral constraints.

Surface area: If a soil particle, say a coarse sand grain, is subdivided, its surface area increases geometrically while its solid volume or weight is constant. Eventually particle size can be reduced to the clay size range. Thus acquired surface area is reckoned as area of particle surface per unit weight. Unless the clay size particles possess an electrochemical charge its behaviour is not akin to clays. For expanding clays like montmorillonite, both exterior and interlayer areas constitute the specific surface, whereas for a nonexpanding clay mineral like kaolinite, only exterior faces make up for its relatively low values. The surface effects of large specific surface due to physico-chemical interactions merit examination.

A specific surface of 2.5 m^2/g has been suggested as the lower limit of the colloidal range. With the diminishing size of the particle, the surface area increases markedly (Table 1.1).

Surface effects: It has been established that the surfaces of soil solid particles carry electrical charges and hence electrostatic interactions with dipolar water molecules prevail. These surface charges give rise to interparticle forces in addition to the intrinsic weight of the soil grains. The magnitudes of the surface forces are proportional to the surface areas of the grains while the weight forces are proportional to the volume of the grains. As the particle size varies, since the surface forces diminish with the square of the particle diameter whereas the weight forces diminish with the cube, consequently the effects of surface forces are relatively more important in fine grained soils than in coarse grained soils. Because of the importance of the effects of surface -activity on the behaviour of fine grained soils, a description of these materials only with reference to particle sizes is practically meaningless. The practical distinction between silt and clay is made not on the basis of an arbitrary size distribution but on the basis of their interaction with water.

Table 1.1. Specific surface of soil particles.

Soil	Specific surface, m^2/g
Clean sand	0.001
Fine sand	0.01
Kaolinite	10-20
Illite	60-100
Montmorillonite	300-500

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1.3.1 *Consistency limits of soil*

It has been demonstrated by Casagrande (1932) as early as 1932 that in the case of coarse grained soils the gradation of soil particles is important for the characterization of the state and engineering properties. For fine grained soils additional parameters arising out of the interaction between soil and water are needed for their characterization. The consistency limits of soils satisfy this requirement. It is essentially a manifestation of surface effects on the state at various water contents. Water content is measured by weighing a sample before and after it is heated to 105°C for sufficient time for it to reach constancy in weight. The water content, w, is the ratio of the weight of water to weight of dry soil expressed as a percentage. In at completely dry state, the soil block may be hard like a solid, while at high water contents it may be almost a slurry (liquid). Intermediate states of consistency are semisolid and plastic. In 1911 the Swedish soil scientist Atterberg (1911) defined the boundaries of these above states of consistency in terms of water content as detailed in Figure 1.3.

All limits are expressed as water contents in percentage. Realistically all these transitions in the rheological states are not abrupt, but gradual. The importance of these limits was realised neither by Atterberg's own field of agriculture nor in the allied area soil mechanics, until Terzaghi (1926) could see the importance and potential as applicable to soil mechanics and Casagrande (1932) developed the method for their determination. More recently the principles and potentials of these consistency limits have been critically reviewed in detail by Nagaraj & Srinivasa Murthy (1988).

Liquid limit (w_L): Of the three consistency limits, the traditional liquid limit and plastic limit tests are used as standard tests. The liquid limit is essentially a meas-

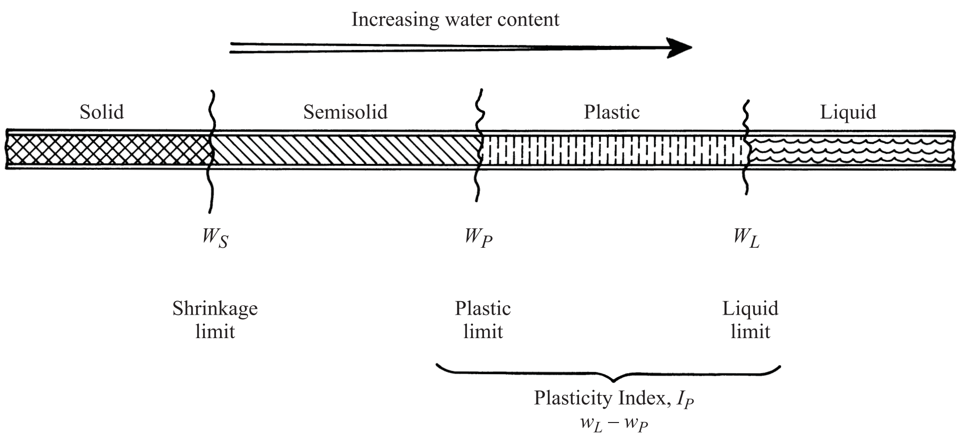

Figure 1.3. Consistency states of fine grained soils.

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ure of a constant value of lower strength limit of viscous shearing resistance as the soil approaches the liquid state, and it is expressed in terms of water content as a percentage. Percussion and cone penetrometer methods, described in most books on soil mechanics, can be used. Brief descriptions of these methods are as below:

Percussion method: Casagrande (1932) developed this method. The device used consists of a standard cup (Fig. 1.4) lifted through a standard height by a cam and dropped on a base of the standard material. With a spatula, soil paste is transferred into the cup and finished to get a smooth surface without air voids. A grooving tool is used to cut a groove of standard dimensions. The number of drops for half an inch (12.5 mm) closure of the groove is noted down. The water content of participating soil is determined. Tests are repeated for paste of different water contents. The linear plot of water content against number of blows on a logarithmic scale is used to identify water content for 25 drops. This is the liquid limit of the soil.

Cone penetrometer method: During the contemporary period as that of Casagrande, a fall cone test to determine the consistency was developed by the adoption of the Brinell Hardness Test (Hansbo 1957). The decision to use a free falling cone instead of pressing a cylinder or ball stemmed from the geometrical similarities of the impression being independent of the depth of the soil medium and elimination of the influence of rate of loading. A schematic sketch of the device can be seen in Figure 1.5. A cone of weight 80 grams with an apex angle of $30^{o} \pm 1^{o}$ fixed to a vertically sliding shaft is positioned with its tip just flush with the surface of clay filled in a standard cup. The cone is released for penetration under its own weight. Water content of the clay corresponding to 20 mm in 5 ± 0.5 sec is reckoned as the liquid limit of the soil. Considering the forces mobilized along the cone surface, the weight of the cone, *W*, and the depth of penetration, *d*, can be related to undrained strength in the form (Hansbo 1957):

$$c_u = k\frac{W}{d^2} \tag{1.2}$$

Where *k* is the proportionality constant for a given cone angle and rate of shear. Hence the cone penetrometer test is a strength test. Wroth & Wood (1978) and Wood & Wroth (1978) have strongly advocated, with evidence, the use of the cone method in place of the percussion method to determine the liquid limit water content. One of the recent significant findings (Nagaraj et al. 1987) in favour of the cone method is the possibility of computing the liquid limit water contents almost immediately without necessarily waiting for conventional oven drying for water content determination. This is due to the obvious fact that the amount of soil paste transferred to the cup is a function of both water content and the liquid limit of the soil and consequently operator independent. Hence in place of water content, bulk density can be determined just prior to cone penetration.

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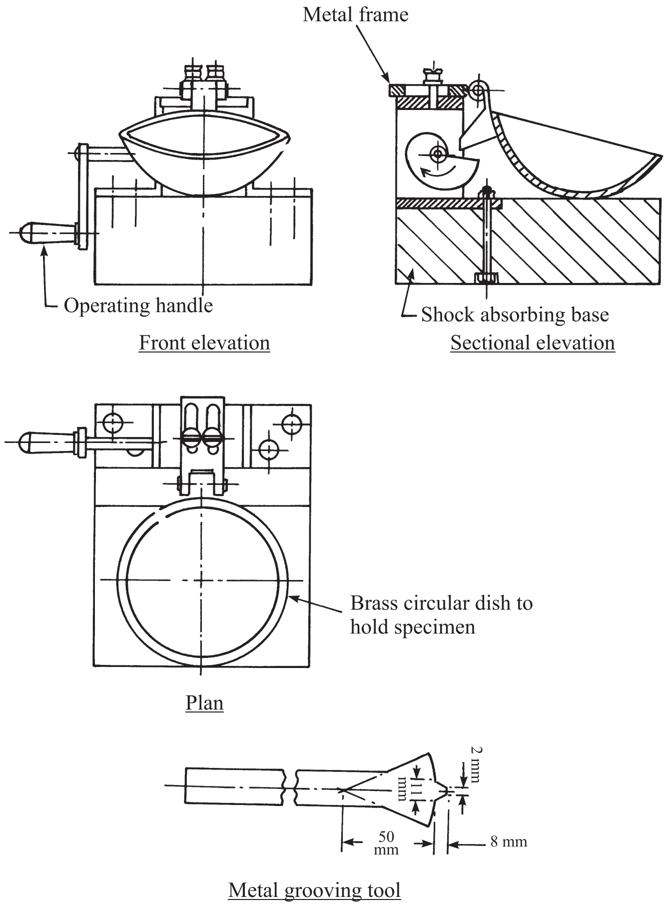

Figure 1.4. Percussion Casagrande's liquid limit device.

Plastic limit (w_P): This is the upper strength limit of consistency, representing the moisture content at which soil changes from a plastic to brittle state. The simple method suggested by Casagrande (1932) is rolling a thread of soil (on a glass plate) until it crumbles at a diameter of 3 mm. The moisture content at this stage is the plastic limit. If the thread can be rolled below this diameter of 3 mm it reflects wet side of the plastic limit, and the dry side if the thread breaks up and crumbles before it reaches 3 mm diameter.

The traditional plastic limit test despite assuming full saturation and incompressibility, has several inherent disadvantages (Whyte 1982) such as dependence of yield stress on:
– the pressure applied to the soil thread,
– the geometry i.e., the contact area between hand and thread,

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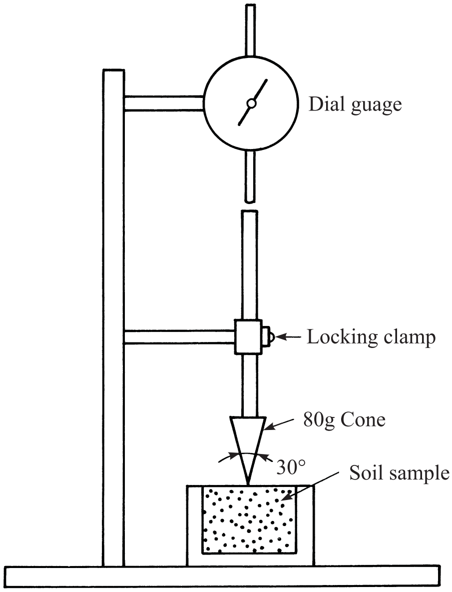

Figure 1.5. Cone penetrometer liquid limit device.

– the friction between soil, hand and base plate, and
– the speed of rolling.

Since none of these variables can easily be controlled, the test becomes highly operator dependent. A further shortcoming is that unlike the liquid limit test, the traditional plastic limit test does not provide a direct measurement of soil strength. Wroth & Wood (1978) and Stone & Phan (1995) have examined the use of the cone penetrometer test, which is a strength test, even to determine the plastic limit of soil. It has been suggested (Wood & Wroth 1978) that the plastic limit be redefined as that moisture content corresponding to a one hundred fold increase in strength over the value at liquid limit, denoted by w_{P100}.

The procedure involves a series of tests with two cone weights W_1, and W_2. The computation of plastic limit water content is based on the dimensional analysis which states that:

$$\frac{c_u d^2}{W} = \text{constant} = c \tag{1.3}$$

where c_u is the undrained shear strength of the soil and d is the penetration of a cone of weight W. According to the critical state considerations (Schofield & Wroth 1968) the relation between water content w and undrained strength c_u, is of the form:

$$w + A \ln c_u = \text{constant} \tag{1.4}$$

The plasticity index can be represented as

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$$I_P = w_L - w_P = A \ln \left\{ \frac{(c_u)_{w_P}}{(c_u)_{w_L}} \right\} \tag{1.5}$$

From Equations (1.2) and (1.4)

$$w + A \ln \left\{ \frac{W c_u}{d^2} \right\} = k_1 \tag{1.6}$$

For geometrically similar cones of weight W_1, and W_2 having the same apex angle it follows from Equation (1.6) that

$$w_1 - 2A \ln d_1 = k_1 - A \ln \{W_1 c_u\} \tag{1.7}$$

Similarly, for the cone of weight W_2

$$w_2 - 2A \ln d_2 = k_1 - A \ln \{W_2 c_u\} \tag{1.8}$$

If W_2 is greater than W_1 the above relations reflect two parallel straight lines in w versus ln d space (Fig. 1.6).

At the same penetration according to Equations (1.7) and (1.8)

$$w_1 - w_2 = \Delta = A \ln \left\{ \frac{W_2}{W_1} \right\} \tag{1.9}$$

Δ is the vertical separation distance in terms of water content w on the linear plots of w versus ln d for two cones. Hence,

$$A = \left\{ \frac{\Delta}{\ln [W_2 / W_1]} \right\} \tag{1.10}$$

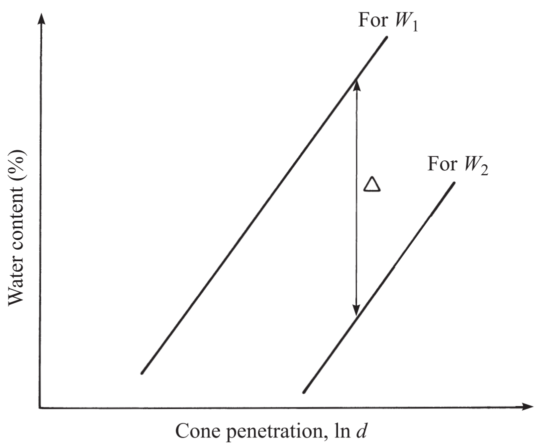

Figure 1.6. Cone penetration versus water content relations for different cone weights.

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From Equation (1.5) for $\left\{\frac{(c_u)_{w_P}}{(c_u)_{w_L}}\right\} = 100$

Equation (1.5) reduces to $I_P = A \ln 100$. From Equation (1.10) and the above relation,

$$I_P = \left\{\frac{\Delta \ln 100}{\ln(W_2 / W_1)}\right\} \tag{1.11}$$

Detailed investigation by Wasti & Bezirci (1986) lends support to this postulation. Further simplification of the above method has been due to Belviso et al. (1985). For a strength ratio of 100 from Equation (1.4) in log scale we have

$$w_L + A \log c_{uL} = w_p + A \log 100\, c_{uL}$$

from which it follows that

$$A = \left\{\frac{w_L - w_P}{2}\right\} = \frac{I_P}{2} \tag{1.12}$$

For a cone of weight W the undrained strength according to Equation (1.3) can be expressed as

$$c_u = \frac{\text{constant}}{d^2} \tag{1.13}$$

On substitution for A and c_u respective parameters in Equation (1.4) and from Equations (1.12) and (1.13)

$$w - I_P \log d = \text{constant} \tag{1.14}$$

This relation suggests that the straight line obtained by plotting the water content versus depth of penetration from the cone penetration test can itself be used to determine both the liquid limit and plasticity index. The water content at 20 mm penetration is the liquid limit. The slope of the line is the plasticity index (Fig. 1.7) of the soil, defined as the water content change for which the undrained strength increase is a hundred fold over that at the liquid limit water content.

Shrinkage limit: As moisture content decreases below the plastic limit, volume decreases and reaches a stage at which the transition from a particulate to a solid state takes place. Further removal of water is not associated with any corresponding volume reduction as the intracluster and intercluster forces tend to be dominantly attractive. This transitional water content is the shrinkage limit of the soil (see Fig. 1.8). The stress levels needed to reach such states would be too high to consider from the point of view of the load carrying capacity of soils. Hence, although the shrinkage limit is one of the Atterberg's limits, its consideration is

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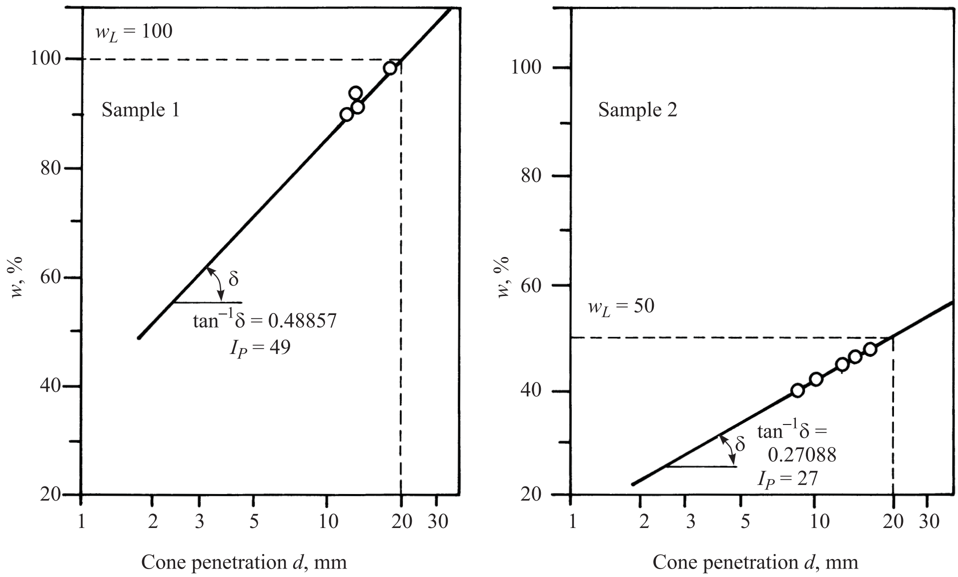

Figure 1.7. Cone penetration test data to determine w_L and w_P, through I_P (Belvisco et. al., 1985).

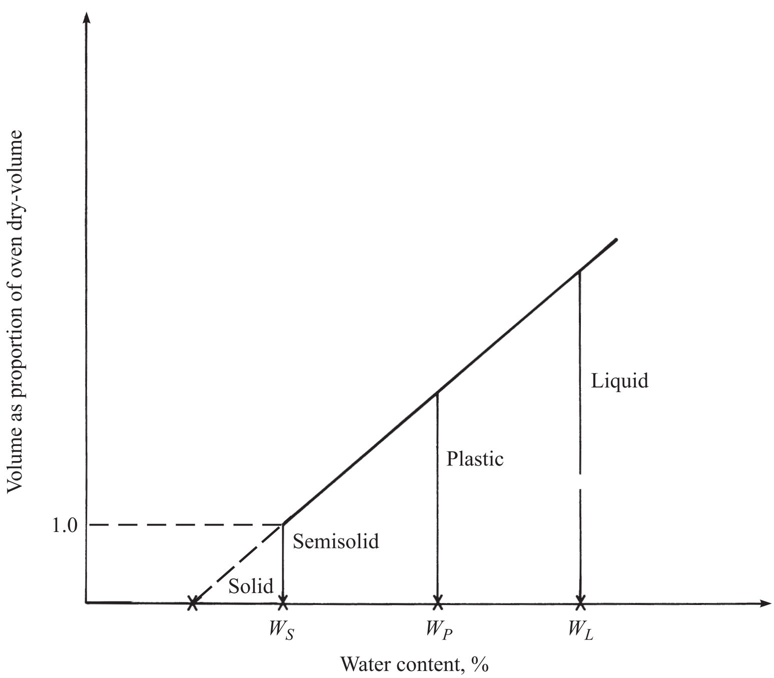

Figure 1.8. Consistency states due to drying from the liquid limit state of the soil.

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primarily in assessing the potential of soils for volume changes due to environmental factors. One such example is in the assessment of the volume changes of soils due to the wetting and drying cycles in arid climatic conditions.

1.3.2 *Soil state*

The water content of soils as a state parameter would be adequate, if the soils encountered are always in their saturated condition and when change in the state can be related to the change in water content. The saturated condition implies that all the void spaces are filled with water resulting in two phase material. Quite often void spaces are only partly saturated and changes in water content do not result in corresponding changes in the state. Hence density of soil would be needed in addition to water content, to define the state.

The density of particulate media cannot be that of the specific gravity of their solid constituents. Void spaces are inevitable due to the size and shape of the solid constituents and mode of packing. This would be the minimum and approaches the limiting densities, if individual particle shapes were to be true cubes and parallelopipeds similar to a child's building blocks. Densities can be far lower than from those of packing considerations if the physico-chemical interactions between the particles and fluid medium are dominant. Clay-water systems are a typical example. To consider the relative proportion of multiphase components in an unit volume, a phase diagram would be helpful (Fig. 1.9).

The void volume, V_v, is the volume not occupied by solids in unit volume. The void ratio, e, accounts both the volume of voids and the volume of solids expressed as their ratio

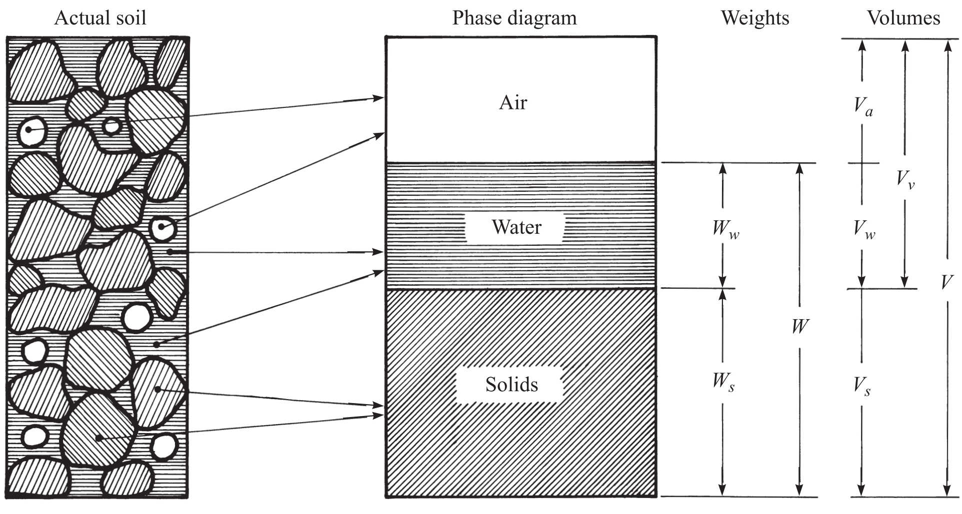

Figure 1.9. Phase diagram of fine grained soil.

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$$e = \frac{V_v}{V_S} \tag{1.15}$$

Another way of expressing the quantity of voids is to relate the void volume to total volume. This is porosity, *n*, defined as the ratio of voids to total volume

$$n = \left[\frac{V_v}{V}\right] \tag{1.16}$$

The interrelation between void ratio and porosity is expressed as

$$e = \left\{\frac{n}{1-n}\right\} \tag{1.17}$$

While the soils are partly saturated only a fraction of the void volume contains water. This fraction is the degree of saturation, S_r

$$S_r = \left\{\frac{V_w}{V_v}\right\}, \quad V_w = eS_r \tag{1.18}$$

The degree of saturation is expressed as a percentage. The limits of this value are zero for dry soil to 100% for completely saturated condition. To determine the degree of saturation, it is necessary to determine void ratio and water content independently.

Three important density computations are total unit weight, γ_b, either in partly or completely saturated state, dry unit weight, γ_d, and buoyant unit weight, γ_{sub}, which is the difference between, γ_{sat}, and unit weight of water, γ_w. Total unit weight is defined as the total weight divided by the total volume, and dry unit weight is the weight of solids (dry weight) divided by the total volume. These weight volume relationships involve specific gravity of solid particles and unit weight of water, being the basic parameters from the phase diagram. The corresponding expressions are

$$\gamma_b = \left\{\frac{G + S_r e}{1+e}\right\}\gamma_W \tag{1.19}$$

for partly saturated condition and for saturated and dry conditions since in Equation (1.19) with $S_r = 1$ and zero respectively, reduces to:

$$\gamma_b = \left\{\frac{G+e}{1+e}\right\}\gamma_w \tag{1.20}$$

$$\gamma_d = \left\{\frac{G}{1+e}\right\}\gamma_w \tag{1.21}$$

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The ratio of two densities as expressed in Equations (1.19) and (1.21)

$$\frac{\gamma_b}{\gamma_d} = 1 + \left\{ \frac{S_r e}{G} \right\} \tag{1.22}$$

Since $S_r\, e = w\, G$

$$\gamma_b = (1 + w)\, \gamma_d \tag{1.23}$$

It is necessary to determine bulk density and water content to compute the void ratio and degree of saturation which in turn define the physical state of the soil.

1.4 EFFECTIVE STRESS PRINCIPLE

Although soils are regarded as continua, in reality, as elucidated above, they are multicomponent multiphase particulate materials with the possibility of water in the pore spaces being under pressure when subjected to stresses. An element of soil will have a set of total stresses acting on the boundaries and a pore pressure acting within the element. The principle of effective stress (Terzaghi 1936) determines the effect of applied stress on the behaviour of a soil, with a given magnitude of total stress. This principle is the single most significant concept in soil mechanics and its importance needs hardly be stressed.

Terzaghi's (1936) principle of effective stress is stated in two parts. In the first part the fundamental effective stress equation is defined as

$$\sigma' = \sigma - u \tag{1.24}$$

As stated by Terzaghi, the principle of effective stress is deceptively simple. Of the terms in the above relation only the total stress can be directly measured. The induced pore water pressure is measured at a point away from the interparticle zone. Hence the effective stress σ' is a deduced quantity.

The second part of the principle enunciates the significance of the effective stress as:

> *'All measurable changes in volume, deformation and mobilization of shearing resistance are exclusively due to the changes in the effective stress.'*

This formulation has all along been tacitly assumed to be valid for all types of soils, provided they are essentially in a saturated condition.

1.5 COMPRESSIBILITY AND CONSOLIDATION

All soils are compressible since the void ratio of a fully saturated soil-water system is relatively large when compared to other construction materials. The stress levels encountered in most practical situations are such that neither the solid par-

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ticles nor the fluid in the pore spaces undergo any compression independently. As such the volume change due to compression is primarily due to the expulsion of water from pores resulting in the reduction of the void ratio. For sands such void ratio changes are minimal up to the stress levels at which grains begin to crush so as to induce marked changes in the packing. On the contrary, void ratio changes in clays can be even of the order of 50 to 60% of the initial void ratio for the effective stress range of engineering interest. The volume change due to a unit load is compressibility and its rate is consolidation. Such volume changes may be one dimensional or three dimensional and may occur immediately or time dependent.

When soil layers covering a large area are loaded vertically, the compression can be regarded as one dimensional. According to Skempton (1960) the first consolidation cell to obtain time-compression data was developed by Frontard in 1910. Floating ring and fixed ring are two types of consolidometers commonly used in the consolidation tests (Fig. 1.10). Rowe's hydraulic consolidometers for simpler loading (Rowe & Barden 1966), consolidometers for tests under a constant rate of strain (Wissa et al. 1971), split ring oedometers for lateral stress measurement (Senneset 1989) are more recently developed versatile testing facilities.

In the presentation of the test data, the change in void ratio in a decreasing scale versus the increase in the loading stress, is one of the common methods of presentation (Fig. 1.11). The curve is roughly logarithmic in form with the void

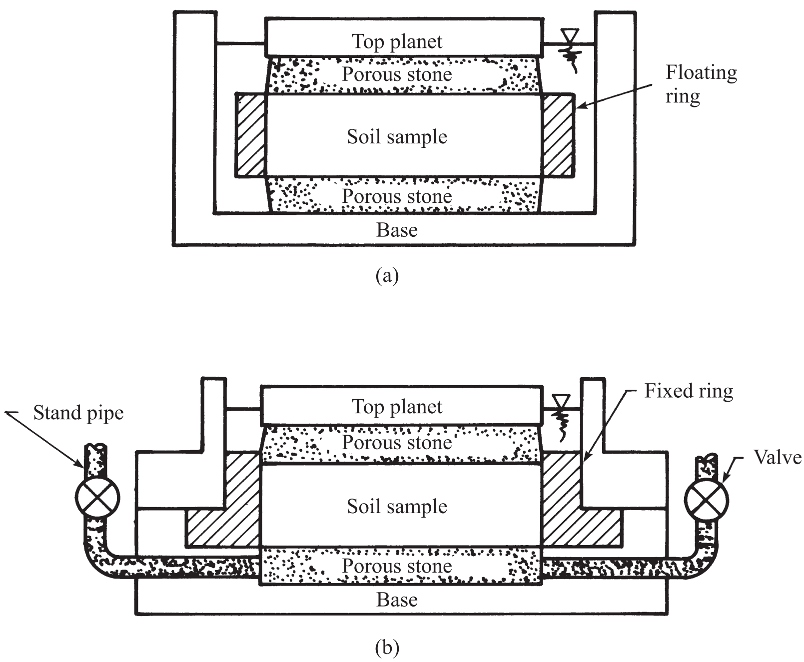

Figure 1.10. Floating and fixed ring consolidometers.

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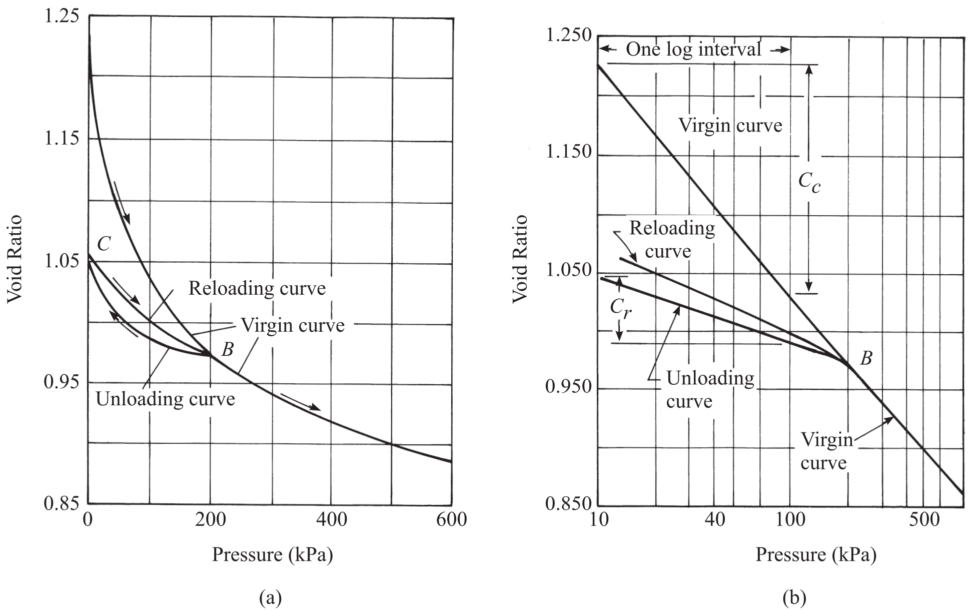

Figure 1.11. Typical void ratio-pressure relations in: a) Linear, and b) Logarithmic pressure representation.

ratio decreasing in proportion to the logarithm of the pressure. An empirical linear relation for the curve can be written as,

$$e = e_o - C_c \log\left\{\frac{\sigma'}{\sigma_o}\right\} \tag{1.25}$$

in which: e is the void ratio at effective pressure σ', e_o is known void ratio at effective pressure σ'_o, C_c is the compression index which defines the relationship between applied pressure and void ratio.

If e_1, the void ratio at unit pressure, is known, then the above relation reduces to

$$e = e_1 - C_c \log \sigma' \tag{1.26}$$

For example if the void ratio of soil at 1 kPa is 0.8 and the compression index is 0.20 then the void ratio at 10 kPa is:

$$e = 0.8 - 0.20 \log 10 = 0.60$$

Since the void ratio versus pressure relationship for a soil is nearly proportional to pressure in the semi-logarithmic plotting, the compression path for monotonic loading is linear, as shown in Figure 1.11b. The slope of the curve is negative on this type of plot and is indicative of the compression index, C_c. From Equation (1.25)

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$$C_c = \left\{ \frac{e_o - e}{\log(\sigma' / \sigma'_o)} \right\} \tag{1.27}$$

If $\sigma' = 10\ \sigma'_o$, $\log 10 = 1$ and $C_c = e_o - e$

The compression index can easily be read from a semi-logarithmic plot as the change in void ratio for a tenfold increase in pressure (one log cycle). If the void ratio at 10 kPa is 1.25 and at 100 kPa is 1.05 then $C_c = 1.25 - 1.05 = 0.20$.

1.5.1 *Stress history effects*

The in-situ state of soil is the resultant effect of the various factors such as stress history, time and environment. In characterizing the soil behaviour, as a first step, assessment of stress history effects in compression provides a means to assess how soil will respond under subsequent loading. In Table 1.2 the causes for sharp

Table 1.2. Mechanisms that cause a transitional stress level in the compression paths.

Causes	Remarks and references
Change in total stress due to:	
1. Removal of overburden	Deep excavations
2. Past structures	Renovation by reconstruction
3. Glaciation	
Change in pore water pressure due to:	
1. Change in water-table elevation	Kenney (1964) gives sea-level changes
2. Artesian pressures	Common in glaciated areas
3. Deep pumping	Common in many cities
4. Desiccation due to drying	May have occurred during deposition or subsequently
5. Desiccation due to plant life	May have occurred after deposition
Changes in soil structure due to:	
1. Secondary compression (ageing)*	Leonards & Ramiah (1960) Leonards & Altschaeffl (1964)
2. Environmental change, such as pH, temperature, salt concentration	Bjerrum (1967, 1972, 1973) Lambe (1958a, 1958b)
3. Chemical alterations due to: weathering, precipitation of cementing agents and ion exchange	Bjerrum (1967)
Change of strain rate on loading	Wissa, et al. (1971), Leroueil et al. (1985)

* The magnitude of ($\sigma'_{VC}/\sigma'_{VO}$) due to secondary compression for mature natural deposits of highly plastic clay may reach values as high as 1.9 or higher (Leonards & Altschaeffl 1964, Bjerrum 1967)

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transitions in the e – log σ_v paths are indicated. Determination of the stress level at which a transition in the compression path occurs can be identified by a simple one dimensional consolidation test irrespective of the reason for exhibiting such a response. The compression – rebound – recompression paths for clays are as shown in Figure 1.12. The soil does not rebound to its original level when the loads are removed. Only some of the compressions are reversible. Upon reloading the resulting slope of the compression path is far less steep than the virgin compression. To make an attempt to assess the entire compression path it is essential to understand the various possibilities and the mechanisms associated with the geological history to which soil deposits are subjected in their formation.

Of the several methods available for the estimation of σ_c, the most popular one is that proposed by Casagrande (1936a). This method suffers from certain drawbacks such as personal judgment and errors caused by the selection of inappropriate scales. Recently evolved methods for estimation of σ_c use the data in the form of log e – log p plots (Jose et al. 1989) and log $(1 + e)$ – log p plots (Sridharan & Prakash 1997). Further, it has been observed that the representation of consolidation data in the form of porosity n versus log pressure plots is found to yield a bilinear relationship for soils of known stress history (Allam & Robinson 1997). The intersection of the two straight lines in the plot lies close to the level of maximum stress to which the soils were subjected.

1.5.2 *Consolidation*

How rapidly soils undergo compression depends upon the rate of flow of fluids from their voids and their proximity to the drainage boundary. This process is

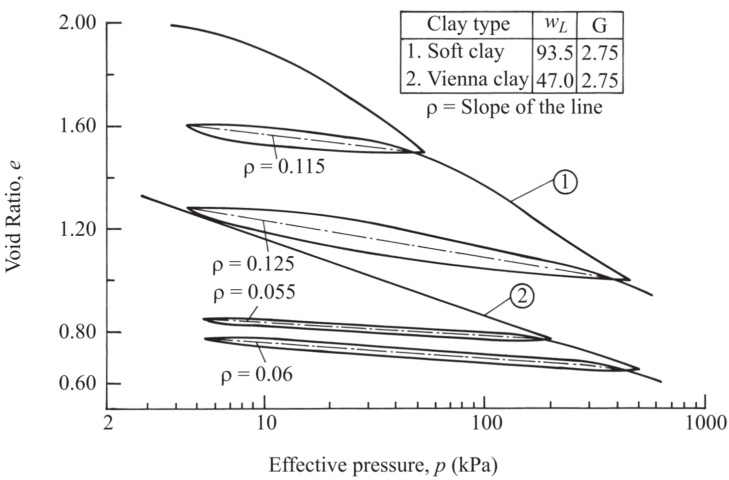

Figure 1.12. Void ratio – effective pressure paths of two clays in normally consolidated and overconsolidated states.

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termed consolidation which is a time-compression phenomenon. The fundamental difference between the compression of granular media and clays is one of the time element. Granular soils compress almost instantly due to high permeability whereas, in clays, consolidation is a time dependent process. A rational quantitative approach for analysis of stress – time – compression data has been possible since the introduction of Terzaghi's consolidation theory (Terzaghi 1923). In a broad sense the time compression diagram is characterized by three zones, viz., immediate, primary consolidation and secondary compression (Fig. 1.13). The consolidation theory serves as a basis for the determination of the degree of primary compression over a given time interval under a specific pressure increment.

The primary assumptions of the theory of consolidation are:

1. The soil is a homogeneous fully saturated clay water system.
2. The compressibility of soil grain and water is negligible in the stress range of engineering interest.
3. The compression is one dimensional and the direction of water flow is governed by Darcy's law.
4. The e-log σ' relationship is linear.

From the equality of rate of change in volume due to water outflow dQ/dt and due to volume change due to changes in pore volume, dV/dt, the governing equation of consolidation theory is

$$c_v \frac{\partial^2 u}{\partial z^2} = \frac{\partial u}{\partial t} \tag{1.28}$$

where: u is the hydrostatic excess pressure γ_w, h where, h is the hydraulic head with respect to position co-ordinate z, t is time, c_v is the coefficient of consolidation given by the above relation.

For an elemental soil cube at a depth z below the surface the amount of pore water outflow during the time interval dt is equal to the change in volume due to

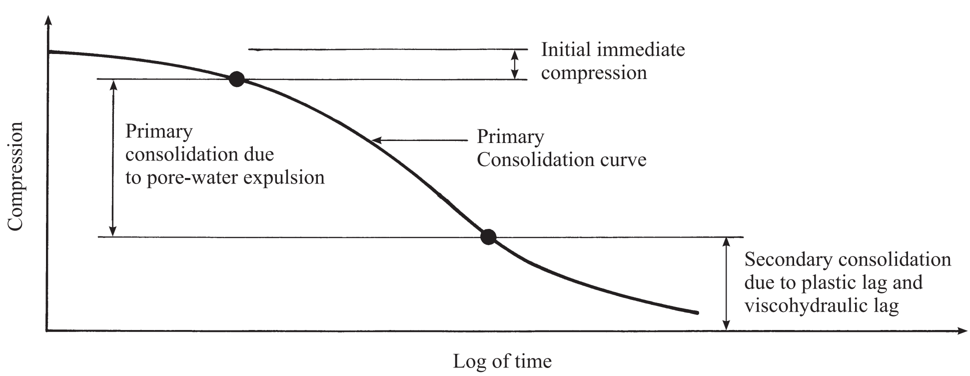

Figure 1.13. Typical time compression diagram for clays.

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the change in the pore volume of the cube in the same time interval, and can be expressed as

$$k_z \frac{\partial^2 h}{\partial z^2} \mathrm{d}x\,\mathrm{d}y\,\mathrm{d}z = \frac{\partial e}{\partial t} V_s$$

$$V_s = \frac{V}{1+e_o} = \frac{\mathrm{d}x\,\mathrm{d}y\,\mathrm{d}z}{1+e_o}$$

$$k_z \frac{\partial^2 h}{\partial z^2} = \frac{1}{1+e_o} \frac{\partial e}{\partial t}$$

since the flow is caused by hydrostatic excess pressure $u = h\,\gamma_w$

$$\frac{k_z}{\gamma_w} \frac{\partial^2 u}{\partial z^2} = \frac{1}{1+e_o} \left\{ \frac{\partial e}{\partial t} \right\}$$

The rate of change in void ratio is a function of coefficient of compressibility, a_v expressed as

$$\partial e = -a_v\,\mathrm{d}\sigma = a_v\,\mathrm{d}u$$

$$\frac{k_z(1+e_o)}{a_v \gamma_w} \frac{\partial^2 u}{\partial z^2} = \frac{\partial u}{\partial t} \qquad (1.29)$$

and in terms of the coefficient of consolidation Equation (1.29) is

$$c_v \frac{\partial^2 u}{\partial z^2} = \frac{\partial u}{\partial t}$$

where

$$c_v = \frac{k_z(1+e_o)}{a_v \gamma_w}$$

Using the method of separation of variables, the general integral of the consolidation Equation (1.29) is arrived at. In order to find the integral a product function

$u(z, t) = f_1(z)\, f_2(t)$ is assumed to be the solution.

The solution of the partial differential Equation (1.28) involves finding the $u = f(z, t)$ that satisfies both the differential equation and the boundary conditions associated with the problem.

$$u(z, t) = (a \cos z + b \sin \lambda z) \exp(-\lambda\, c_v\, t)$$

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is the general integral solution. For the determination of constants *a* and *b*, and *k* the characteristic value, boundary conditions are used. In terms of nondimensional parameters the solution is expressed in the form

$$\Delta u = \Delta\sigma \sum_{n=0}^{\infty} f_1(z) f_2(T)$$

where $f_1(z)$ is non-dimensional geometrical parameter, z/H, $f_2(T)$ is nondimensional time factor, T is related to coefficient of consolidation, c_v by

$$T = \frac{c_v t}{H^2} \tag{1.30}$$

The progress of consolidation after time t at any depth z in the consolidating layer can be related to the void ratio, c_v at that time to the initial and final void ratios. The consolidation ratio is expressed as

$$U_z = \frac{e_1 - e}{e_1 - e_2}$$

This ratio in terms of pore water pressure

$$U_z = \frac{(u_i - u)}{u_i} = 1 - \frac{u}{u_i} \tag{1.31}$$

From the above relationships it is evident that U_z, often expressed as percent consolidation, is zero at the start of loading and gradually increases to 100% as the void ratio changes from e_1 to e_2. Figure 1.14 shows the relationship between average degree of consolidation U and time factor T. Taking advantage of the similarity of theoretical U-T curves and the experimental, c_v is determined by different curve fitting methods. Casagrande's logarithm of time fitting method to find t_{50} and Taylor's square root of time method to find t_{90} are the most commonly used methods. More recently, observation of the plot of consolidation U which can be approximated by a rectangular hyperbola in the range of 60-90%, led to the development of the rectangular hyperbola method (Sridharan et al. 1987) to determine the coefficient of consolidation.

By further examination of the theoretical U-T curve (Pandian et al. 1992), it has been shown that the log (U/T) versus log T plot exhibits a bilinear behaviour. The intersection point corresponds to a theoretical time factor of 0.793 at a degree of consolidation of 88.5%. Using this characteristic feature, the proposed equation for computation of the coefficient of consolidation is

$$c_v = \frac{0.793 d^2}{t_{88.5}} \tag{1.32}$$

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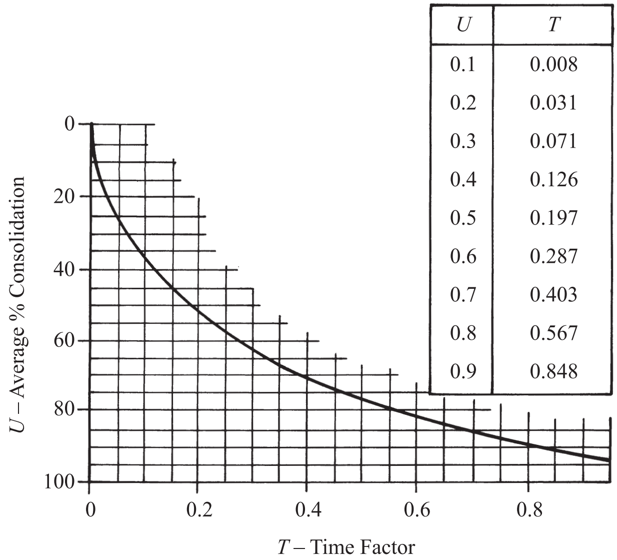

U	*T*
0.1	0.008
0.2	0.031
0.3	0.071
0.4	0.126
0.5	0.197
0.6	0.287
0.7	0.403
0.8	0.567
0.9	0.848

Figure 1.14. $U - T$ relation for uniform stress distribution in clay.

where time $t_{88.5}$ corresponds to 88.5% consolidation, which can be obtained from the log δ/t versus log t plot and d is the drainage path.

At present, computer-controlled data logging systems favour the use of non-graphical methods for analysis of time compression data. Towards this endeavour, the most recent analyses made by Robinson (1997) and Robinson & Allam (1998) are very significant. It has been observed that the degree of consolidation U while plotted against the logarithm of the theoretical time factor T_v shows an initial parabolic region up to about $U = 60\%$ followed by a steep portion, and after $U = 95\%$ it flattens rapidly, approaching a horizontal asymptote corresponding to $U = 100\%$. When the slope of $U - \log T_v$ curve, $M = dU/d \log T_v$ is plotted against $\log T_v$ the inflection point of maximum slope, M_i has been found to be at $U = 70.15\%$ and $T_{vi} = 0.405$ with $M_i = 0.6868$. This characteristic feature formed the basis for consolidation analysis by the inflection point method to determine the coefficient of consolidation, coefficient of secondary consolidation and total primary compression directly as the time compression data with time for each of the load increment is logged. In the method suggested by Robinson & Allam (1998) the values of initial compression, δ_i, total primary consolidation, δ_{100}, and coefficient of consolidation, C_v, can be obtained from the laboratory δ-t data by direct matching with consolidation theory.

Another recent innovation by Sridharan et al. (1999) enables us to markedly reduce the time to obtain the $e - \log \sigma'$ path in the conventional laboratory compression test to 4 to 5 hours, which otherwise requires several days. This rapid method of consolidation test procedure is basically the same as the conventional method, except that the time allowed for each load increment is much less. During

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the consolidation process the degree of consolidation is obtained by the rectangular hyperbola method (Sridharan et al. 1987). This would allow one to compute the effective stress at any time in the consolidation process. This defines the void ratio and effective stress position along the compression path. Next the load is incremented keeping the load increment ratio near to unity. By adopting this procedure, all conventional compressibility characteristics, C_c, C_v and m_v can be obtained for a soil within a single working day.

1.6 SHEAR STRENGTH

Soils, as particulate materials, derive their ability to support themselves and/or external loading by their shear strength. As such the analysis and design of stability problems entails the assessment of shearing resistance of soils. In soil masses, deformations result largely from relative displacements and slippage between particles in coarse grained soils or by distortions to the clay microstructure. Accordingly, the term shearing resistance is synonymous with strength. If a well defined slip surface develops the failure plane can be identified directly. However, if slippage occurs simultaneously along many surfaces within the soil body, the failure plane is undefined. In such cases failure is expressed in terms of maximum principal stresses that can be mobilized. From these stresses the compressive strength is defined as the difference between major and minor principal stresses at failure, $(\sigma_1 - \sigma_3)_f$. The compressive strength per unit of effective minor principal stress at failure, the effective stress ratio, $(\sigma'_1/\sigma'_3 - 1)$ is also a measure of shearing resistance. When actual rupture takes place failure is defined by residual deviatoric stress, which is unique. In the absence of such a situation failure is defined by a certain limiting deformation.

1.6.1 *Strength tests*

Many type of shear tests have been devised, at various stages of the development of soil mechanics, for measuring the shearing resistance of soils. Ever since Coulomb's classic work in 1776, direct shear tests have been in use. The test is designated so, because failure is induced on a specific plane on which the normal and shear stresses at failure are known. The apparatus for performing direct shear is made up of a rectangular box consisting of lower and upper halves. The sample to be tested is compressed under specific normal load and the upper half of the box is moved laterally by a shear force, either by the controlled rate of strain or stress. The mobilization of shearing resistance, along the horizontal plane between the two halves of the apparatus, increases until the sample fails. Primary modifications to conventional apparatus to avert possible rotation of the loading platens are the provision of roller bearing arrangements (Wernick 1977) (see Fig. 1.15).

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Another independent attempt to apply uniform shear strains is the development of simple shear devices. This is similar to the direct shear test, except that in the simple shear device one pair of platens is allowed to rotate so as to induce uniform deformation (see Figs 1.16a and b). The NGI type (Fig. 1.16a) is used in a number of commercial laboratories (Bjerrum & Landva 1966). The membrane is reinforced with a spiral winding of the wire which prevents horizontal direct strain but allows rotation of sides of the sample as illustrated. In another type developed at Cambridge, around the same time (Roscoe 1970), the sample is contained within a set of interlocking rough rigid platens as illustrated in Fig-

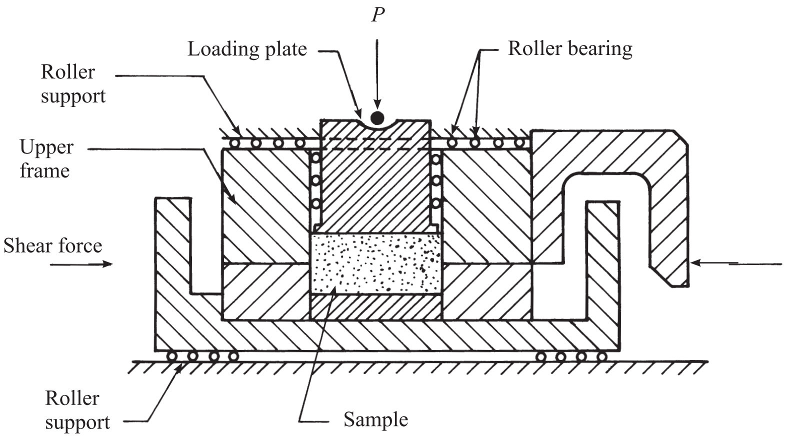

Figure 1.15. Modified direct shear box (Wernick 1977).

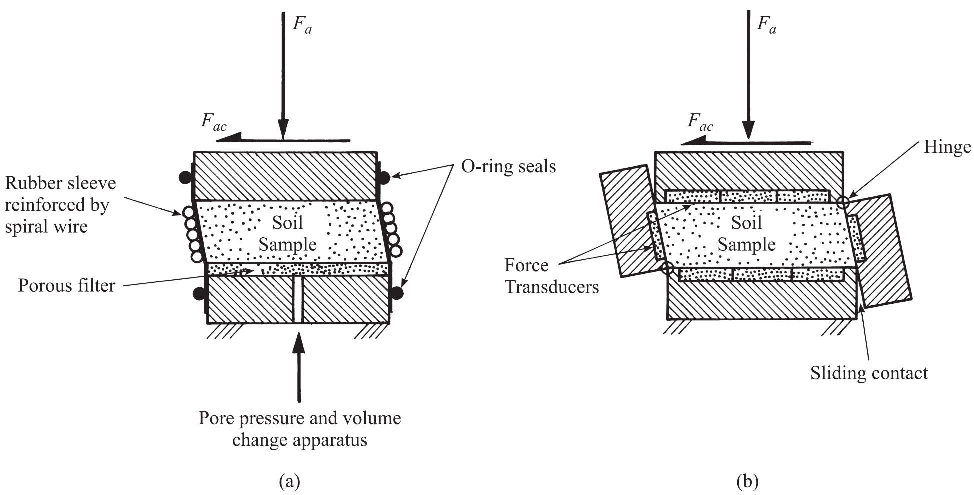

Figure 1.16. Simple shear devices. a) NGI type (Bjerrum & Landva (1966), and b) Cambridge type (Roscoe 1970).

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ure 1.16b. During a simple shear test, the principal planes of stress and strain rotate, as the state of stress in the conditions imposed by the apparatus do not require that the principal planes of stress and of strain coincide.

Cylindrical compression tests either unconfined or confined by fluid pressures are the most widely used types of tests for measuring the shearing resistance of soils. In the unconfined compression test the axially applied compressive stress is the principal stress, σ_1, while the other two stresses are zero.

The test is an undrained test on saturated cohesive soils whose angle of shear resistance is zero. Figure 1.17 shows that if $\phi = 0$ half the difference of the principal stresses represent the shear stress at failure.

$$\sin\phi_f = \left\{\frac{\frac{\sigma_1}{2}}{\frac{\sigma_1}{2} + c\cot\phi_f}\right\} = \left\{\frac{\sigma_1}{\sigma_1 + 2c\cot\phi_f}\right\}$$

$$c = \left\{\frac{\sigma_1\left(1 - \sin\phi_f\right)}{2\cos\phi_f}\right\} = \frac{\sigma_1}{2}$$

In the triaxial compression test, a cylindrical specimen is subjected to a uniform axisymmetric confining pressure and an axial displacement applied at the ends of the specimen. The principle of the test is illustrated in Figure 1.18.

The main advantages of the triaxial tests are of practical significance. Among these are:
- simulation of in-situ stress conditions
- drainage conditions can be controlled
- no rotation of the plane of principal stresses during shearing, and
- contrary to the direct shear test, failure is not forced to occur across a predetermined plane.

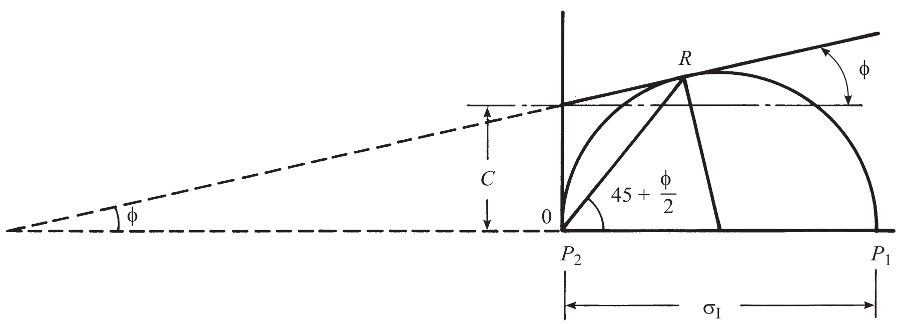

Figure 1.17. Mohr's circle for the unconfined compression test.

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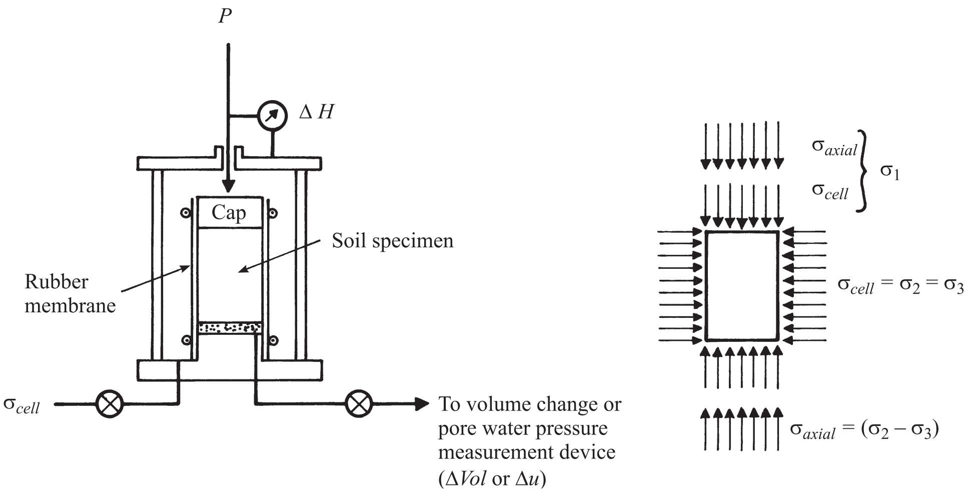

Figure 1.18. Details of conventional triaxial compression test.

The test methods elucidated by Bishop & Henkel (1962) as early as 1962 are still followed in routine investigations. More recently a re-evaluation of the conventional triaxial methods has been presented by Baldi et al. (1988). With the introduction of computer-controlled triaxial system with digital controllers (Menzies 1988), the triaxial test has far more built in flexibility to simulate various total and effective stress paths. The three basic types of tests for evaluating the shearing resistance of fine grained soils are (1) undrained (2) consolidated undrained and (3) drained tests. In the case of the undrained condition, either positive or negative pore pressures will result from application of both normal and shear stresses. Consolidated undrained shear tests are those in which primary consolidation is allowed by permitting drainage, while no drainage is allowed during the subsequent application of either normal and shear stresses. In drained tests the pore pressure is not allowed to develop at any time during the test. This is accomplished by adopting an appropriate rate of strain in the triaxial compression test.

From the analysis of triaxial test data of stress, strain, pore pressure or volume change it would be possible to obtain stress-strain-pore pressure/volume change relationship of soils. Further plots of the deviator stress and effective normal stress can be made in the form of modified Mohr Coulomb diagrams to obtain shear strength parameters (c, ϕ). Typical plots of such diagrams are provided in Figure 1.19.

1.6.2 *Strength theories*

Soil elements will be subjected to a wide range of stress paths arising out of different field situations. Hence it is necessary to consider what governs failure un-

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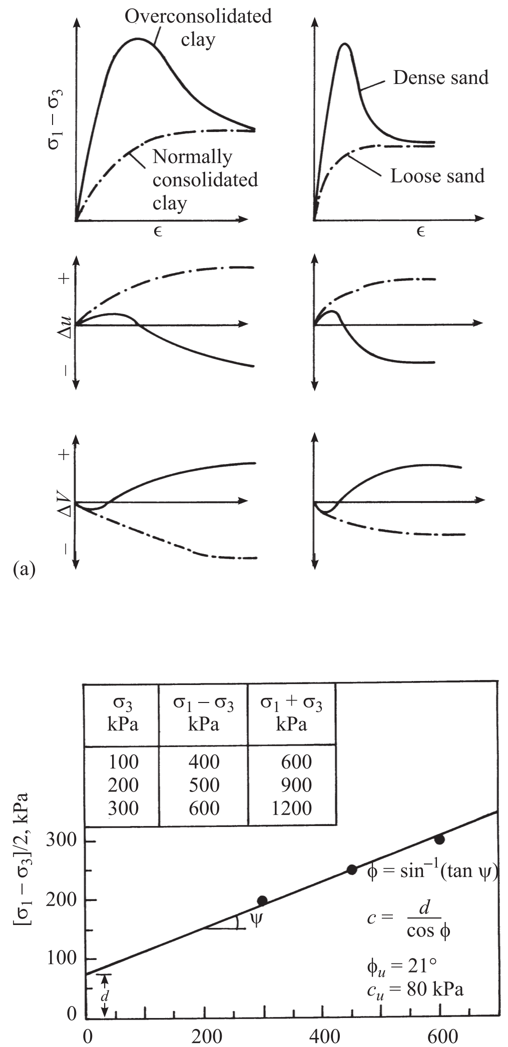

Figure 1.19. a) Schematic representation of stress-strain-pore pressure/volume change response of soils, and b) Modified Mohr Coulomb representation of triaxial compression test results.

der the general state of stresses, apart from the above parameters obtained from conventional triaxial tests. In general, strength theories are used to predict the overall stress state at various material states. The terms yield and failure are often used to describe the response of soils under various stress levels. These terms cannot be used indiscriminately. The term yield in the field of plasticity is used to describe the onset of plastic deformation, whereas failure implies the fracture of material by distinct surfaces of separation. Failure signifies the collapse of a material as the terminal stage of the process of yielding.

The strength theories, to provide the greatest generality, are developed in three dimensions. A state of stress which is just necessary to result in plastic yielding or

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in fracture of an element may be described by three principal stresses, σ_1, σ_2, and σ_3. In totality, different states of stress for the above conditions form a surface generated by the function:

$$f_1(\sigma_1, \sigma_2, \sigma_3) = 0 \tag{1.34}$$

which may be called the limiting yield surface. Stress combinations capable of rupturing the body may be represented by a second surface

$$f_2(\sigma_1, \sigma_2, \sigma_3) = 0 \tag{1.35}$$

which is the limiting surface of rupture.

Soils being particulate media, effective normal stress pronouncedly controls the shearing resistance that soils can mobilize. Hence Mohr (1900) symbolically expressed the criterion as

$$\sigma_{\max} - \sigma_{\min} = f(\sigma_{\max} + \sigma_{\min}) \tag{1.36}$$

This theory specifies that shearing stress at yield or failure is a function of normal stress σ_n acting on the plane considered.

$$\tau_{\max} = f(\sigma_n)$$

This failure theory proposed by Mohr (1900) is an extension of earlier work by Coulomb (1776) on retaining walls. This theory, which considers failure by both yielding and fracture, is expressed in a functional form between shear stress on the failure plane, τ and normal stress, σ as

$$\tau = \sigma \tan \phi + c \tag{1.37}$$

where: τ is angle of shearing resistance and c is cohesion (strength component independent of normal stress).

Considering the stresses as represented in the Mohr diagram (Fig. 1.20) the failure criterion Equation 1.36, can be expressed as

$$\sigma_{\max} - \sigma_{\min} = 2c \cos \phi + (\sigma_{\max} + \sigma_{\min}) \sin \phi$$

As a special case when $\phi = 0$, Mohr Coulomb and Tresca criteria coincide, according to which failure occurs when the maximum shear stress reaches a critical constant value.

Expressed in its most general form the failure surface corresponding to the Mohr Coulomb condition of failure is

$$\begin{aligned} &\left\{(\sigma_1 - \sigma_2)^2 - \left[2c \cos \phi + (\sigma_1 + \sigma_2) \sin \phi\right]^2\right\} \\ &\times\left\{(\sigma_2 - \sigma_3)^2 - \left[2c \cos \phi + (\sigma_2 + \sigma_3) \sin \phi\right]^2\right\} \\ &\times\left\{(\sigma_3 - \sigma_1)^2 - \left[2c \cos \phi + (\sigma_3 + \sigma_1) \sin \phi\right]^2\right\} = 0 \end{aligned} \tag{1.38}$$

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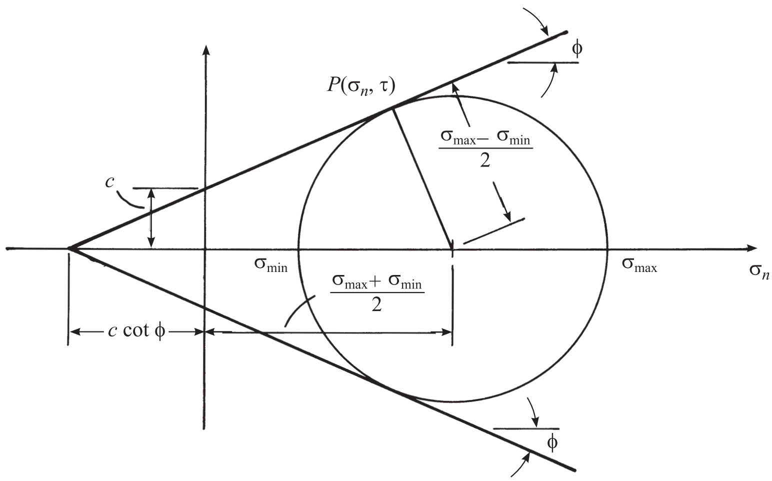

Figure 1.20. Representation of failure condition in the Mohr diagram.

The failure surface defined by Equation (1.38) is a pyramid with the space diagonal $\sigma_1 = \sigma_2 = \sigma_3$ as axis with an irregular hexagon with non-parallel sides of equal length as the cross section.

Figure 1.21 shows the intersection of the pyramid with the plane along which $\sqrt{2}\sigma_1 = \sqrt{2}\sigma_2$. For the sake of clarity a section on the octahedral plane $\sigma_1 + \sigma_2 + \sigma_3 =$ constant is also shown.

Granular soils: If dry granular material is poured from a single point above the ground it would assume a conical shape. Even with additional material, it eventually slides down to the same slope. The angle of this slope to the horizontal is the angle of repose. The heap of dry sand is stable without support. The angle of repose corresponds to the angle of friction of granular material in its loosest state. Sand dunes are an example from nature for the angle of repose.

Apart from the above simple picture of shear strength of particulate media, a soil system subjected to external loading is considered. Both the initial mean principal stress and shear stress values change during shearing. At this stage, an important distinction can be made between the behaviour of soils, particulate media and metal, a highly bonded system. When metal is subjected to shear stresses it deforms in a manner similar to soils, with the volume of the material or density remaining constant during the shearing process. On the contrary, in the shearing of soils, volume change manifests even if the mean effective stress remains constant. Consequently, the shear strength of the soil is pressure dependent, i.e., increases with the increase in confining pressure, unlike in metals.

For granular soils being only frictional in nature shear strength is given by the relation:

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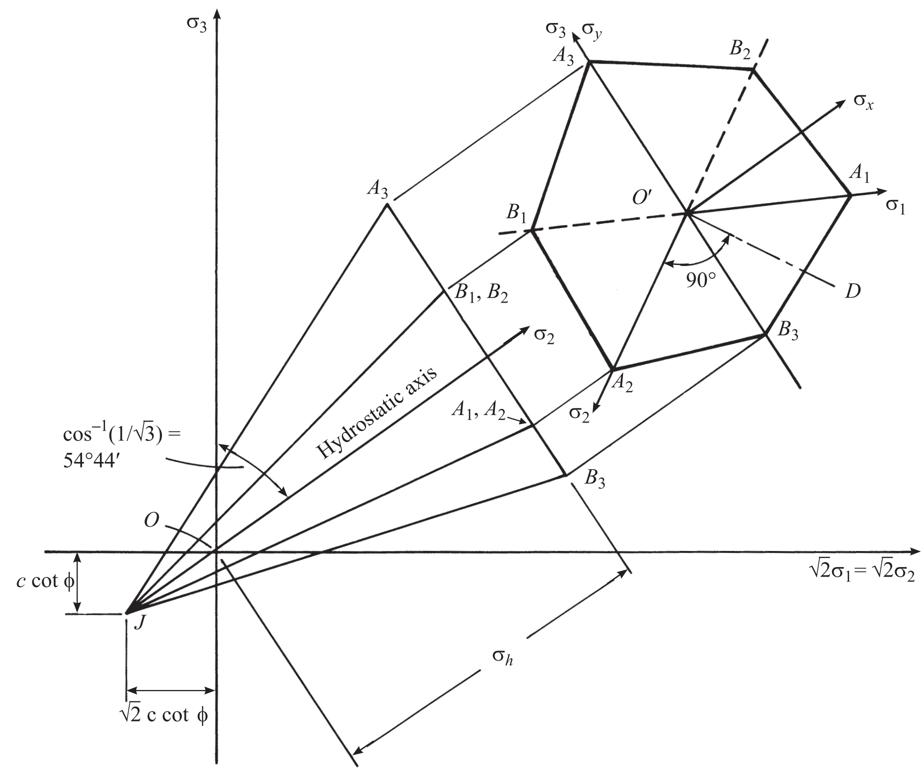

Figure 1.21. Mohr-Coulomb failure pyramid surface along with section on octahedral plane.

$$\tau = \sigma' \tan \phi'$$

where σ' is the effective stress.

Although the angle of shearing resistance is not strongly influenced by the presence or absence of water, the ϕ' value with respect to effective stresses is influenced slightly by the presence of water compared to the variation in ϕ' between granular soils of different composition. The following factors, in order of importance, have a significant effect (Leonards 1962):

– State of compaction,
– Coarseness of grains,
– Particle shape and roughness of grains, and
– Gradation.

Saturated remoulded clays: The general concepts concerning the strength characteristics of saturated clays have evolved as a result of tests on remoulded clays. This provides a basis from which to examine the responses of

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natural deposits which are most often influenced by stress history, time and cementation. Scanning the available literature reveals that the most comprehensive set of data on shear strength of remoulded clays was obtained by Hvorslev, Henkel & Parry and their co-workers (Hvorslev 1936, 1937, Henkel 1956, 1958, 1960, Parry 1956). There is a unique relationship between water content of a soil and the effective stress to which it is subjected, both for normally consolidated and overconsolidated conditions while considered separately. Considering cohesion to be the result of interparticle forces (thus the interparticle spacing and hence void ratio) Hvorslev (1937) expressed the shear strength of a soil in the form

$$\tau = c_e + \sigma' \tan \phi_e \tag{1.39}$$

where c_e and ϕ_e are true cohesion and true angle of friction respectively. Figure 1.22 shows the effective stress Mohr's circles at failure corresponding to-

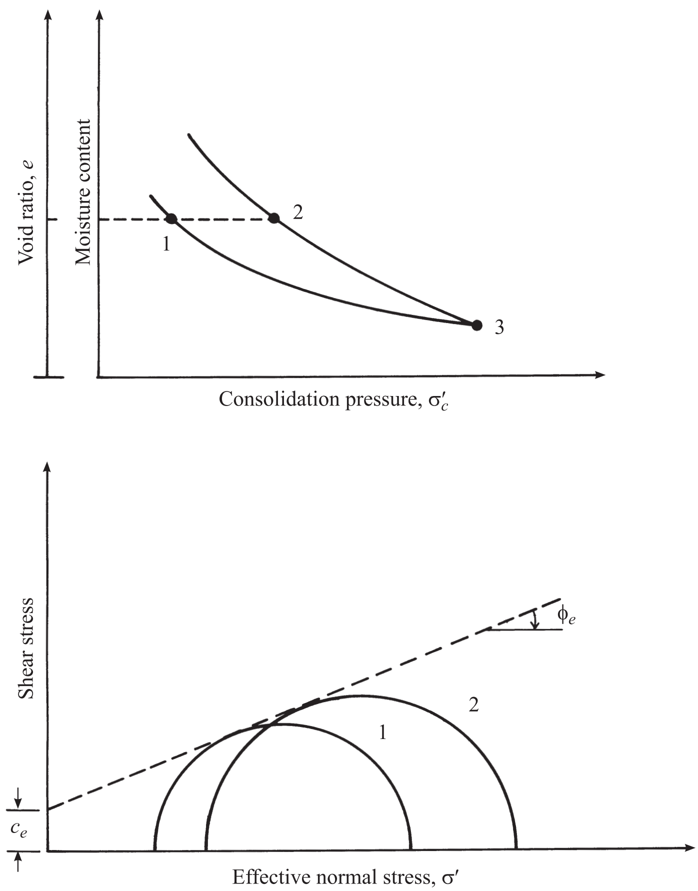

Figure 1.22. Schematic representation of Hvorslev strength parameters.

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points 1 and 2 corresponding to overconsolidated and normally consolidated states of the clay. The soil specimens at points 1 and 2 have the same water content, hence, the same void ratio. The slope of the common tangent is ϕ_e and the intercept on shear stress axis c_e. According to Hvorslev's failure criterion, the undrained strength of the clay can be expressed as:

$$s_u = \frac{\sigma_1 - \sigma_3}{2} = c_e \cos\phi_e + \frac{\sigma_1' + \sigma_3'}{2} \sin\phi_e \tag{1.40}$$

The strength of soil at failure is dependent on the moisture content at failure. As such Henkel (1960) pointed out that there is a unique relationship between moisture content at failure and strength of the clay soil (see Fig. 1.23). For normally consolidated clays the variation of water content, w, vs log $(\sigma_1 - \sigma_3)$ is approximated by the linear relation. For overconsolidated clays, this relationship is similar but lies slightly below the relationship of normally consolidated specimens. These curves merge at the preconsolidation pressure. Henkel (1960) has also shown that a simple general failure envelope for normally consolidated and overconsolidated soils can be obtained. This is done by plotting the ratio of the major to minor effective stress at failure, $(\sigma'_{1f}/\sigma'_{3f})$ versus the maximum consolidation pressure to the average effective stress at failure, J_m/J_f (Fig. 1.24). The drained and undrained test results of compression and extension tests of Weald clay lie on the same curve. The case of London clay is similar. Although the unique relationships can be observed by analysis of results of tests on the remoulded clays, there is a need to examine their findings in detail from the fundamentals of soil behaviour. Despite this fact, these relationships provide a satisfactory framework by which the effective stresses, the shear stresses and the volume changes or water contents can be correlated.

1.7 CONSTITUTIVE MODELLING OF SOIL BEHAVIOUR

Generally, partly for simplicity in practice and partly because of the historical development of soil mechanics, there have been two distinct classes of computations in the analysis of soil engineering problems, viz., stability analysis and settlement computations. These two aspects have for quite some time been treated separately, in rather an unrelated manner. In a broad sense, in stability analysis in saturated clays, apart from unit weight of soil, the only soil parameter used is the undrained shear strength. In complete contrast, conventional settlement computations are only concerned with deformations and strains, with no account of failure or limiting stresses. Soil parameters needed for such analysis are only a pseudo-elastic modulus from undrained triaxial tests or a compressibility modulus from consolidation tests. It has now been realized that what is desirable for more realistic analysis and design, is a complete knowledge of stresses and strains at all compatible loading levels right up to failure.

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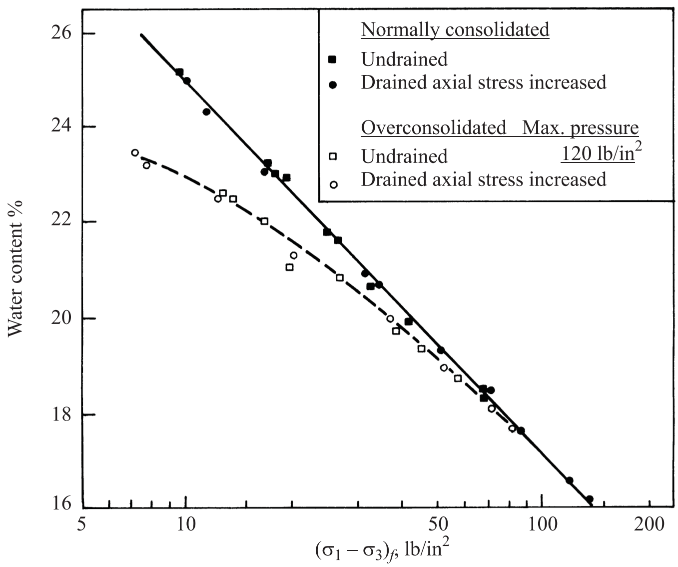
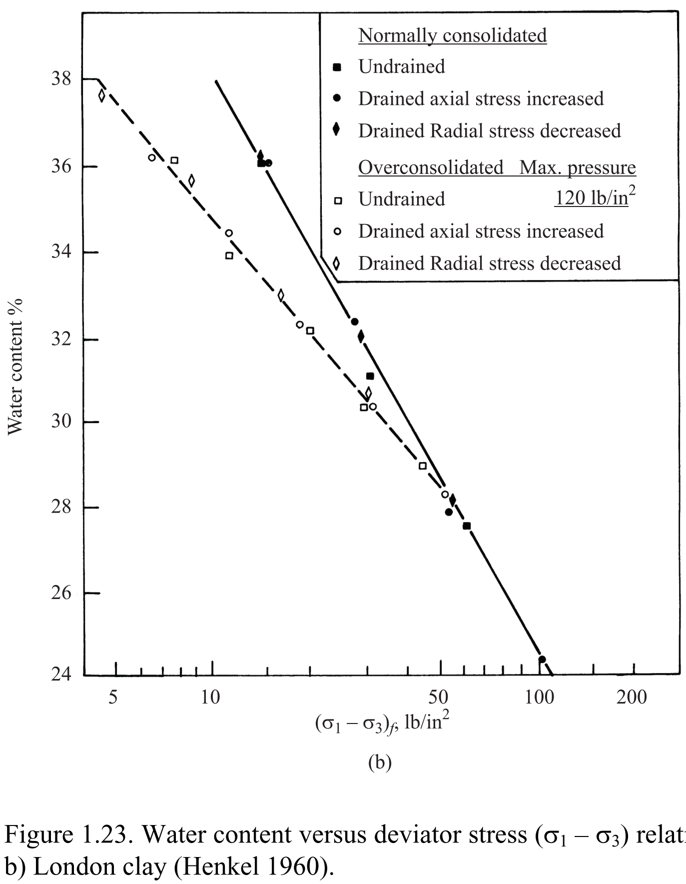


(a)



(b)

Figure 1.23. Water content versus deviator stress $(\sigma_1 - \sigma_3)$ relation for: a) Weald clay, and b) London clay (Henkel 1960).

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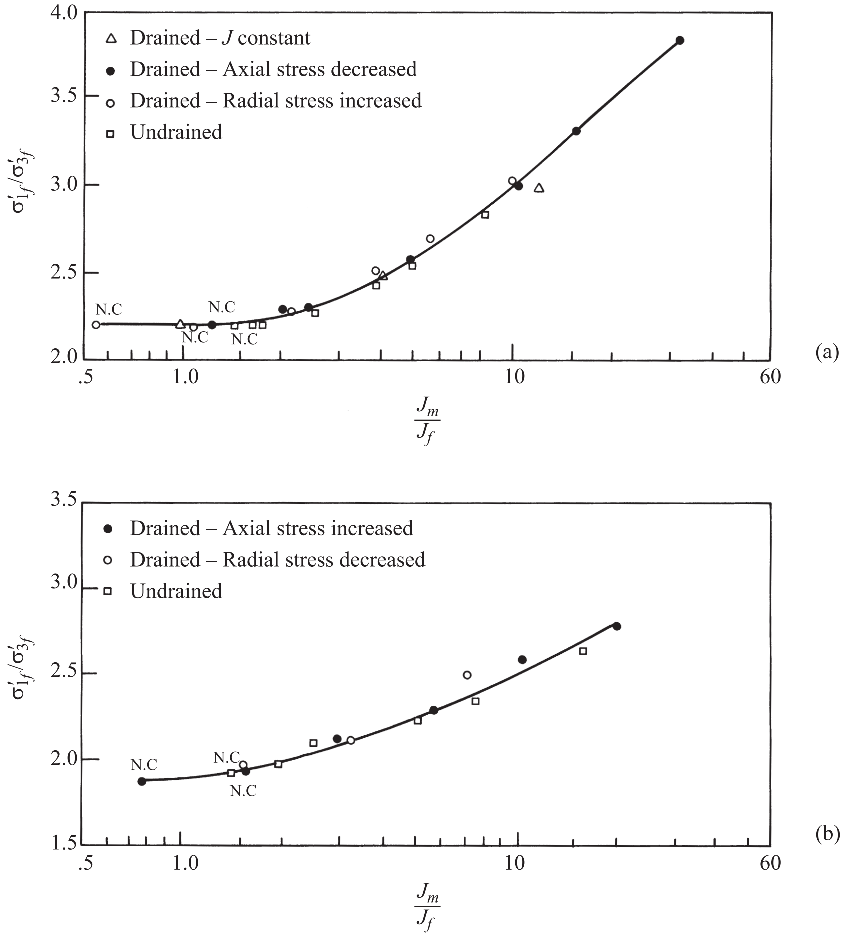

Figure 1.24. ($\sigma'_{1f}/\sigma'_{3f}$) versus ($J_m/J_f$) relation for: a) Weald clay, and b) London clay (Henkel 1960).

Development of constitutive relations is a significant step in this direction of integrating both the above approaches. These constitutive relations or stress-strain laws embrace information on both shear stresses and deformations at all stages of loading right from prefailure states to failure. The increased complexity of the loading conditions to which soils are subjected to, as well as the advent of large capacity computers coupled with efficient numerical methods, have led to intense efforts in the constitutive modelling of soils.

In a broad sense a constitutive law is a functional correspondence between the causes and effects of physical processes manifested by the material. It defines the relation between the external stimuli and the consequent material response. As cited above, constitutive relations of soils involve relations between the stresses

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imposed and the resulting strains. Admittedly, the behaviour of soils is more complex than is amenable to simple description. Soil behaviour is highly non-linear with irreversible strains for compression and shear loadings (Fig. 1.25). Hence the emphasis has been to identify an approach to describe adequately all the relevant aspects of the mechanical behaviour of the soil without losing sight of simplicity and exactness. This has resulted in producing new mathematical descriptions of soil behaviour within the framework of elasto-plasticity.

Drucker et al. (1957) were the first to suggest that soil can be modelled as an elasto-plastic material with work hardening or strain softening effects. With the concept of the critical void ratio of Casagrande (1936), an isotropic strain hardening model was proposed by Roscoe et al. (1958). This was designated as the Cam clay model for clays and the Granta Gravel model for sands by Schofield & Wroth (1968). These materials are not real soils in the sense that one cannot find deposits of them at some location in the ground. However the analytical formulations describe many real soils, if appropriate material parameters are obtained by simple laboratory tests.

1.7.1 *Cam clay model*

The basic Cam clay model has been developed for axisymmetric coordinates i.e., $\sigma_2' = \sigma_3'$, which corresponds to the stress states in a sample during a conventional triaxial test.

Three parameters p', q' and v, describe the state of the soil during this test. p' and q' are the spherical and deviatoric components given by the invariants of the form

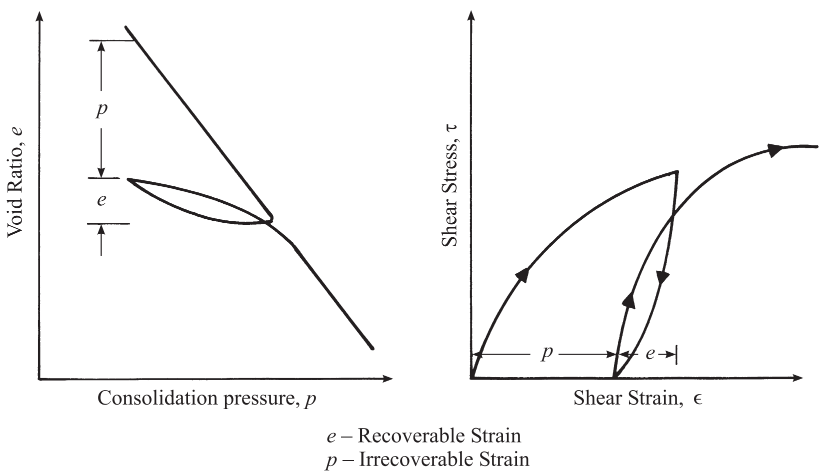

Figure 1.25. Recoverable and irrecoverable components of clay deformation during compression and shear.

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$$p' = \left(\frac{\sigma_1' + 2\sigma_3'}{3} \right) \tag{1.41}$$

$$q' = (\sigma_1' - \sigma_3') \tag{1.42}$$

v is the specific volume i.e., the volume of soil containing unit volume of solid material

$$v = 1 + e \tag{1.43}$$

where e is the void ratio.

Corresponding to the stress parameters p' and q' are strain parameters ε_v the volumetric strain and ε_s, octahedral shear strain:

$$\varepsilon_v = (\varepsilon_1 + 2\varepsilon_3) \tag{1.44}$$

$$\varepsilon_s = \frac{2}{3}(\varepsilon_1 - \varepsilon_3) \tag{1.45}$$

The factor 2/3 appears in the definition of shear strain, ε_s, from the consideration that work done by a small increment of straining is equal to $p'\delta\varepsilon_v + q'\delta\varepsilon_s$. Thus the stress and strain parameters correspond to one another such that multiplication leads to the correct evaluation of the work done due to deformation.

The progress of shearing of a soil sample during a triaxial test can be represented by a series of points describing a line in a three dimensional space with axes p', v and q' (Fig. 1.26). These simply correspond to two orthogonal views of p', v, q' space (Fig. 1.25). Critical state soil mechanics provides a means to calculate test paths in (p', v, q') space. Usually two of the three parameters are determined by the test and there is a simple procedure for determining the third parameter.

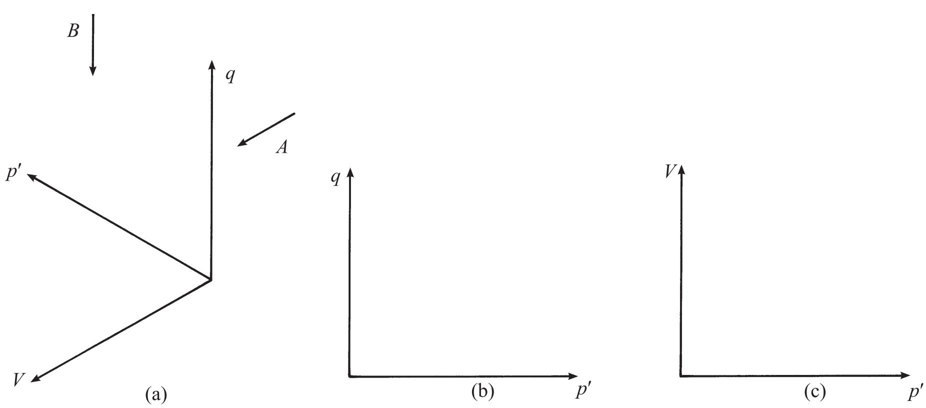

Figure 1.26. Three dimensional space with axes p', v and q'.

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Basic relations: If a saturated soil sample is subjected to isotropic compression and rebound tests then it follows the paths in p', v plots as shown in Figure 1.27.

This is basically similar to the more familiar σ_v, e plots obtained from oedometer tests. The average rebound – recompression paths in ln p', v plots can be regarded as linear paths with slopes $-\lambda$ and $-\kappa$ respectively as shown in Figure 1.27. The equation for the isotropic normal compression line is

$$v = N - \lambda \ln p' \tag{1.46}$$

where N is defined as the specific volume of a normally consolidated clay at $p' = 1$ kN/m^2. The value of N depends upon the units which are used to measure pressure.

Corresponding to Equation (1.46), the average rebound recompression path is given by

$$v = v_k - \kappa \ln p' \tag{1.47}$$

Before encompassing the failure states within the general framework of Cam clay models it is necessary to examine the undrained and drained shear behaviour of saturated clays compressed to different isotropic stress levels, p'.

Figure 1.28 shows the results of undrained triaxial tests on samples compressed to different levels of p' denoted by p'_e. Samples compressed to higher p' sustain higher values of q' at failure. The shape of q' versus ε_1 curves are similar and as such it is possible to normalize paths by plotting q'/p'_e against ε_1 as shown in Figure 1.28. The stress paths followed by a family of such tests in q', p' space are as indicated in Figure 1.29. Since the shapes of these curves are similar, all the curves could be collapsed into one by plotting q'/p'_e. Similar paths and generalizations can also be obtained even in the case of consolidated drained triaxial shear tests (Fig. 1.30). The stress paths followed in drained tests rises at a slope of 3 in q', p'

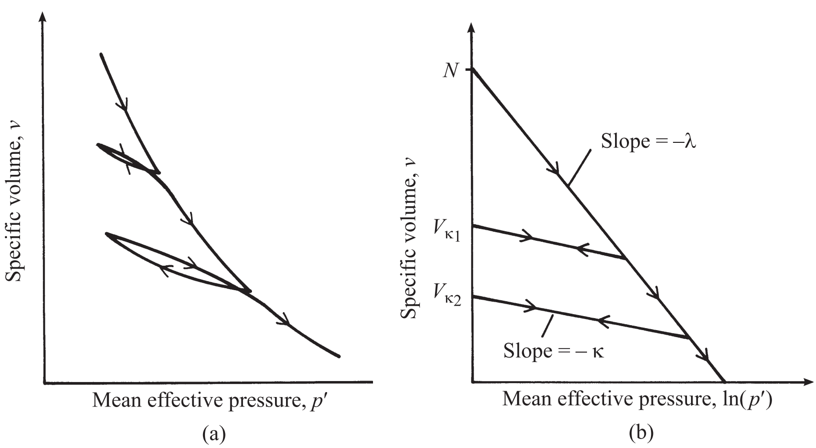

Figure 1.27. Typical and idealized specific volume effective mean stress relations.

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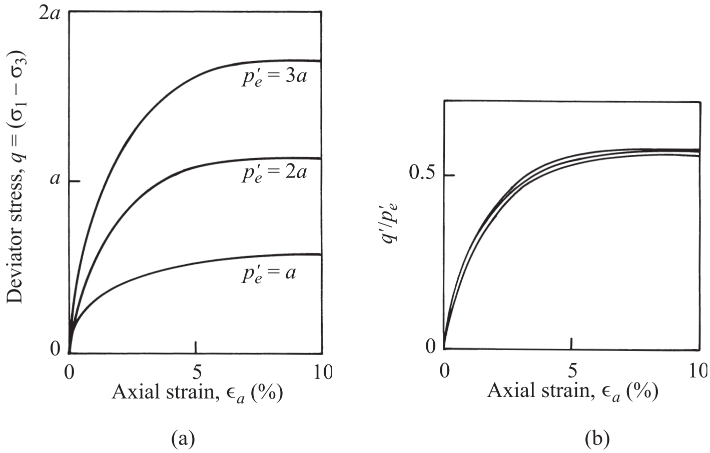

Figure 1.28. Relationships between: a) Deviatoric stress and axial strain, and b) Normalized deviatoric stress and undrained axial strain in undrained test.

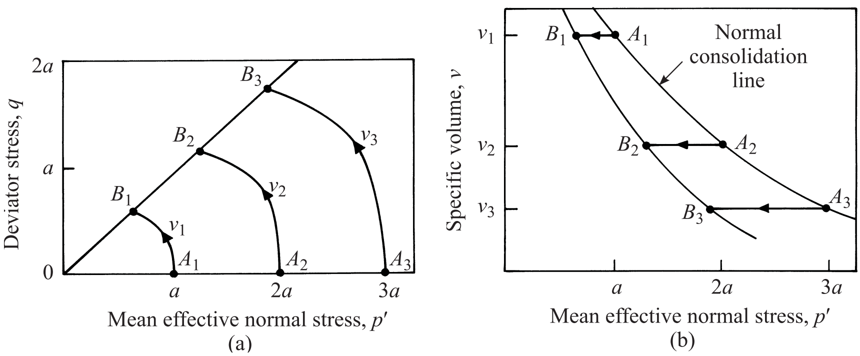

Figure 1.29. Stress paths in: a) q' versus p', and b) v versus p' for normally consolidated clays.

space from the initial value p'_o of mean normal effective stress at $q' = 0$ (Fig. 1.31). The corresponding paths in $v - p'$ space are shown in Figure 1.31.

It is interesting to observe that all failure stress states pertaining to undrained and drained tests define a single straight line through the origin in q', p' space and a single curved path in v, p' space whose shape is similar to normal consolidation line. These are critical state lines (Fig. 1.32). The projection of the critical state line on to the q', p' plane can be expressed by the relation

$$q' = M\,p' \tag{1.48}$$

where M is the friction factor.

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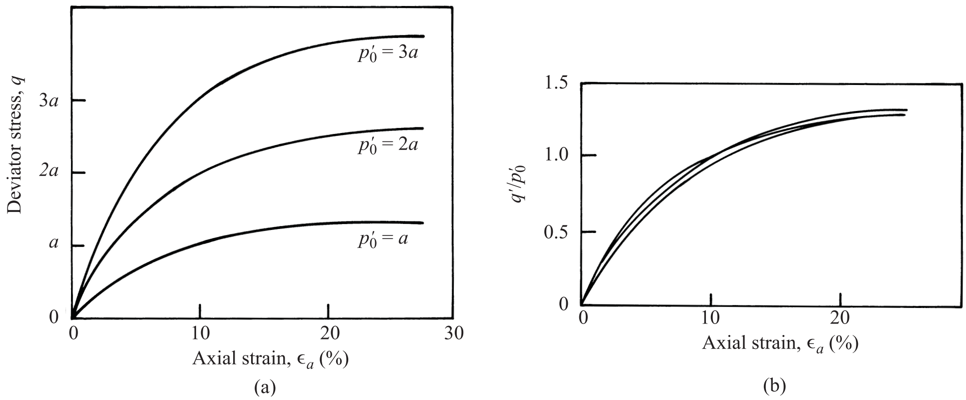

Figure 1.30. Relationship between: a) Deviatoric stress and axial strain, and b) Normalized deviatoric stress and axial strain in drained test.

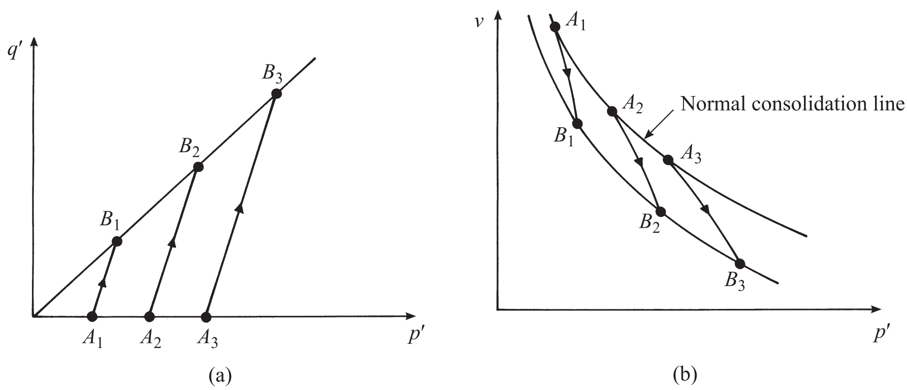

Figure 1.31. Stress paths in: a) $q' - p'$, and b) $v - p'$ space for drained tests on normally consolidated clay.

The projection of the critical state line onto the v, p' plane is curved. This critical plane is curved. This critical state line can be expressed by the relation

$$v = \Gamma - \lambda \ln p' \tag{1.49}$$

where Γ is defined as the value corresponding to $p' = 1.0$ kN/m^2.

The critical state line represents the final state of soil samples in triaxial tests when it is possible to continue to shear the sample with no change in stress states or volume of the sample. Hence at the critical state

$$\frac{dv}{d\varepsilon} = 0,\ \frac{dq'}{d\varepsilon} = 0,\ \frac{dp'}{d\varepsilon} = 0 \tag{1.50}$$

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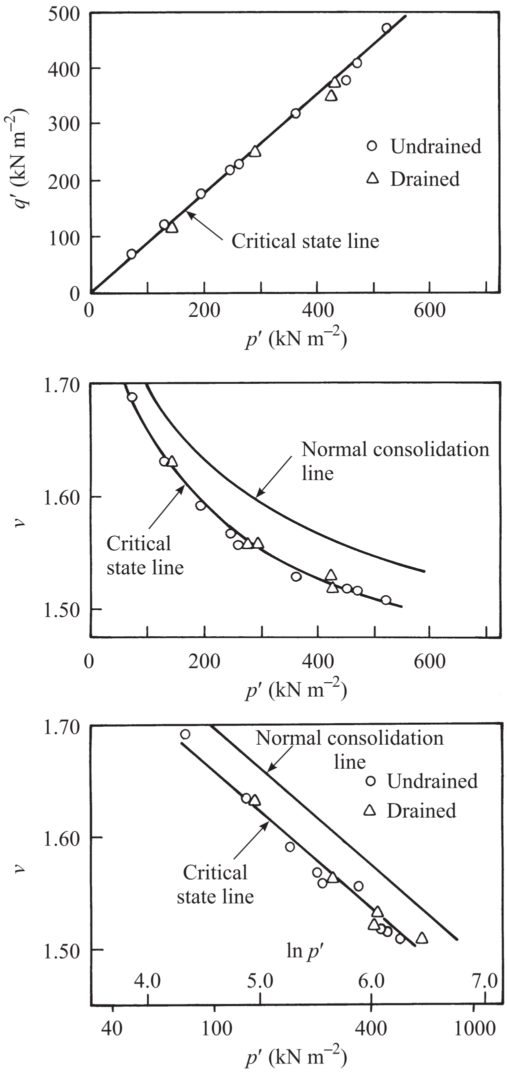

Figure 1.32. Critical state lines in $q - p'$ and $v - p'$ space (Parry 1960).

Figure 1.32 shows the failure points for undrained and drained tests on normally consolidated specimens of Weald clay as reported by Parry (1960) in $q' - p'$ and $v - p'$ plots. The straight line in $v - \ln p'$ represent the critical state line.

Critical state soil conditions describe a curved path in three dimensional p', q', v space both for undrained and drained conditions (Fig. 1.33).

For systematic analysis and computation of strain components the following basic features merit consideration (Chen & Mizuno 1990).

1. The existence of an initial yield surface and subsequent loading surfaces.
2. Formulation of an appropriate hardening rule that describes the evolution of the subsequent loading surfaces.

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3. A flow rule that specifies the general form of stress-strain relationship.
4. A failure criterion which defines the limiting or ultimate stresses.

The Cam clay model provides a means to compute plastic strains for different states of the stress. One of the key assumptions of Cam clay theory is that flow rule follows the normality condition.

This implies that plastic strain increment vector is everywhere normal to the yield locus. Further, the assumption of a pure frictional form of dissipation of energy during shear gives rise to the flow rule expressed as

$$\frac{d\varepsilon_v^p}{d\varepsilon_s^p} = M - \frac{q'}{p'} \tag{1.51}$$

This equation leads to the associated yield curve given by the relation:

$$\frac{q}{Mp'} + \ln\frac{p'}{p'_x} = 1 \tag{1.52}$$

where p'_x is the value of p' at the intersection of the yield curve with the projection of the critical state line at x with zero slope as shown in Figure 1.34.

A family of all such curves at different specific volumes forms a yield surface in $p' - q' - v$ space. As the critical state point (p'_x, q'_x) also lies on the rebound line using which p'_x is eliminated so as to obtain the equation of the yield surface as

$$q' = \frac{Mp'}{(\lambda - \kappa)}\{\Gamma + (\lambda - \kappa) - v - \lambda \ln p'\} \tag{1.53}$$

Figure 1.35 shows the isotropic view of this surface. When the state of a specimen of soil can be represented by a point below the surface, then soil behaviour is elastic. Soil states on the surface indicate yielding, and it is impossible for the soil

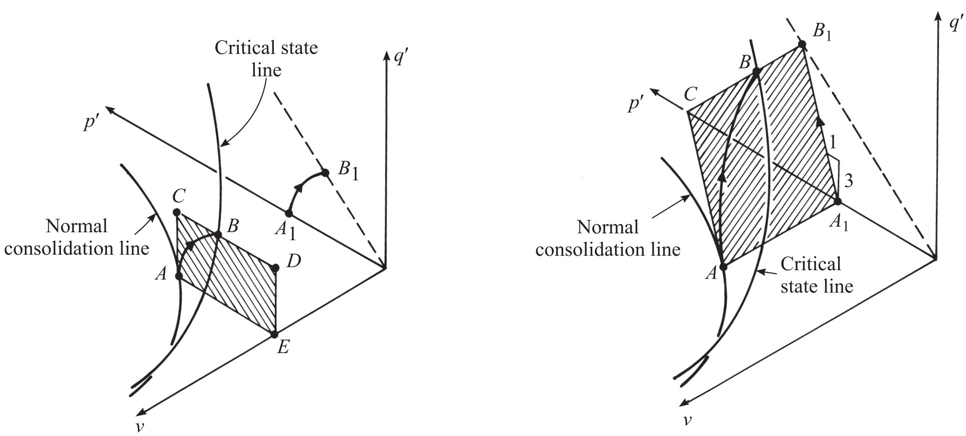

Figure 1.33. The undrained and drained test paths in q', p' and v space.

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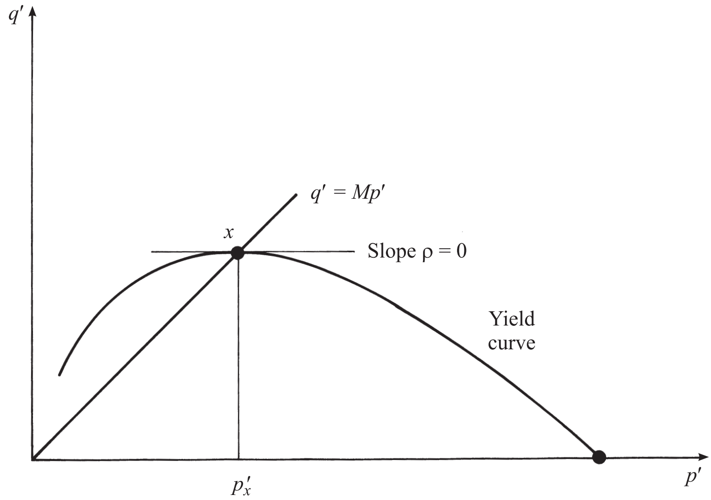

Figure 1.34. Stable state boundary surface in p', v and q' space.

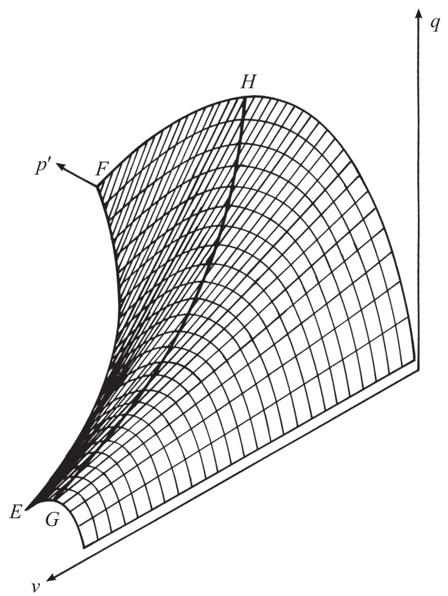

Figure 1.35. Yield curve as predicted from Cam clay model.

samples to exist in states equivalent to points above the surface. As such the surface is known as the stable state boundary surface (SSBS). Elastic straining underneath SSBS corresponds to a movement along a κ line with a corresponding change in v. When such a sample is brought to the point of yield it will lie both on the κ-line and on the SSBS. Therefore the intersection of the SSBS with the κ-line equation results in the current yield surface

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$$q = Mp' \ln \frac{p'_c}{p'} \tag{1.54}$$

The equation for the state boundary surface can be rewritten as

$$v = \Gamma + (\lambda - \kappa) - \lambda \ln p' - \left[\frac{(\lambda - \kappa) q'}{Mp'} \right] \tag{1.55}$$

The form of yield curve which this yield function depicts is shown in Figure 1.36.

From the general expression (Equation 1.53) for the yield surface, expressions for specific cases can be examined, viz.,

1. The isotropic compression line (Equation 1.46) can be obtained. For this case $q = 0$. To satisfy this condition in Equation (1.53)

   $$\Gamma + (\lambda - \kappa) - v - \lambda \ln p' = 0$$

   since $\Gamma + (\lambda - \kappa) = N$,

   $$v = \mathrm{N} - \lambda \ln p'$$

2. The critical state line (Equation 1.49) can be obtained for the condition $q = Mp'$. Equation 1.53 would be of the form

   $$Mp' = \frac{Mp'}{(\lambda - \kappa)} \left[\Gamma + (\lambda - \kappa) - v - \lambda \ln p' \right]$$

   $$1 = 1 + \frac{\Gamma - v - \lambda \ln p'}{(\lambda - \kappa)}$$

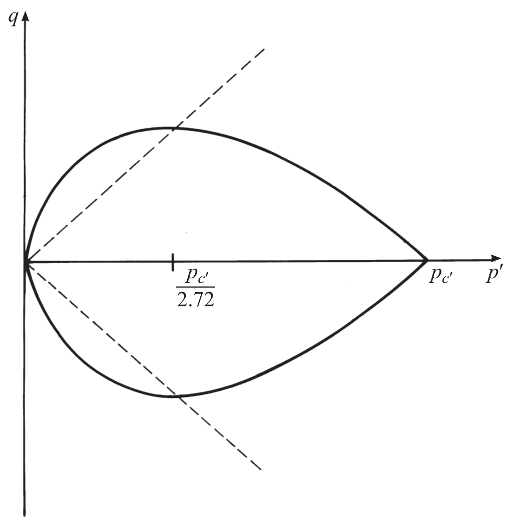

Figure 1.36. The Cam clay yield locus.

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On simplification

$$\Gamma - v - \lambda \ln p' = 0$$

$$v = \Gamma - \lambda \ln p'$$

Further, upon differentiation of Equation (1.54).

$$\mathrm{d}v = -\lambda \frac{\mathrm{d}p'}{p'} - \frac{(\lambda - \kappa)\,\mathrm{d}q'}{Mp'} + (\lambda - \kappa)\, q \frac{\mathrm{d}p'}{\mathrm{d}Mp'^2} \tag{1.56}$$

Change in p' causes not only plastic change of volume but also an elastic change of volume equivalent to a movement on a rebound-recompression line. This is given by the relation

$$\mathrm{d}v^e = -\kappa \left(\frac{\mathrm{d}p'}{p'} \right) \tag{1.57}$$

The plastic (irrecoverable) specific volume change is

$$\mathrm{d}v^p = \mathrm{d}v - \mathrm{d}v^e$$

From Equations 1.56 and 1.57

$$\mathrm{d}v^p = -\frac{(\lambda - \kappa)}{Mp'} \left[\left(M - \frac{q'}{p'} \right) \mathrm{d}p' + \mathrm{d}q' \right] \tag{1.58}$$

This leads to the plastic volumetric strain

$$\mathrm{d}\varepsilon_v^p = -\left(\frac{\mathrm{d}v^p}{v} \right) = \left\{ \frac{\lambda - \kappa}{vMp'} \right\} \left[\left(M - \frac{q'}{p'} \right) \mathrm{d}p' - \mathrm{d}q' \right] \tag{1.59}$$

Upon applying normality conditions the shear strain component would be

$$\mathrm{d}\varepsilon_s^p = \left\{ \frac{1}{M - \dfrac{q'}{p'}} \right\} \mathrm{d}\varepsilon_v^p \tag{1.60}$$

It is further assumed that the elastic shear strains are zero.

The equation for the undrained stress path which is a constant specific volume v section of the general yield surface is of the form,

$$\frac{q}{Mp'} + \frac{\lambda}{\lambda - \kappa} \ln \left(\frac{p'}{p'_0} \right) = 0 \tag{1.61}$$

In an undrained test, since $\delta\varepsilon_v = 0$ i.e., $\delta\varepsilon^e{}_v + \delta\varepsilon^e{}_p = 0$. From this condition the incremental stress ratio for undrained condition can be expressed as

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$$\frac{dq}{dp'} = \frac{q}{p'} - \frac{\lambda}{\lambda - \kappa} M \tag{1.62}$$

The procedure to be followed to obtain the stress-strain path of a clay is to obtain experimentally the constants λ, κ, M, N for the clay and current stress and state i.e., p'_0, v_0. In the case of undrained tests for an applied increment of stress dp', dq is evaluated using Equation (1.61) and vice versa. For these stress increments the strain increments are computed using the relations provided in Equations (1.56) to (1.60) with the current stress and strains updated each time. In the drained tests the increments of stresses applied are known beforehand and only the strain components have to be computed using the relationships provided in the same Equations (1.56)-(1.60).

1.8 PERMEABILITY OF CLAYS

Soils being particulate media in equilibrium, possess voids, mostly continuous, permitting the flow of fluids. Permeability is the property of such porous materials, and governs the rate of flow of a fluid through them. The pores in clay soils are generally so small that the flow of water through them is laminar. However in very coarse soils the flow may be turbulent. Darcy (1856) while studying the rate of flow of water through sand filter beds found that the macroscopic velocity was proportional to the hydraulic gradient:

$$v = \frac{Q}{A} = k\frac{\Delta h}{\Delta x} = ki \tag{1.63}$$

where: Q is the quantity of water flow in cc or ft^3/unit time, A is the cross sectional area of the bed in cm^2 or in^2, v is the flow velocity cm/sec or in/sec, k is the constant defined as hydraulic conductivity, cm/sec, $i = \Delta h/\Delta x$ is the hydraulic gradient.

The Darcy's equation in which k is also known as Darcy's constant. Since i is dimensionless, k has units of velocity. If the properties of pore fluid are considered, Equation 1.63 can be rewritten to include fluid properties as

$$v = k'\left(\frac{\gamma g}{\eta}\right)i \tag{1.64}$$

where: γ is the mass density of fluid, η is the viscosity of fluid, k' is the intrinsic permeability, cm^2, g is the gravity.

In the computation of the velocity of flow from the discharge, the computation is based on the cross sectional area of soil through which the flow occurs. It is obvious that water cannot flow through solid particles but only through interstices in the microstructure. The velocity is superficial velocity, v_s. Consider the unit width of a sample (Fig. 1.37).

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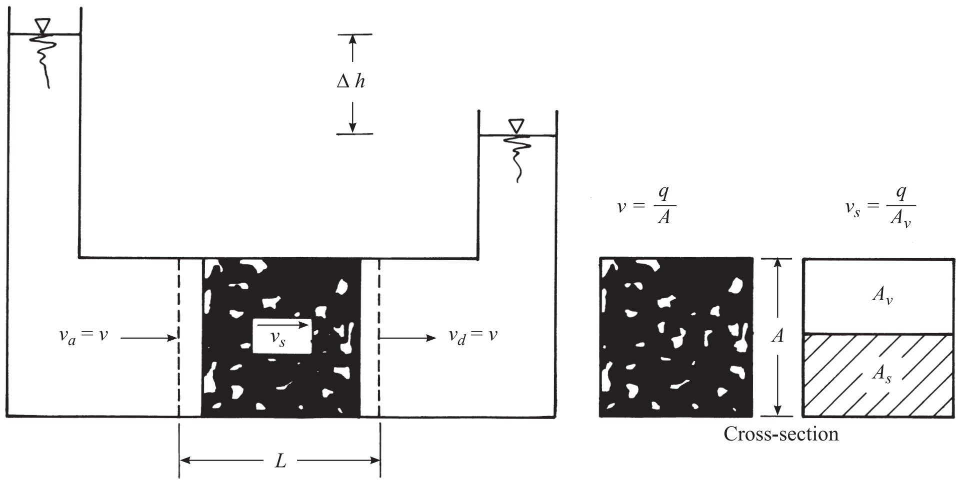

Figure 1.37. Superficial and seepage velocities in laminar flow.

$$e = \frac{v_v}{v_s} = \frac{A_v}{A_s} \tag{1.65}$$

Approach velocity, v_a, and the discharge velocity, v_d, from continuity considerations are equal to superficial velocity, v_s, computed by the considerations of discharge, q, and the total cross sectional area, A, with velocity, v, such that

$$q = vA = v_a\, A = v_d\, A = v_s\, A_v$$

$$\frac{A_v}{A} = \frac{V_v}{V} = n$$

Then

$$v = \left(\frac{A_v}{A}\right) v_s \quad \text{i.e.,} \quad v = nv_s \tag{1.66}$$

The above considerations indicate that the magnitude of k is not altered by use of v with A, being the total cross sectional area.

To consider both the surface area of soil particles and porosity of the soil in flow through capillary tubes, along with the hydraulic radius models of Kozeny-Carman (Kozeny 1927, Carman 1956) the permeability of both coarse grained soils and clays merits examination. According to the Kozeny-Carman equation for cohesionless soils such as sand, the relation between the permeability, k and void ratio, e is of the form

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$$k = C_s\left(\frac{e^3}{1+e}\right) \tag{1.67}$$

where C_s is the shape factor.

For fine grained soils with appreciable clay fraction relation according to the Equation (1.67) is less successful due to presence of long and short range forces of interaction. In proposing theoretical models for the assessment of the permeability characteristics of soils the influence of the following factors should be incorporated:

1. Factors associated with permeant, viz., viscosity, pressure and density.
2. Soil properties such as tortuosity, void ratio, soil water potential and pore size distribution.
3. Soil-water interaction reflected in heat of wetting, ionic concentration and double layer interactions.

Permeability determination: Permeability determinations being done both in the laboratory and in situ. Laboratory methods are appropriate in the case of reconstituted consolidated soils, compacted soils and undisturbed samples. On the other hand field determination has specific advantages, such as that the field sample is much larger, and contains all natural macrofabric features along with natural boundary conditions.

Permeameters are the apparatus for laboratory determination. There are two basic types, the constant head and the variable head. In an apparatus in which the head remains constant the quantity of water flowing through a soil sample of known area and length in a given time can be measured from Darcy's law

$$k = \left(\frac{QL}{hA}\right) \tag{1.68}$$

In highly impervious soils the quantity Q is so small that accurate measurements of its value is difficult. The essential characteristic of the variable head method is that the quantity of percolating water is measured indirectly by observations of the rate of fall of the water level in the standpipe above the specimen. The velocity of fall is $-\mathrm{d}h/\mathrm{d}t$ and the rate of flow according to Darcy's equation is $k(h/L)$ are equated. This yields:

$$-a\left(\frac{\mathrm{d}h}{\mathrm{d}t}\right) = k\left(\frac{h}{L}\right)A$$

upon integration of the above expression the resulting relation is,

$$k = 2.3\left(\frac{aL}{At}\right)\log_{10}\left(\frac{h_o}{h}\right) \tag{1.69}$$

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where, a, refers to area of standpipe and L and A refer to sample length and cross sectional area, h_o refers to head at the start of the experimental at zero time and h after time, t.

With these two basic methods many modifications have been made to take care of many deficiencies apart from enhancing their potential.

In all of the above basic methods the sample mould is of the rigid wall type. For any of the moulds of the above types of permeameters the undisturbed, compacted or slurry consolidated samples cannot be transferred effectively without the risk of leakage of permeating fluid when permeability tests are conducted. As such there is potential for spurious leakage of permeating fluid along the sides of the test specimen. If the sample itself is prepared in these moulds as in the case of compacted clays, the possibility of leakage of fluid along the sides can be averted. For such purposes the double ring compaction mould permeameter has been developed (Fig. 1.38).

To circumvent the disadvantages of the rigid wall permeameter altogether, a flexible wall permeameter has been developed. The samples are confined by a flexible membrane in a chamber so that the confining pressure applied to the outside of the membrane presses it against the sample, conforming to its exact shape and size at all stages of testing. This eliminates the flow along the sides of the sample. Figure 1.39 shows a typical triaxial flexible wall permeameter (Carpenter & Stephenson 1985, Uppot & Stephenson 1989).

More recently test facilities have been developed to determine permeability, not only in the conventional vertical direction, but also in a radial direction. This has relevance for the design of sand drains. Al-Tabba & Wood (1987) have reported the details of the modified oedometer for the measurement of both vertical and radial permeabilities. The details of the setup are as shown in Figure 1.40.

It is not always possible to determine the permeability characteristics of soils in the laboratory due to the difficulty of testing a sample of large size with all the inherent micro- and macro-fabric features. As such, infiltrometers have been devel-

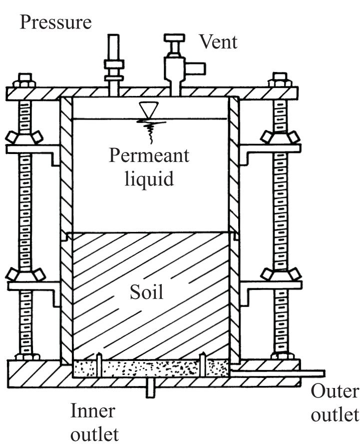

Figure 1.38. Double ring compaction mold permeameter (Anderson et al. 1985).

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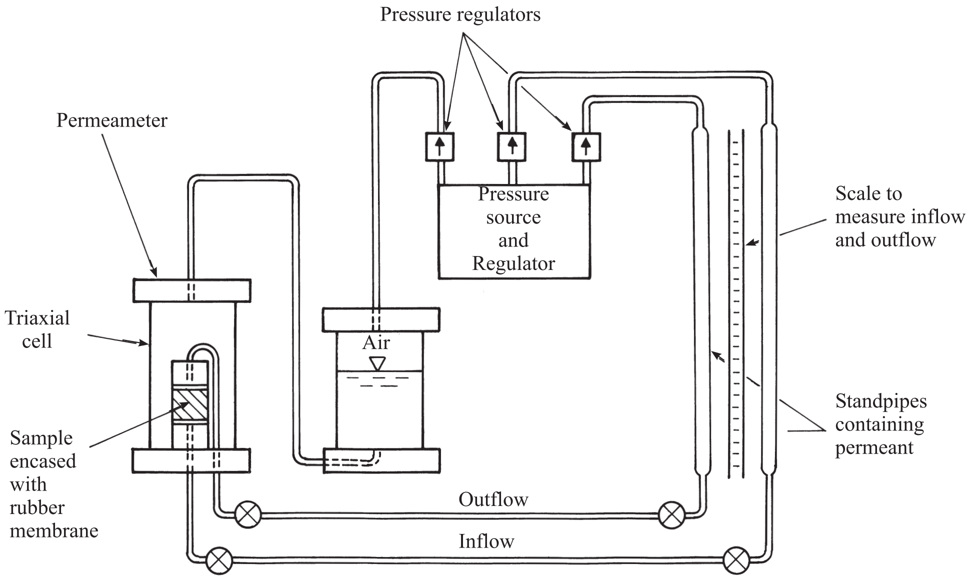

Figure 1.39. Flexible wall permeability apparatus (Carpenter & Stephenson 1985).

oped (Daniel 1989) to determine the permeability of the soils in situ. The open single ring infiltrometer is the simplest in the series. The ring is sealed with a bentonite grout. The test involves filling the ring with water and monitoring the rate of infiltration. This is the rate at which a given volume of water passes into the soil over an unit area per unit time. The rate of infiltration, *I*, is given by

$$I = \frac{Q}{At} = \frac{q}{A} \tag{1.70}$$

where: Q is the quantity of flow, A is the area of flow, t is the elapsed time and q is the rate of flow.

Hydraulic conductivity can be calculated from the relation

$$k = \frac{I}{i} = \frac{IL_f}{\left(H + L_f\right)} \tag{1.71}$$

where: I is the rate of infiltration, h is the depth of ponded water, L_f is the depth of wetting front.

For one dimensional flow conditions L_f would be within the depth to which the ring is installed. It is also possible to assess the depth of infiltration using the volume of the fluid, porosity, dry density, degree of saturation and area of the soil. To overcome evaporation losses and to have the flexibility of handling a low rate of infiltration either by the variable or a constant head test, the ring is sealed at the

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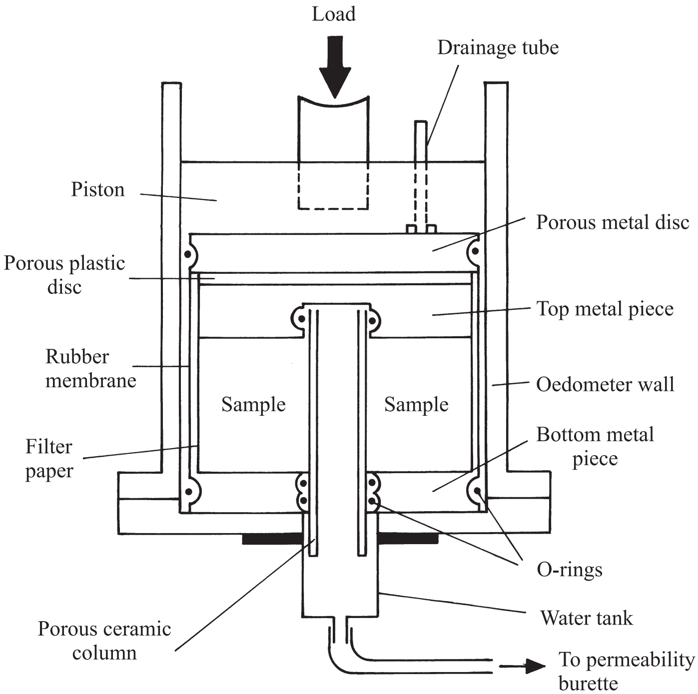

Figure 1.40. Details of modified oedometer for radial permeability test (Al-Tabaa & Wood 1987).

top by a plexiglass lid with provision for fixing a graduated cylinder or constant pressure system along with an arrangement to monitor the volume of inflow water. Figure 1.41 shows the details of the infiltrometer sealed at the top with graduated cylinder for the variable head test. Equations (1.68) and (1.69) can be used for computation of permeability. Even by using the above method it is not possible to take into account the influence of greater depths.

Unless the in-situ soil stratum is homogeneous and devoid of macro-fabric features, even laboratory tests on undisturbed samples would not be representative of soil conditions over a large volume. Hence in situ tests are inevitable. Figure 1.42 presents the basic schematic of one of the several methods known as the pumping test, which is used to determine the coefficient of permeability in the field. The method involves the use of three wells (although two wells may suffice) and a pump. A perforated casing is sunk through the pervious stratum into the impervious stratum, if one exists, or to a considerable depth below the water table if an impervious stratum does not exist. This is to be used as the test well. Two additional perforated casings are sunk at some distance from the test well (30- to 60-m spacing) to a depth well below the anticipated draw-down curve shown in Figure 1.42. These are observation wells.

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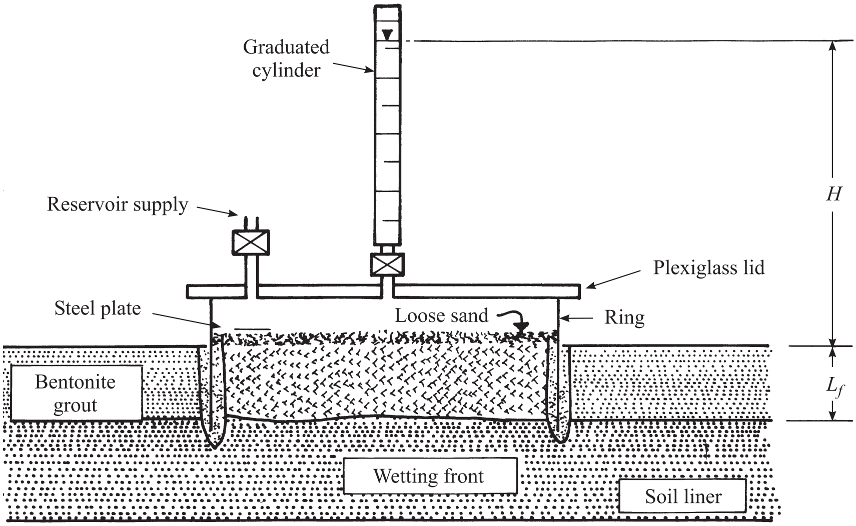

Figure 1.41. Details of scaled single ring infiltrometer (Daniel 1989).

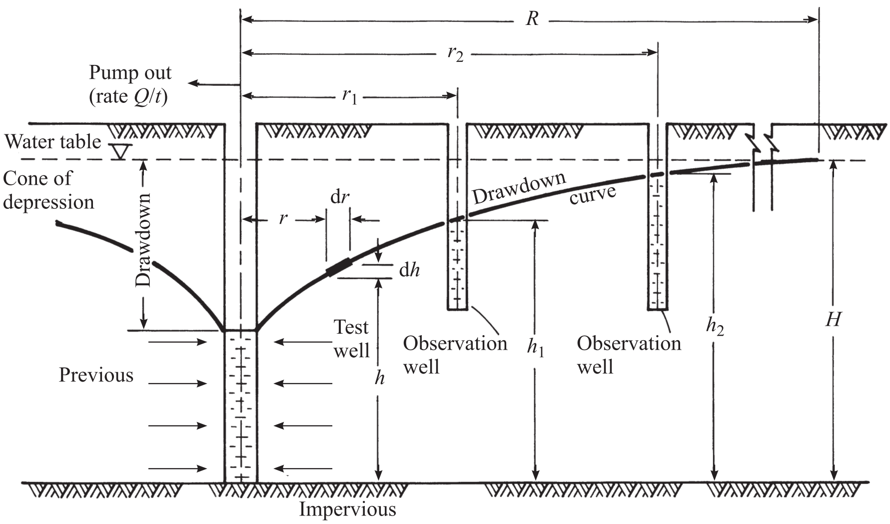

Figure 1.42. Illustration of the principle of the well point method.

The water in the test well is then pumped out until a steady-state flow into the well is apparent. This could be determined by observing that the level of water in the test well remains at a relatively fixed elevation with continuous pumping. When this occurs, the level of the water in the two observation wells is recorded; also recorded is the distance of each observation well from the test well.

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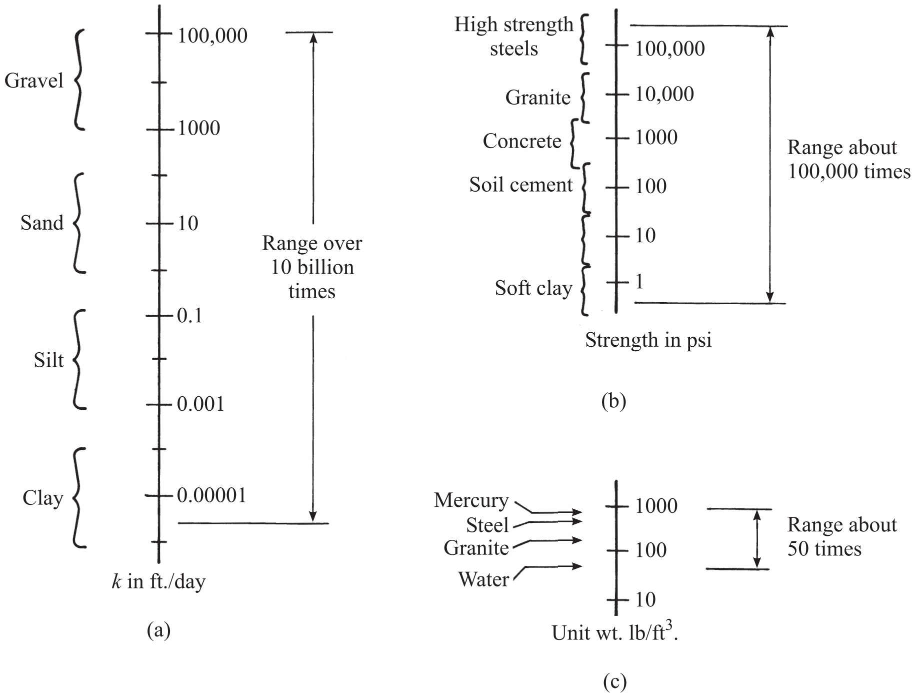

Figure 1.43. Variation of permeability of soils compared with strength and unit weight of materials (Cedergren 1967).

Now let us assume that the water flows into the well in a horizontal, radial direction through the walls of the casing. The surface area of a cylindrical section of radius r, and height, h, is $2rh$. From Darcy's law where $i = \mathrm{d}h/\mathrm{d}r$, and assuming that the total discharge Q equals the flow into the well (steady-state flow), the Rate of pumping is Q/t

$$\frac{Q}{t} = k2\pi rh\left(\frac{\mathrm{d}h}{\mathrm{d}r}\right)$$

separating variables,

$$\left(\frac{Q}{t}\right)\left(\frac{\mathrm{d}r}{r}\right) = k2\pi h\mathrm{d}h$$

upon integration

$$\left(\frac{Q}{t}\right)\ln\left(\frac{r_2}{r_1}\right) = \pi k\left(h_2^2 - h_1^2\right)$$

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from which

$$k = \left[\frac{Q \ln \frac{r_2}{r_1}}{\pi t \left(h_2^2 - h_1^2 \right)} \right] \tag{1.72}$$

It is interesting to note that there is no other engineering property of soil, in fact of any construction material, which is as variable as permeability. The range of permeability is over ten billion times for any soil or rock whereas the variation of bulk density of the material is in the range of fifty times and that of strength is of the order of one tenth million (Fig. 1.43) (Cedergren 1967). Thus the relative comparison of permeability values is done by orders of magnitude rather than by arithmetical ratios. For example, let us consider two values 5×10^{-8} and 2.25×10^{-8} for comparison. The arithmetical ratio is 2.22 whereas comparison by orders of magnitude it is only 0.275. This value is the difference between the two values divided by one order of magnitude i.e., ten.

1.9 CONCLUDING REMARKS

In retrospect, certain principles of engineering significance have been briefly discussed. This brief discussion is intended to provide only necessary background information so that the treatment in the subsequent chapters can be better appreciated. Before discussing the principles of generalization of fine grained soil behaviour it is necessary to provide a brief description of the various methods of soft ground improvement in order to highlight the situations wherein assessment of soft clay behaviour becomes apparent.

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CHAPTER 2

Soft clay engineering

2.1 INTRODUCTION

The design and construction of infrastructure facilities in the soft soil of coastal regions made necessary by extensive urbanization and industrialization, quite often in many lowland areas, entail detailed assessment of their engineering properties and their adequacy to meet the practical needs. In recent years the sharp decline of ground available for use is evidenced by the number of constructions spreading out over both land and sea, built to ever-deeper levels below ground, and rising higher into the sky. The need to improve unfavourable ground to serve the specific need poses a challenge to geotechnical engineers. In many land reclamation projects, strengthening of the reclaimed ground is inevitable. There are situations where special techniques may have to be adopted during the construction phase itself in order to allow the project to proceed smoothly. In addition, it has become increasingly necessary to strengthen the soft soil under existing structures in distress to restore their stability.

The desirable practical requirements of the in-situ ground are increased strength, reduced compressibility and appropriate permeability to take care of stability, settlement and ground water, and other environment-related problems.

To achieve these requirements, the basic approaches employed are drainage, densification, cementation and reinforcement. These techniques, developed hundreds of years ago, remain valid even today. The art of building taking into account the conditions of the ground were fairly in use even in the 16th and 17th centuries, almost at the same time when Hooke's and Newton's laws were conceived. During the course of developments over the past few decades, at various stages, well documented state of-the art reports dealing with the techniques of improving ground conditions, design methodology and specific case records of adopting different methods have been published (Mitchell 1970, 1981, Broms 1979, 1987, Kamon 1991, Kamon & Bergado 1991, Bell 1993, and others). The recent book by Bergado et al. (1996) provide extensive details of the application of many soft soil improvement techniques. Most of the recent developments in the above basic approaches are highly innovative due to an all-round development in appropriate field equipment for implementation and instrumentation for monitoring to realize improvements in soil conditions.

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The relevance of this chapter, in the present context, is primarily to identify the situations where an assessment of soft clay behaviour is warranted at various stages of construction on soft clay with or without ground improvement.

Before a brief discussion of the different methods available for improvement of soft ground, it might be worthwhile to discuss how soft clay formations originate in coastal regions and their specific characteristics which warrant ground improvement.

2.2 FORMATION OF CLAY SEDIMENT

Broadly, soft ground encompasses soft clay soils, soils having large fractions of fine silt, peat and loose sand deposits below ground water table (Kamon & Bergado 1991). From the geotechnical engineering viewpoint, soft ground refers to deposits having potential for high compressibility and possessing low strength. The discussions in the present context pertain only to soft clay deposits.

Soils being essentially particulate media, clay sediment formations are influenced by the nature of solid soil particles and their interactions with the surrounding pore fluid medium. Stress, time, and environment are dominant factors in soft clay formations. Before a discussion of the geological aspects and depositional environments responsible for the formation of deposits, the physical and physico-chemical interactions between clay particles and water are briefly elucidated.

2.2.1 *Soil particles*

As discussed earlier, soils in nature encompass a wide range of particle sizes ranging from coarse sand of 2 mm to clay colloids with the smallest size of stable unit being 10 Å (1 Å = 10^{-8} cm = 0.1nm). Generally particles in the coarse range are bulky and isometric in shape and exist as individual stable units. The common minerals constituting these particles are silicates (feldspars), oxides (silica and iron), carbonates (calcium and magnesium) and sulphates (calcium). Generally, finer soil fractions are made up of clay minerals. These are hydrated aluminium, and iron in a crystalline form of relatively complicated structure. Clays range in mineralogical composition from kaolins, made up of individual particles which cannot be readily divided, through illites to montmorillonites and other non-sheet-clay minerals. A more complete discussion on the structure and composition of clay minerals can be found in the book by Grim (1968).

2.2.2 *Soil-water interactions*

It has been very well established that the surfaces of soil solid particles carry electrical charges. Since water molecules are dipolar in nature it is logical to ex-

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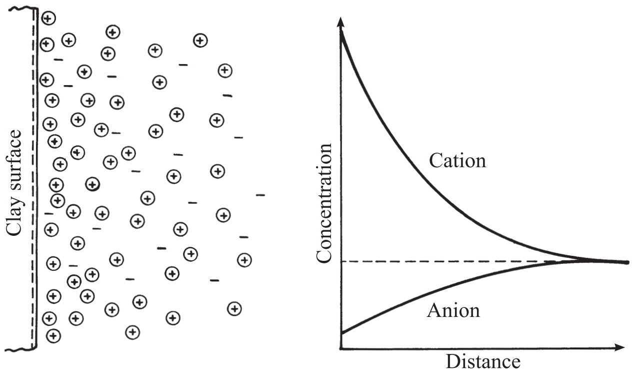

Figure 2.1. Ion distribution adjacent to clay particle surface.

pect electrostatic interactions between clay particles and water with the level being defined by the quantum of surface charge per unit mass. In coarse-grained soils the magnitude of surface charges per unit mass is relatively low. Hence the presence of water does not significantly contribute to the internal force field. On the contrary, clays are strongly influenced by the presence of water due to their high surface activity. The clay-water interaction results in a tendency of the counter ions (resulting from the surface ionization of the clay particle) and other dissociated ions (exchangeable cations present in the water) to diffuse away from the surface, to counterbalance the electrostatic attraction. This defines the concentration of ions at any point since the electrostatic attraction falls off with the distance from the surface. The concentration of the attracted cations will also diminish while the concentration of counter ions increases with distance from the surface (Fig. 2.1). Such a distribution of ion concentration with distance resembles atmospheric ion distribution. The charged surface and the strongly held cation at the surface together with the relatively mobile counter ions in the medium adjacent to the surface are considered to be two layers. Hence the whole system is referred to as the 'Diffuse double layer'. The concentration of charges is greater near the surface and diminishes with distance away from the particle surface.

In suspensions, clay particles of colloidal size with adsorbed and free water molecules around them in an electrolyte suspension are subjected to mutual random impacts. This causes the clay particles to experience Brownian movements. Such random movements will, from time to time, bring clay particles together to distances within the range of interparticle forces. Then the subsequent behaviour of the particles will depend on the net force between them at the separating distance to which the chance movements have brought them closer. The three stages of sediment diagenesis, viz., pre-depositional, depositional and post-depositional, have a significant role to play in soft clay layer formation (Silva 1974).

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If the net force between the particles is repulsive the clay water system tends to be in a dispersed system. If it were the case that all clay particles are of the same clay mineral and the suspension is uniform, it is very likely that a dispersed structure is realized as soil-water reduction due to overburden loading is resisted by the net interparticle repulsion. On the contrary, if the net force is attractive due to higher concentrations, the chance approach between particles in the sediments may bring them even closer. This leads to coagulation. If the particles in suspension were of different sizes, the larger coagulated particle would probably come across smaller particles and further coagulate, contributing to a faster downward movement in the suspension. Under these circumstances the clay particles, as flocs, settle relatively rapidly. The sediment at this stage is likely to be of knit assemblages of grouped particles of a large number of individual particles.

Sediment formation characteristics have been studied in detail by Imai (1981), according to whom the different stages identified are non-settling but with floc formation, a second stage of settlement of such flocs, with the last stage being self-weight consolidation. Further the sediment formation from suspensions is controlled mainly by factors such as: 1) Current velocity and wave energy, 2) Amount and composition of suspension load, 3) Salinity 4) organisms, and 5) Organic matter (Brenner et al. 1981).

After deposition in a certain environment the clay sediments may undergo various diagenetic changes depending upon pressure, time and changes in the environment. Young sediments, such as soft clay deposits formed during reclamation of land would undergo little diagenesis. As the overburden becomes appreciable the soft clays consolidate. Tan (1995) has provided an exhaustive geotechnical perspective on the formation of reclaimed land right from sedimentation to consolidation. If cementitious materials are present, the soft clay deposits may as well be cemented.

If cementation does not take place during the earlier stages of formation, the clay gets consolidated with continuous adjustments in the soil fabric taking place even after the end of the primary consolidation phase. Bjerrum (1967) termed the reduction in volume under constant external load, without any development of measurable pore pressure and its dissipation, as delayed compression. This term is often taken as being synonymous with secondary compression. Figure 2.2 is a schematic plot of void ratio versus consolidation published by Bjerrum (1967, 1972, 1973) to illustrate the effect of time (geological age) on the compressibility characteristics of a normally consolidated soft clay. For example, if AB is the decrease in void ratio due to delayed compression, upon subsequent loading due to additional overburden the compression path followed is that of the normally aged consolidated path shown in Figure 2.2. The pressure σ_{vc}' is often designated as critical pressure or apparent preconsolidation pressure.

Post-depositional processes are many and have profound effect on the geotechnical properties of soft clays. Of the several processes, leaching of cementation compounds are briefly discussed.

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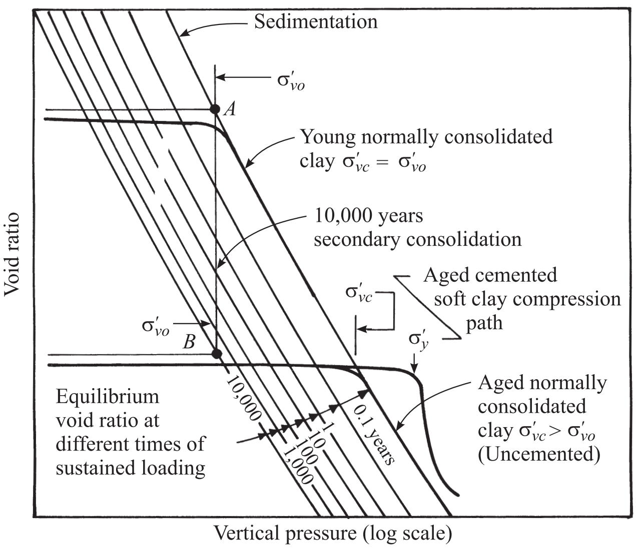

Figure 2.2. Effect of geological history of normally consolidated clays in uncemented and cemented states (Bjerrum 1973).

Leaching: Removal of salts from the soil profile can take place due to hydraulic gradient or by diffusion. Leaching requires the availability of fresh water. This may become available in the form of infiltrating rainwater or slow, upward-moving artesian groundwater. Pioneering work on this aspect of marine soft clays is due to Rosenqvist (1953) and Bjerrum (1967). Skempton & Northey (1953) ascertained from detailed leaching experiments on sensitive soft clays that the liquid limit and remoulded strength decreased with the removal of salt, while water content and the undisturbed strength remained practically unchanged. Consequently, the liquidity index and sensitivity increased. Analysis also revealed that remoulded strength was a unique function of the liquidity index. Bjerrum (1973) found that leaching causes a reduction in undrained strength and an increase in the compressibility of clay.

Cementation: This is a diagenetic process. Bjerrum (1967) defines a cemented clay as one which contains particles held by strong bonds which are of a different nature to the equilibrium state which are due to interparticle forces similar to those operative in non-cemented clays. No universal agreement currently exists regarding the exact nature of cementation bonds although there is general agreement that cementation bonds develop between the structural units of fabric. Calcium carbonate and amorphous silica are two of the common cementing agents generally encountered in cemented soft clay formations. The most significant influence of cementation is on sensitivity. This is usually defined as the ratio of the

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undisturbed shear strength to the remoulded shear strength at the same water content. Sensitivities of 2 to 4 are very common among normally consolidated clays and even with values of 4-8 cannot be ruled out. Heavily overconsolidated clays are insensitive (Skempton & Northey 1953). On the other hand quick clays are also encountered. These are clays whose consistency changes by remoulding from a solid to a viscous fluid. Such quick clays, according to Rosenqvist (1953), are further subdivided into four groups viz., 8-16 slightly quick clays, 16-32 medium, 32-64 very quick and greater than 64 clays are regarded as extra-quick clays. Further it has been the general observation that quick clays have a low plasticity index in the range of 10-15. Critical analysis of the various factors responsible for the formation of the different levels of quick clays has been made by (Quigley 1980) (see Table 2.1).

Originally it was believed that low salinity as a result of leaching was responsible for producing a quick clay (Rosenqvist 1953). Although Norwegian clays were found to conform to this phenomenon, in the case of Canadian and Swedish clays sensitivity is not uniquely dependent on salt concentration. It can be stated that quick clay cannot be obtained by leaching alone, if the pore water contains Na^+, K^+, Ca^{2+} and Mg^{2+}. Leaching will only cause a relative enrichment of divalent cations and the clay will not tend to be a quick clay.

A general theory has been proposed by Rosenqvist (1975, 1978) for quick clay formation. The specific processes focused on are that:

Table 2.1. Factors affecting sensitivity (Quigley, 1980).

$S_t = \dfrac{S_{\text{und}}}{S_{\text{rem}}}$	*Factors producing high undisturbed strength and high Sensitivity* 1. Depositional flocculation Saline (low zeta potential) High sediment concentration divalent cation adsorption 2. Slow increase in sediment load 3. Cementation bonds: Carbonates & sesquioxides (amorphous)
	Factors producing low remoulded strength and high sensitivity 1. High water content ($w_n > w_L$) Underconsolidation or decrease in w_L > decrease in w_n 2. Low specific surface of soil grains. High silt content or high rock flour content in < 2 mm fraction. High primary mineral = low clay mineral content 3. High zeta potential (expanded double-layers = high interparticle repulsion = dispersed or peptized state). Low salinity by leaching (< 2 g/l) Organic dispersants (anion adsorption) Inorganic dispersants (anion adsorption) High monovalent cation adsorption relative to divalent cations 4. Low amorphous content 5. Low smectite content

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1. Non-expanding clay minerals would sediment in a flocculated state. This is attributed to the low zeta potential of the diffuse double layer either because of salinity of the pore fluid or due to strongly held counter ions.
2. The potential of a diffuse layer increases after slight consolidation of the deposited sediment. Leaching of salts would also contribute to this situation.
3. Due to high repulsion, reflocculation is inhibited and, as the water content reduces, a specific clay microstructure is formed.

As a result of extensive and intensive efforts, both in the laboratory and in the field, particularly in Norway, Sweden, Eastern Canada and Japan over the past 40 years, depositional and post-depositional processes in the formation of clay deposits are now fairly understood. Another post-depositional factor which merits consideration is land subsidence due to continuous withdrawal of the groundwater. This has been the dominant factor in the formation of the low lands of Saga Plain (Miura et al. 1988, Miura & Madhav 1994). This is a lowland affected by the six-metre tides of the Ariake Sea. The plain is underlain by 15-40 m of soft, compressible and highly sensitive marine (Ariake) clay below which aquifers of water bearing strata exist. Land subsidence started about 35 years ago due to ground water withdrawal. Subsidence of the order of 80 cm to one metre has been observed. The rate of subsidence accelerated to 16 cm per year due to the extremely hot summer and great demand for water in 1994 (Miura et al. 1995).

Generally, the importance of geological and physico-chemical factors in the interpretation and analysis of test data on compressibility, shear strength and permeability have been clearly recognized.

2.3 INHERENT CHARACTERISTICS OF SOFT CLAYS

For a rational approach to find practical solutions to the geotechnical problems encountered in soft clays, it is necessary to determine the compressibility, shear strength and permeability characteristics of such clays. Since the soft clays most often are encountered with some degree of natural cementation, it is necessary to examine how these basic engineering properties compare with those of uncemented saturated clays.

2.3.1 *Compressibility*

Naturally cemented clays exhibit greater resistance to compression with commensurate yield resistance. Soils can acquire cementation bonding both in their normally consolidated and overconsolidated states. In the former case it tends to be soft and sensitive and in the latter case stiff and cemented. Depending on the intensity of the cementation and the initial state of the soil, different soil states ranging from soft highly sensitive to highly stiff are possible (see Fig. 2.3). The stiff cemented soils are generally not problematic as their compressibility is low

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in the stress range of engineering interest. Hence it would be of interest to examine more intensely the behaviour of soft sensitive clays.

A typical compression path of soft cemented soil is of inverse – *S*-shape. The three zones of the compression path are as shown in Figure 2.4. In zone 1, i.e. up

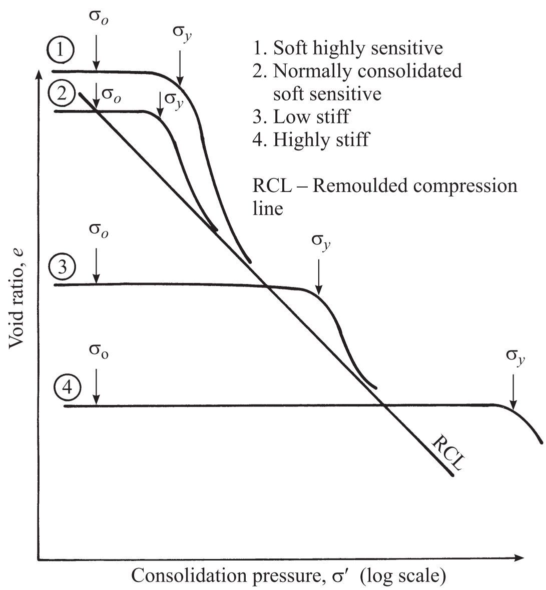

Figure 2.3. Schematic representation of soils of different degrees of cementation.

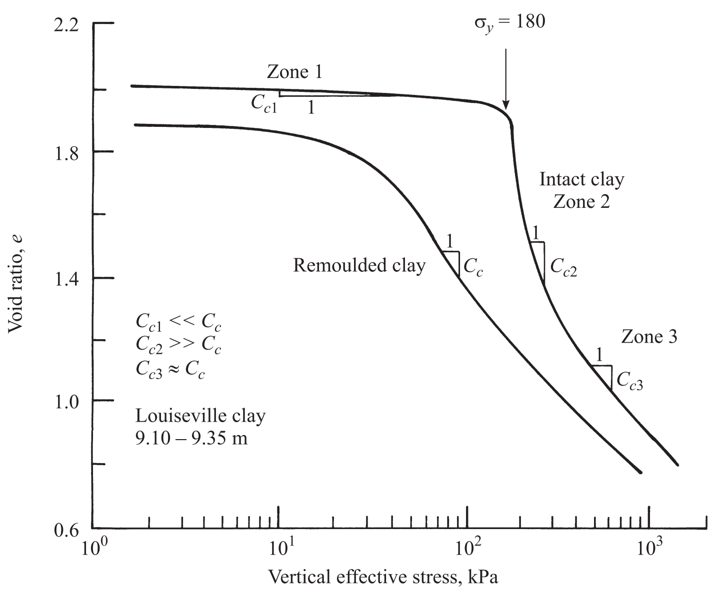

Figure 2.4. Typical compression path of soft sensitive clay in undisturbed and remoulded states.

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to yield stress σ_y' the compression is practically negligible, beyond which compression is high (zone 2) which in zone 3 changes its curvature, seemingly appearing to merge with the compression path of the same clay in its completely remoulded state. The compression index C_c for the entire stress range can be regarded as being of the same magnitude. On the contrary, as the yield stress level increases the stress range over which negligible compression is experienced would also increase. This would provide a higher stress level to which soft clays can be loaded without the risk of culminating in appreciable settlement. Beyond the first zone there is sudden compression of the clay, with the compression index far greater than that for the remoulded state. In zone 3 the compression index reduces to nearly the same as that of the remoulded state of the same clay.

2.3.2 *Shear strength*

The consistency of a clay can be described by its unconfined compressive strength, q_u or by its undrained shear strength S_u (= $q_u/2$). Clay is regarded as very soft if its unconfined compressive strength is less than 25 kPa and as soft when the strength is in the range of 25 to 50 kPa (Terzaghi & Peck 1967).

Soft clays can exhibit this order of undrained strength even in uncemented states. As the sediments get compressed due to overburden the effective stress increases with a commensurate increase in the undrained strength. For the clays to exhibit strength in the range of 25-50 kPa without cementation, water contents have to be lower than their liquid limit water contents. For example, let us consider three soils compressed monotonically from a high water content to a consolidation pressure of 200 kPa (Fig. 2.5). Let the initial water contents be in the range of their respective liquid limits to be devoid of any stress history effects. The undrained shear strength is in the same order even though the water contents of the clays are distinctly different. It is possible that clay may be soft from a shear strength point of view, but the compressibility could be low if the liquid limit water contents are relatively low. On the other hand if the liquidity index of a soft clay is greater than unity, with water contents higher than the liquid limit, the undrained strength of such clays can be in the strength range cited above primarily due to cementation. This is due to the fact that for water contents corresponding to their liquid limit the undrained shear strength is only in the range of 1.5 to 2.5 kPa. Hence the additional strength contributed is by cementation bonds. As such the undrained strength would only become a viable parameter to designate a clay as soft clay.

Most often soft cemented clays exhibit a relatively high degree of anisotropy when compared to soft uncemented clays. This arises due to microstructural, anisotropic conditions. In the initial stages of clay deposit formation at K_o-condition the anisotropic microfabric could be pinned down by cementation between clay aggregates thereby enhancing the anisotropic characteristics of soft cemented clays. This has been identified as structural anisotropy (Parry & Wroth 1981). The

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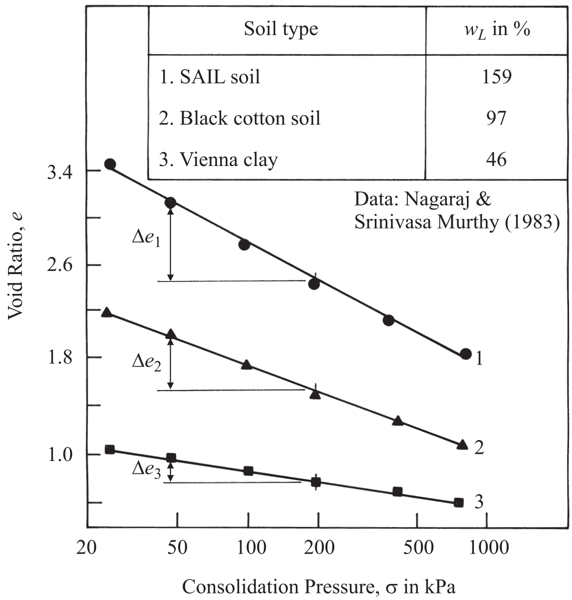

Figure 2.5. Compression paths of three clays from their liquid limit water contents.

strain increments in mutually perpendicular directions would be different for the same magnitude of imposed principal stress increments. Apart from this, stress anisotropy prevails in soft cemented clays. The anisotropy has a marked effect on the undrained shear strength, because it dictates the stress path and consequently the point where the stress paths meet the failure envelope. This tends to yield a decreasing trend in triaxial compression strength as the orientation of the specimen from vertical to horizontal is considered.

Another factor to be considered in the analysis of the strength of soft clay is the influence of the strain rate on the undrained shear strength. It has been observed in some clays that there is a drop of 15% in shear strength for a tenfold decrease in the testing rate. Consequently the testing rates in laboratory considered are to be appropriate.

In the development of geotechnical engineering a stage has been reached when it is no longer necessary or advisable to make a distinction between strength and consolidation or between stability and settlement. They are all facets of the mechanical behaviour of soils which are intimately interconnected.

2.3.3 *Permeability*

The importance of permeability of natural soft clays is increasingly realized in solving many geotechnical problems such as the design of waste disposal facilities, in the analysis of ground water regimes in slopes and in the determination of seepage and in the computation of the rate of settlement. While engineers were concerned with augmenting ground water supplies, the main consideration was

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that of the permeability of coarse-grained soils. Clays in a way were treated as almost impervious. In recent times, the need for understanding the permeability behaviour of fine-grained soils has assumed considerable importance, owing to their use as natural clay liners and in slurry walls to impede the movement of leachates which could contaminate ground water.

The analysis of engineering problems requires two types of permeability values viz. the magnitude and anisotropy of 'k' at in-situ void ratio and its variation as clays are compressed. Mesri & Tavenas (1983) contend that for the range of stress normally encountered, the linear relationship between the void ratio and logarithm of permeability is tenable. This fact has been further strengthened by Tavenas et al. (1983a) for Champlain, Canadian and Swedish clays. The merit of such a linear relationship is to define permeability in terms of a permeability index, defined as the slope of $e - \log k$ path, and to relate it to the change in void ratio.

$$e = C_k \, \Delta \log k \tag{2.1}$$

where C_k is the permeability index similar to the compression index C_c and dependent on void ratio. Tavenas et al. (1983b) and Leroueil et al. (1990) analyzed a considerable volume of data and as a first approximation suggest an equation of the form:

$$C_k = 0.5 e_o \tag{2.2}$$

where C_k is the permeability index and e_o is the initial void ratio. It has also been observed that Darcy's law is valid for natural soft clays for gradients ranging from 0.1 to 50. Leroueil et al. (1990) have also examined the permeability anisotropy of clays as a function of strain. Their findings are that the $e - \log k_v$ and $e - \log k_h$ relationships are essentially parallel and it follows that permeability anisotropy does not vary significantly for strains up to 25%. Further $C_k = 0.5\ e_o$ is valid for both vertical and horizontal permeabilities.

The above brief discussion of the specific characteristics of soft cemented clays in relation to soft uncemented clays can be summarized as:

1. The compressibility of cemented soft clays drastically changes from almost negligible levels to far higher levels than that of the same clay in its remoulded state (devoid of cementation bonds) beyond the yield stress level.
2. The available undrained shear strength is commensurate with cementation bond strength. Values higher than this can be realized only if the commensurate increase in conventional effective stress takes place beyond cementation bond strength. This would be possible if corresponding compression takes place with dissipation of pore water pressure.
3. The permeability of soft cemented clays is a function of the initial void ratio. The $e - \log k$ relation can be characterized by a linear path.

Before embarking upon the ground improvement methods to alter the compressibility, strength and permeability characteristics of soft ground, it would be worthwhile to examine possible alternative approaches to ground improvement to handle practical situations.

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2.4 ALTERNATIVE APPROACHES TO GROUND IMPROVEMENT

2.4.1 *Bypass poor ground*

Relocation of a structure to an acceptable site is usually not possible due to strategic needs. It is likely that one may not find a location with appropriate engineering properties in the whole region. One way of bypassing the poor ground is to reach greater depths, at which level the soil properties are such that the load can be transferred both with regard to stability and settlement considerations. Pile foundations, with or without raft foundations, taken down to a hard stratum, is one such method of by-passing the soft ground. Even for identification of soft ground conditions which do not meet the practical requirements it is necessary to assess the engineering properties of the whole region up to significant depths as expeditiously as possible. *For this it is necessary to have simple and quick methods to assess the compressibility, strength and permeability characteristics with the aid of minimum input parameters normally determined in routine investigations.*

If bypassing of soft ground is not possible the next alternative is to examine the replacement or displacement of the soft ground.

2.4.2 *Replacement*

If the thickness of the soft ground is shallow, i.e. 2 to 3 m, the possibility of removal of the material by excavation and its replacement by inert, coarse frictional material merits examination. Such methods using sand, gravel or rockfill have been used both for road embankments and buildings (Thorburn & MacVicat 1968). Even the possibility of removing the soft material, mixing it with quicklime or cement, partially drying it and recompacting it in situ, has received due attention. Obviously, for resorting to this technique, ground water levels and their fluctuations must not pose any problem.

Draglines and dredgers are usually used for excavation of soil under water. The sides of the excavation have to be stable. The slope depends upon the shear strength of the soil and the depth of the excavation. The width of the excavation for an embankment varies between the road width and a line with a slope of 1.5:1 or 1:1 which extends from the edge of the embankment as shown in Figure 2.6. The depth of the excavation can be reduced to 1 to 2 m, if rockfill were to be used as fill in excavation. For very wide embankments and when the thickness of the compressible layers is relatively small, only the soil along the edges of the embankments is excavated and replaced with more competent material to reduce the lateral displacement of the soil, as illustrated in Figure 2.7. This method can be used to advantage if the soil contains pervious silt and sand layers which increase the consolidation rate, when the unexcavated soil bearing capacity is not sufficient along the edge of the embankment.

The disposal of the excavated material can often be a problem. Streams and rivers may be polluted if the excavated soil is not handled properly. It may be neces-

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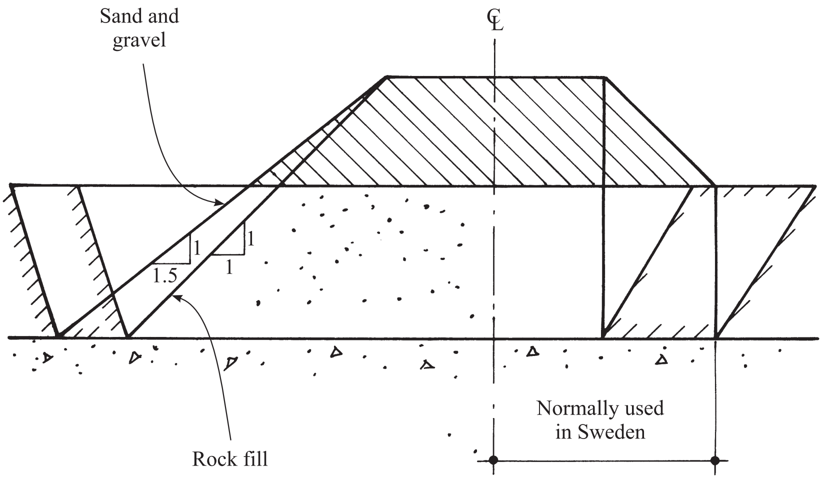

Figure 2.6. Adjustments in excavation for slope of embankment.

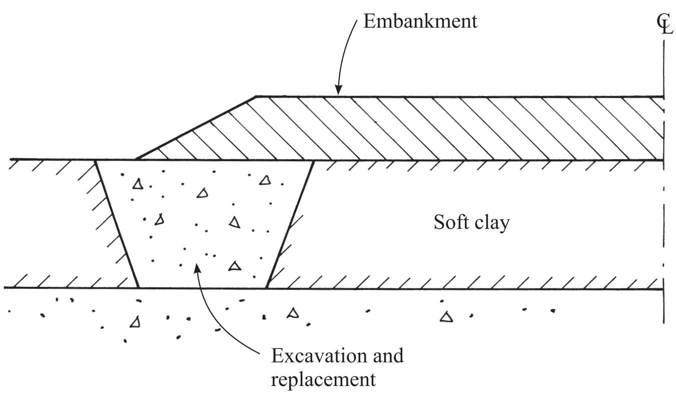

Figure 2.7. Strengthening of embankment along the edge.

sary to stabilize the excavated material with quicklime so as to make it friable with reduced water content for easy handling for disposal (Nanri & Onitsuka 1995).

2.4.3 *Displacement*

The entire embankment is forced down through the soft compressible soil. The height of the embankment is increased until the bearing capacity of an existing road embankment of the underlying soft clay is approached. The bearing capacity is then exceeded by blasting on either side of the embankment as shown in Figure 2.8. The depth and spacing of the charge depends on the depth and shear

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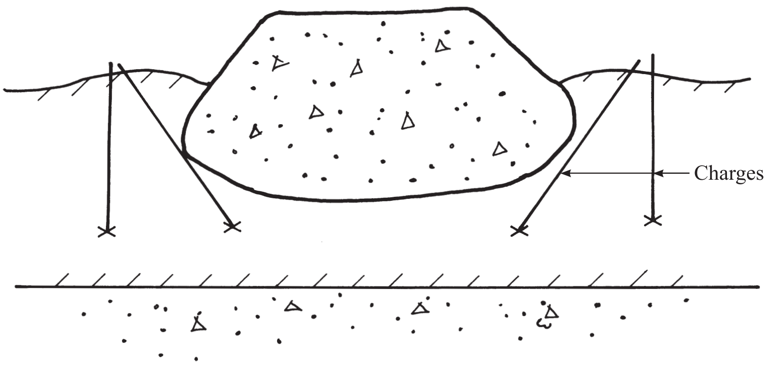

Figure 2.8. Blasting below an embankment.

strength of the compressible layers. *For proper calculation of the details, the assessment of the undrained strength of soft clay becomes imperative.*

If the thickness of the compressible layer is large (> 5 to 6 m), it will often be economical to displace the soft material by loading and causing failure of the soft material. The soft soil below the embankment moves in the direction of least resistance, forming a mud-wave in front of the fill. The displaced soft soil can be directed by changing the shape of the nose of the embankment, with a nose angle in the range of 60 to 90°. If the nose is unsymmetrical, soft soil will move to one side as illustrated (Fig. 2.9).

The mud-wave from the displaced soil acts as a pressure berm which increases stability. In some cases it may be necessary to excavate the displaced soil 10 to 15 m ahead of the fill in order to facilitate the sinking of the fill (Fig. 2.10). The direction of the displacement of the soil can be guided by the excavation. In the case described by Weberg (1962) it has been estimated that the mud-wave increased the bearing capacity by approximately 30%. Thus, reducing the bearing capacity by removing the mud-wave would be effective in minimising soil trapped below the embankment. There are instances where the displacement method has been successfully used even when the compressible layers have been 15 m deep.

Pockets of soft soil which are trapped below the fill frequently occur and become the source for additional settlements. A surcharge load is normally used to reduce the time for reconsolidation of soil below an embankment.

2.4.4 *Load reduction*

A reduction of the applied load by using light-weight materials, is often the simplest and most economical means to improve stability and to reduce settlements. The merit of light-weight substances is in their low unit weight. A wide spectrum of materials can be exploited for this purpose. Figure 2.11 illustrates the unit

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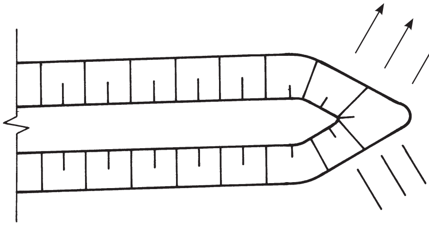
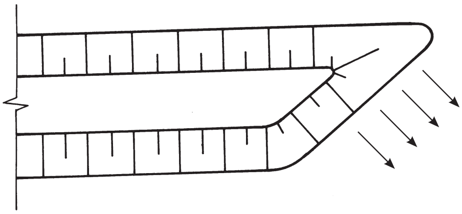
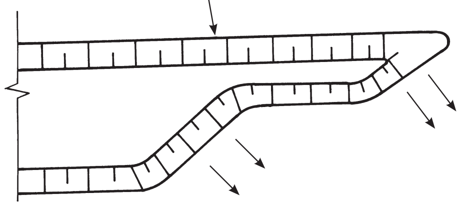


(a) Symmetrical nose



(b) Assymmetrical nose



(c) Guided embankment

Figure 2.9. Guided embankment for displacement of soft soil.

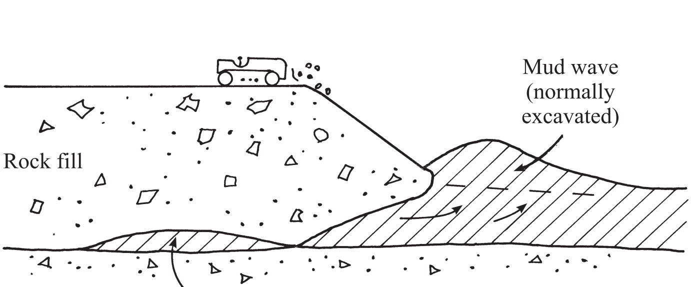


Figure 2.10. Placement of rock fill.

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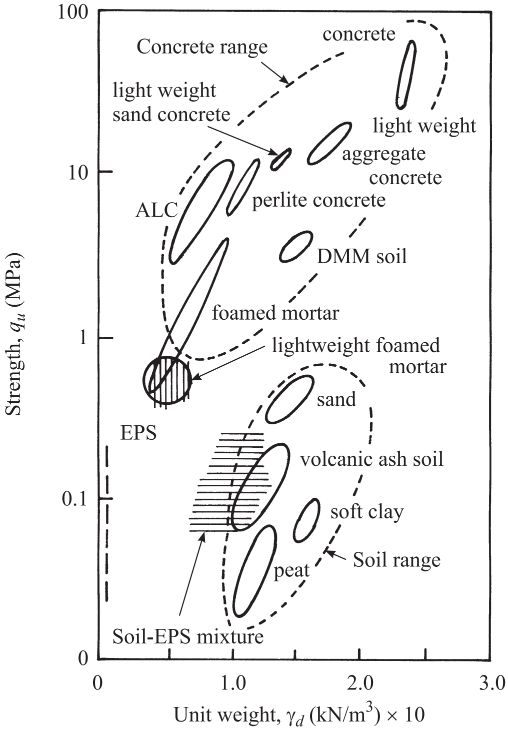

Figure 2.11. Strength versus unit weight of construction materials (Kamon & Bergado 1991).

weight of materials in relation to their strengths (Kamon & Bergado 1991). The weight of a building can be reduced by use of light-weight materials, light-weight panels, and by changing the geometry of the structure. The height of an embankment can be reduced either by decreasing the height or by use of light-weight materials, such as expanded shale, slag and ponded fly ash. Use of slags and fuel ash as embankment material has been reported by Popovics (1978) and Schwab & Pregl (1978) as early as 1978. The reduction in load was marked due to the low unit dry weight of water-cooled slags which was in the range of 11 to 12.5 kN/m^3 and that of fuel ash between 13.8 and 17 kN/m^3. Recently, Humpherey and Manion (1992) have explored the possibility of using tire chips made from waste tires, a non-biodegradable material, to realize compacted density of fills as low as 6.5 kN/m^3. It was found that these materials provided the necessary shearing resistance to ensure slope stability. The angle of shearing resistance, although decreased from very high to low at, confining pressure, was still adequate to create the internal resistance required for stability.

If it is not possible to markedly reduce the weight of a structure, and a reduction in live load cannot be effected without functional impairment, the actual load at foundation level can be reduced markedly by lowering the foundation level. By doing so the applied load takes the place of the overburden pressure released. For this option, the foundation must be redesigned. *In order to implement this ap-*

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proach it is necessary to have a realistic assessment of the yield stress of the soft clays and the compressibility characteristics within and beyond yield stress levels.

2.4.5 *Re-design as compensated foundations*

In the case of soft clays of high to very high compressibility, extending to greater depths, compensated foundations can be adopted to overcome the problem of excessive settlements (Zeevaert 1959). This implies that the weight of the building is balanced by the weight of earth removed. This is also regarded as 'floating the building', since the weight of the structure equals the weight of the soil displaced (Golder 1986). Unless the soil is a semifluid deposit, the analogy is not strictly correct. The term 'Buoyancy Rafts' (Tomlinson 1986) has also been used to imply the same principle of having foundations on highly compressible soft clay. Although this technique has gained importance and acceptance in recent times, as early as 1932 Casagrande (1932a) had categorically stated *'Lighten the load by excavation so that the settlement due to consolidation of clay will not exceed the arbitrary fixed limit'*. The principle of balancing stresses in foundation excavation is illustrated in Figure 2.12.

The load increase, for example, from a two storey building would be in the range of 20 kPa at the rate of 10 kPa per storey. The required depth of excavation to completely compensate for the weight of this building is about 1.3 m for an assumed unit weight of the excavated soil of 16 kN/m^3, which corresponds to that of a clay with a water content of about 70%. Upon removal of soil and water by such an excavation it is evident that the total vertical pressure in the soil below the excavated depth is not changed significantly, as illustrated in Figure 2.12. The economy of this type of foundation is a governing factor, especially in those cases when the cost is high owing to the foundation structure having to be supported by means of very long piles or piers to bear on a deep-seated hard stratum.

Bjerrum (1967) has described the successful use of compensated foundations for tall structures located over deep deposits of highly compressible marine clays in the Oslo area. The piled basement reduced the gross loading of 65 kN/m^2 to a net value of 23 kN/m^2. A settlement of 120 mm was noticed in 10 years. By comparison, an unpiled structure founded on a buoyancy raft, which reduced the gross loading of 55 kN/m^2 to zero, settled by only 40 mm. This settlement was mainly due to reconsolidation of the soil which had heaved during the excavation for the raft.

At first sight, the compensated raft would appear to be an attractive and economical solution to handle highly compressible soils. The problem to be faced is that of the heave of the base. Failure from bottom heave would also occur when the depth of excavation exceeds the critical depth, H_{cr}, as calculated from the relation:

$$H_{cr} = \frac{N_o c_u}{\gamma} \tag{2.3}$$

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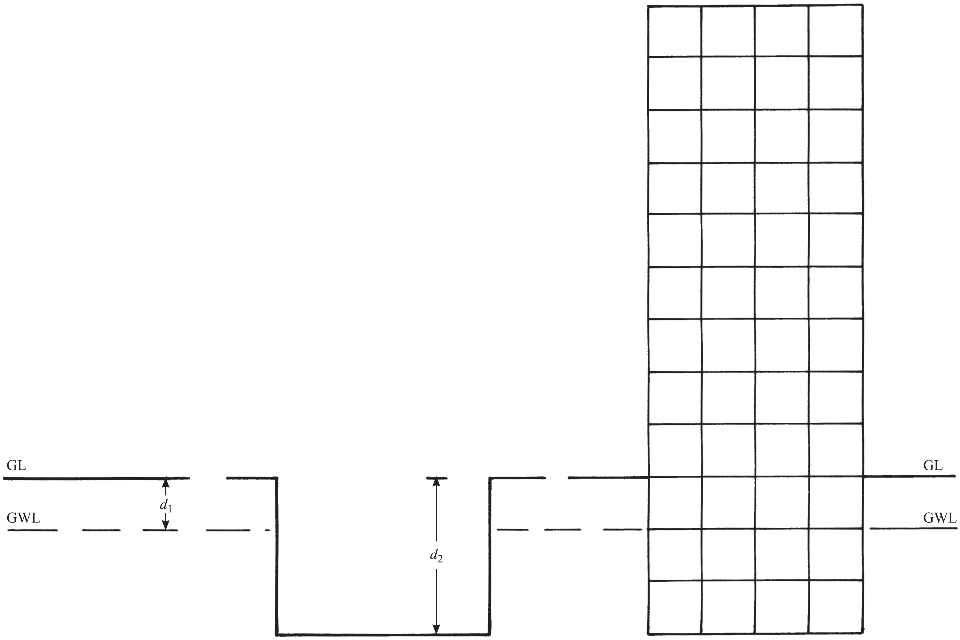

Figure 2.12. Illustration of the principle of compensated foundation.

where N_o is a stability factor, c_u is the undrained shear strength of the soil below the foundation level and γ is the unit weight of the clay. The risk of bottom heave can be reduced by casting the base slab in narrow strips at the bottom of the excavation as construction progresses. However, the weight of the base slab has to be adequate to prevent further heaving when the clay in between the strips is excavated. It would also be necessary to prevent the substructure from floating and tilting before the superstructure is constructed. Loads must be of sufficient magnitude to prevent uplift. During construction, uplift can be prevented by keeping the water table drawn down by continuous pumping. Another alternative is to have a piled raft arrangement. The rigidity of the raft in this combined foundation system involving partial floating condition merits consideration. Depending upon the settlement of the adjoining portions of the structure, in order to prevent the cracking of the raft due to differential settlements, a flexible system may have to be provided. One such example is the provision of a polymer grid reinforced granular course over the top of skin resistance piles in the soft clay foundation of a sluice way across an embankment, also resting on soft ground (Miura et al. 1994a).

In practice it is not possible to completely avoid settlement. Fluctuations in the water table affect the level of compensation. Another factor causing settlement of a compensated foundation is the reconsolidation of soils which have been re-

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bounded as a result of the overburden stress release following excavation for the substructure.

For economy in the depth of excavation for substructure construction, it is advantageous to allow a certain magnitude of net additional load. The allowable intensity of pressure is determined by the maximum total and differential settlements which can be tolerated by the structure. Despite this consideration there is an additional risk of damage to the structure by excessive settlements caused by a lower ground water table. One way to overcome this possibility is to have an excavation deeper than that actually required. *For assessment of the settlement and to decide about the partial compensated foundation, it is necessary to have an assessment of the yield stress in compression of the soft clay deposit encountered*.

As worldwide development imposes an increasing demand for land reclamation and the utilization of onshore or offshore regions, ground engineering and improvement techniques have been developed and employed extensively, especially over the last three decades. One mode of grouping suggested by Van Impe (1989) has been:

1. temporary improvement techniques: limited to the period of construction,
2. permanent soil improvement: techniques applied to improve the natural soil without the addition of materials,
3. permanent soil improvement with the addition of materials.

Further, according to the methods employed, grouping is done as below (Hausmann 1990):

1. hydraulic modification,
2. mechanical modification,
3. in-situ deep mixing and grouting,
4. modification by inclusion as in the case of stone columns, soil nailing and other forms of soil reinforcement.

There are bound to be overlaps of many methods and groups cited above. For example, drainage methods can either be regarded as temporary or a permanent method when the ground is improved by preloading techniques.

2.5 DEWATERING AND DRAINAGE TECHNIQUES

Among the more common problems in construction work, there often arises the necessity to handle subsurface water encountered during construction, together with its disposal after construction, so that the finished building is not damaged or its usefulness impaired by ground water fluctuations.

2.5.1 *Water table lowering*

For temporary excavations an increase in effective stress can be effected by lowering the water table. This would in turn enhance the shear strength and hence the

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bearing capacity. Open drainage or interceptor ditches can only be an expedient and relatively inexpensive method of lowering the water table, provided the soils are pervious. Where fine-grained soils are to be dewatered it is necessary to provide a sand filter around the well point. This is a closed end pipe with perforations along its lower end. The installation of multistage well points (Fig. 2.13) would also help to lower the ground water table to greater depths. Unless there are compatible changes in the state of the soil due to increases in effective stress the effect of de-watering are not permanent. Generally, dewatering in fine-grained soils becomes synonymous with forced consolidation induced by preloading or electro-osmosis.

2.5.2 *Electro-osmotic drainage*

Under the influence of an applied electric field, water will migrate through porous media. As discussed earlier in Section 2.2.2 clay-water interactions result in the formation of a 'Diffuse Double Layer'. Applying an electrical potential to such a system causes the hydrated positive ions to move towards the negative electrode simultaneously dragging the water held in capillary pores. The pore water flows towards the cathode. By continuous pumping of water collected at the cathode, the ground water table can be temporarily lowered, with a consequent increase in effective stress, thus imparting the required shear strength to handle practical construction problems.

Mitchell (1970) and Johnson (1978) discuss and summarize developments in the theory of electro-osmosis and its relevance to practical applications. In an analogy to the hydraulic gradient, i_h, the electrical gradient i_e is considered to be

$$i_e = \frac{E}{L} \tag{2.4}$$

where E is potential difference across the length L. The coefficient of electro-osmotic permeability, k_e, as a function of the drainage parameters, is:

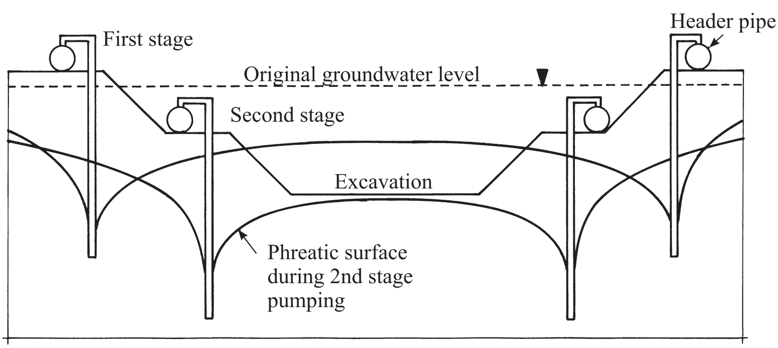

Figure 2.13. Multistage well-point system.

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$$k_e = \left(\frac{Q}{A}\right)\left(\frac{1}{i_e}\right) = \left(\frac{\text{cm}^3}{\text{sec} \times \text{cm}^2}\right)\left(\frac{\text{cm}}{\text{volt}}\right) = \left(\frac{\text{cm}^2}{\text{sec} \times \text{volt}}\right) \tag{2.5}$$

Because the hydraulic coefficient of permeability, k_h, is measured in cm/sec it is more convenient to define k_e also in cm/sec for a constant potential gradient of one volt/cm.

For a bundle of N straight capillaries of cross-sectional area 'a' in the total cross-sectional area A, with a void ratio e, the quantity of flow Q_h is:

$$Q_h = Nq_h = \frac{A}{a}\left(\frac{e^3}{1+e}\right)C_h\, i_h\, a^2 = \left(a\,\frac{e^3}{1+e}\,C_h\right) i_h\, A = k_h\, i_h\, A \tag{2.6}$$

since flow in the capillary, q_h, from consideration of Poiseuille's law,

$$q_h = \frac{\pi r^4}{8\eta}\gamma_w\, i_h = \frac{\gamma_w}{8\pi\eta} i_h a^2 = C_h\, i_h\, a^2$$

governs laminar flow. In this relation r is the radius of the capillary, γ and η are the density and viscosity of the pore fluid. In electro-osmotic flow, Q_e, the relation with the other flow parameters is:

$$Q_e = Nq_e = \frac{A}{a}\left(\frac{e^3}{1+e}\right)C_e\, i_e\, a = \left(\frac{e^3}{1+e}\,C_e\right) i_e\, A = k_e\, i_e\, A \tag{2.7}$$

In the above relation k_e does not involve 'a' being the size of the capillaries. Hence electro-osmotic permeability is practically independent of the soil pore size. In other words k_e is nearly of the same magnitude for sands, silts and clays, provided that the electro-osmotic potential is about the same for most of the mineral matter in the soil. For all practical purposes, for most saturated soils a coefficient of electro-osmotic permeability of 5×10^{-5} cm/sec per volt/cm can be used for dewatering and consolidating silts and clays. The relative efficiency of electro-osmosis can be better appreciated by expressing it in terms of the ratio between k_e and k_h, referred to as the coefficient of electro-osmotic effectiveness. It increases with the decrease in k_h values. For example consider a natural clay whose k_h is 10×10^{-9} cm/sec.

Since $i_h\, k_h = i_e\, k_e$

$$i_h = \frac{k_e}{k_h} i_e = \frac{5\times10^{-5}}{10\times10^{-9}} i_c = 5000\, i_c \tag{2.8}$$

For assessment of the relative efficacy of electro-osmotic dewatering, we must estimate the hydraulic conductivity of the soft clay. Using electro-osmotic drainage is equivalent to increasing the gradient by 5000 times. In Table 2.2 the data of k_e values of a number of soils along with the efficiency factor, F, due to electro-

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Table 2.2. Efficiency factors due to electro-osmotic drainage.

Soil type	Water content	k_e (10^{-5} cm/sec) cm/volt	$F = k_e/k_h$
London clay	52.3	5.8	5800
Blue Boston clay	50.8	5.1	5100
Commercial kaolin	67.7	5.7	570
Clayey loam	31.7	5.0	50
Rock flour	27.2	4.5	450
Ca-bentonite	170	2.0	2000
Na-bentonite	2000	12.0	12000
Mica powder	49.7	6.9	1.25
Fine sand	26.0	4.1	0.4
Quartz powder	23.5	4.3	0.4

Note: Water content data and k_e values are from Casagrande (1952) and k_h values in estimation of k_e/k_h values are from Mitchell (1993).

osmosis has been indicated. It can be seen that the electro-osmotic method is not suitable for fine sand and silts.

Another distinct feature of electro-osmotic drainage is flexibility in controlling the direction of flow by suitably placing the cathode in relation to the anode. Mitchell (1970) states that electro-osmosis can be used to

1. reduce friction during pile driving (pile as cathode),
2. reduce the negative skin friction (pile as cathode),
3. increase the friction after driving by consolidation of soil around the pile (make the pile an anode), and
4. aid in pulling sheet piles (make pile a cathode).

It is also possible to increase stability of a slope by reversing the direction of flow by suitably placing the anode and cathode (Fig. 2.14).

The stabilizing effect due to strengthening by electro-osmosis can be enhanced by appropriate choice of the electrode material (e.g., aluminum instead of steel) or by the addition of sodium silicate or calcium chloride at the anode. This results in electro-chemical hardening or electro-grouting in addition to effecting drainage and consolidation.

2.6 PRECOMPRESSION OF SOFT CLAYS

Precompression by preloading increases bearing capacity and reduces compressibility of soft ground. It is achieved by placing surcharge loads on the soft clay, in excess of those likely to be imposed by the structure, prior to its construction. It is essentially a method of pre-empting potentially damaging settlements of a structure on soft soil. It is particularly well suited when there is sufficient time available. The surcharge generally consists of an earth fill, removed after compression

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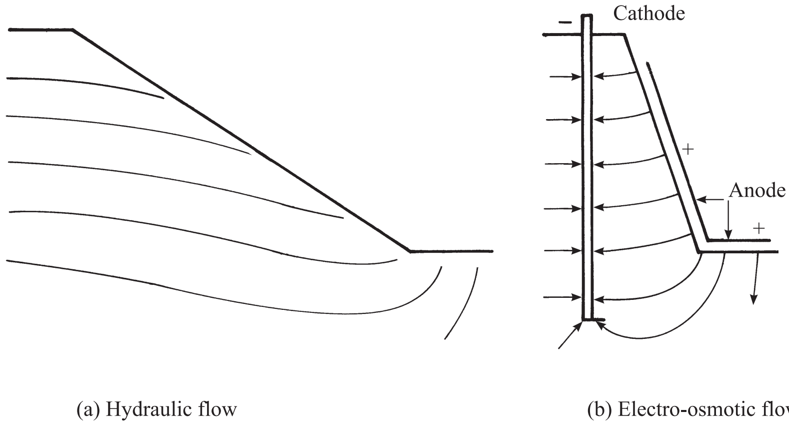

Figure 2.14. Cathode and anode deposition to control direction of water flow.

is achieved. There are situations where the preloading is left in place, as in the pavement-supporting embankments. Preloading is also used in intermediate stage of construction, as is sometimes necessary for liquid storage tanks.

Although earth fills are the most commonly used type of preload, any system that drains pore water resulting in compression is acceptable. Water in tanks, vacuum preloading, and even anchor and jacking systems have been used for precompression. The treatise by Stamatopoulos & Kotzias (1985) provides good background information on this method of ground improvement.

The process of consolidation can be speeded up using vertical drains. Only relatively impermeable soil with C_v of the order 3×10^{-7} m^2/s may benefit from vertical drains, according to Rowe (1968). These drains are particularly effective where a clay deposit contains many thin horizontal micro-layers of sand or silt lenses. The major beneficial effects of preloading and vertical drains are illustrated in Figures 2.15a and b. Obviously vertical drains speed up the settlement process but do not reduce the magnitude under a given load.

In early attempts to provide drainage, the sand drains provided consisted simply of boreholes filled with sand. By using a casing, filling the casing with water before adding the sand, and then withdrawing the casing, necking can be reduced (Datye 1982). Common vertical drain patterns are shown in Figure 2.16. A wide sand or gravel drain in fine-grained soil not only enabled rapid consolidation but also provided vertical compressive reinforcement. The higher the reinforcing effect, the lower would be the surcharge-induced consolidation in the soft substratum. Hence too great a diameter may even defeat the very purpose of preloading.

With the advent of many types of prefabricated strip drains and their ready availability, coupled with standardized installation procedures and a high rate of installation to depths greater than the 30 m normally possible by sand drains, these drains have an edge over conventional sand drains. The first cardboard strip

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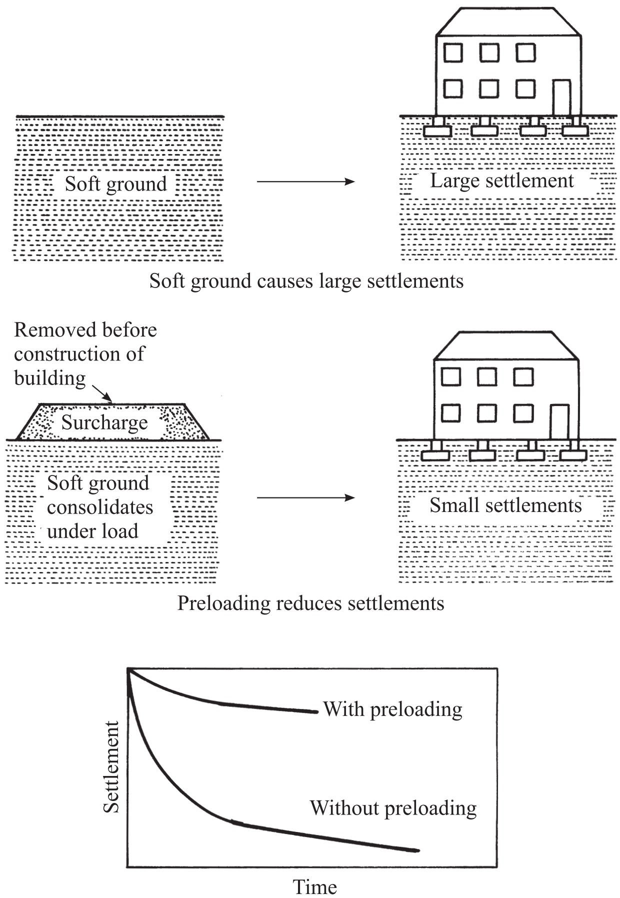

Figure 2.15. a) Beneficial aspects of preloading.

drain with internal ducts was developed by Kjellman (1948) as early as 1948. This type was later superseded by the thin fluted PVC drain. Today more than 50 types with varied degrees of composite construction are commercially available. In selecting an appropriate drain for a job, the following properties of the drain should be examined.

1. *Hydraulic characteristics*
   – discharge capacity
   – permeability of the filter
2. *Mechanical properties*
   – tensile strength of the core
   – tensile strength of the filter
   – buckling
   – durability

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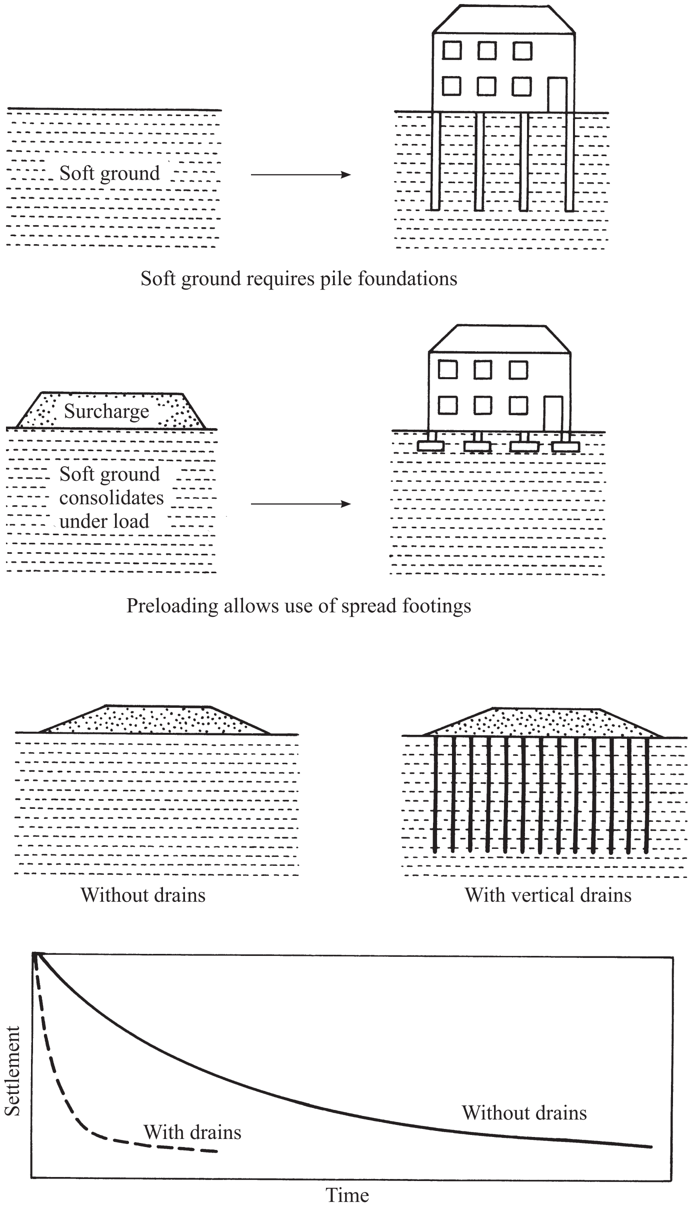

Figure 2.15. b) Beneficial aspects of preloading.

A drain stitcher can be coupled to the hydraulic system of an excavator or wheeled carrier for installation depths up to 12 to 15 m. A special power pack and pile driving rig as a carrier will be required for depths exceeding 20 m. Installation rates will vary between 0.5 to 1.3 m/s depending on the soil and site condi-

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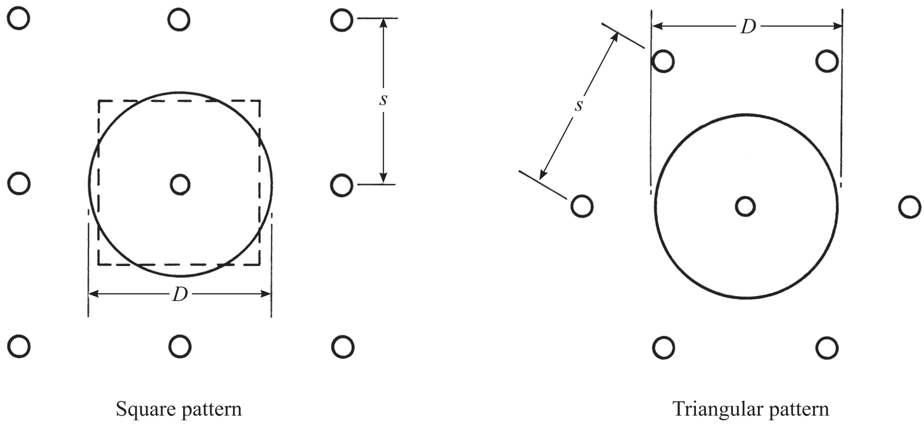

Figure 2.16. Vertical drain patterns.

tions. Specially designed steel mandrils travel through the mast enclosing the drain and guide this into the soil. The band drain is fed through the mandril from a storage reel mounted on the mast. The drains are installed in different patterns (Fig. 2.16) to meet the time requirements. After installation of the drains, the area is preloaded with an earth fill exerting a pressure equivalent to the total load of the contemplated structure. It is important to consider the level of loading due to the structure in relation to the yield stress of the clay due to cementation. If the yield stress has been underestimated due to sample disturbance, then the level of preloading may be within the actual yield stress, thereby rendering the preloading ineffective. On the otherhand, the disturbance caused due to installation of sand drains or band drains would cause greater settlement due to remoulding effects compared to settlements of the untreated ground (Miura & Madhav 1994). For loading of the structure higher than the yield stress of the soft clay due to natural cementation, the preloading level must also be higher than the yield stress. Beyond the yield stress level, the preloading of the soil must be done in small increments to avert any possible bearing capacity failure. The above considerations necessitate the *assessment of yield stress in compression of the soft clay*. For detailed treatments of various aspects of preloading and subsequent consolidation, the reader is referred to the coverage by Hausmann (1990).

Apart from accelerating consolidation by geo-drains, it is also possible to improve stability in soft clays by increasing the effective stress at the base (Fig. 2.17). Further, deep excavations in soft clays can cause stability problems. With the installation of a system of vertical prefabricated drains, consolidation of the soil at the bottom of excavation can be accelerated, thereby ensuring overall stability (Fig. 2.17). *For close monitoring of the effectiveness of the preloading in enhancing the strength of soft clay, appropriate relationships between the change in water content and shear strength as the preload becomes effective, have to be developed.*

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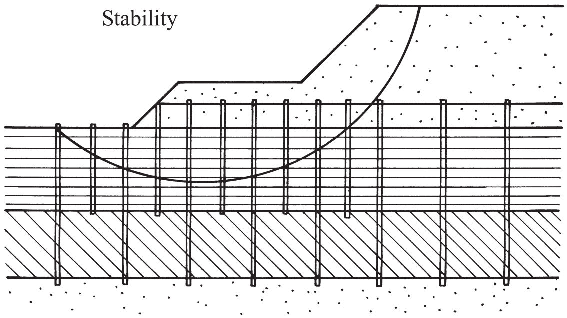
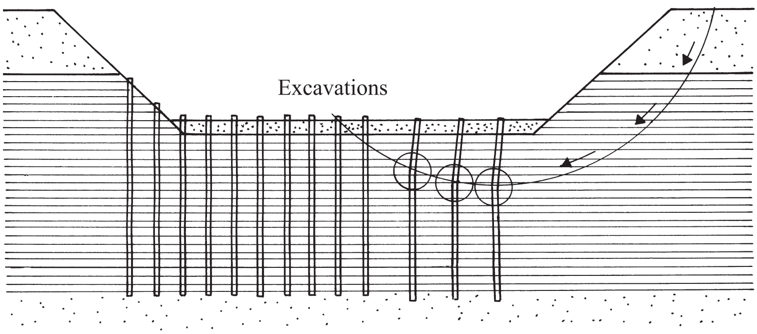


(a)



(b)

Figure 2.17. Vertical drains to enhance stability of slope and bottom of excavation.

2.6.1 *Vacuum preloading*

The vacuum preloading method, as an effective method of improving soft soil conditions, was introduced by Kjellman (1952) as early as 1952. In order to achieve best results this method is applied in combination with geo-drains, which are installed before vacuum preloading is done. An airtight plastic sheet is buried in the surrounding separation walls. Water and air can be drawn out from the sand drains through the system of perforated pipes by a pump (see Fig. 2.18). The pressure difference between above and below the separation surface is the surcharge pressure which is referred to as the degree of vacuum. When pore water and air are pumped and withdrawn from the ground, a difference in pressure is formed at the separation surface which induces compression of the clay.

The increase in effective stress tends towards isotropic with the compressive lateral deformations. Since there is no shear failure, preloading can be applied at a rapid rate. It has been observed that the vacuum preloading system creates a more even pattern of stress throughout the region of soil being improved. Efficiency of vacuum preloading has been indicated for three sites in Table 2.3.

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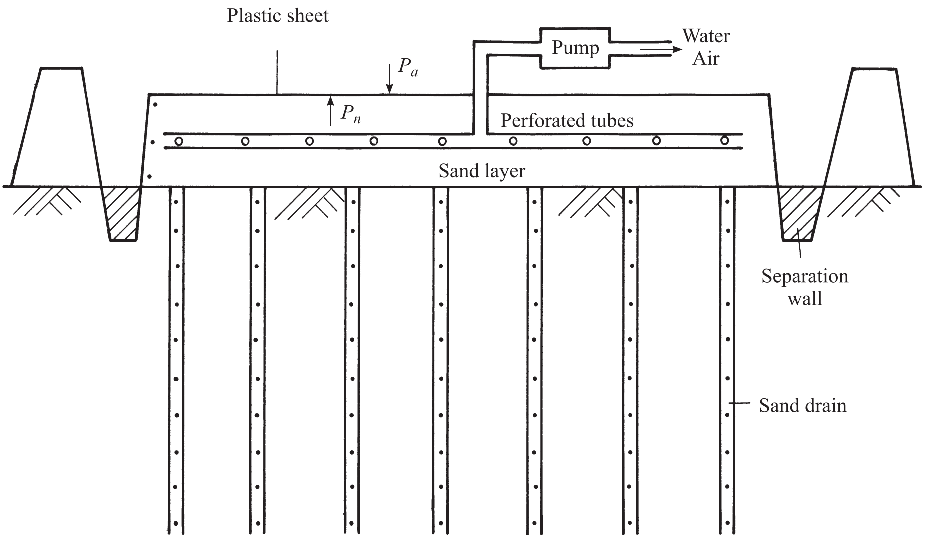

Figure 2.18. Layout for vacuum preloading with sand drain.

Table 2.3. Efficiency of vacuum-preloading.

Project	Tanjin New Harbour, China	Northeast New Railway, Japan	Factory in Lianyungang City, China
Soil type	Silty clay	Peat and Silt	Marine clay
Water content (%)	55	580-860	69-85
Void ratio	1.4	–	1.62-2.36
Natural bulk density (g/cc)	1.73	1.05	1.54-1.61
Initial shear strength (kPa)	16.7-20.6	3.4-5.9	5.7-19.6
Area of involvement (m^2)	1250	1950	4000
Thickness (m)	16	13	10
Degree of vacuum (mm Hg)	600	700	650
Increase in strength kPa	131-190	186-190	170-440
Increase in bearing capacity (%)	300	200-300	250
Estimated settlement (mm)	811	2040	1000
Measured settlement (mm)	565	1490	700
Reduction in settlement (%)	69.5	73	70

2.6.2 *Preloading using inflatable tubes and vacuum*

Another innovative way of preloading, suggested by Leong (1977), is to use vertical rubber tubes inserted at identified locations and inflating them for development of lateral pressures on the soft clay. A detailed study on the feasibility of this method was made by Kabir et al. (1988). Figure 2.19 details the location of the inflatable tubes,

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drains and sand blanket between the pressurizing rubber tubes. Over the sand blanket a geomembrane has to be created. Vacuum can be applied and sustained by an industrial grade water ring vacuum pump. By servo-controlled compressor units, constant water pressure can be maintained in the inflatable tubes using air-water pressure vessels. The main purpose of the vacuum is to provide a counterweight against the upward thrust due to upward bulging of the soil when compressed by the inflatable tubes. The spacing of the tubes will be that required for generation of uniform pore water pressure throughout the soil mass. After consolidation of the clay, the holes left by the inflatable tubes can be filled up by granular material. This will offer additional stability and drainage paths. To explore the full potential of the above method of preloading, field tests must still be conducted and performance monitored.

2.6.3 *Pre-compression by electro-osmosis*

In electo-osmotic dewatering, although the water collected at the cathode is continuously pumped out, due to continuous availability of water at the anode there would not be any reduction in the water content of the clay. Consolidation would result if water is removed at the cathode and not replenished at the anode. The net result is reduction in the moisture content, with a corresponding increase in strength and reduced compressibility. This process is equivalent to consolidation, often called electrokinetic consolidation.

2.7 DYNAMIC CONSOLIDATION

This is one of the methods of mechanical ground modification, used to increase density by use of external energy. The other methods are roller compaction, vibro-flotation and vibro-replacement. These methods cannot be employed for den-

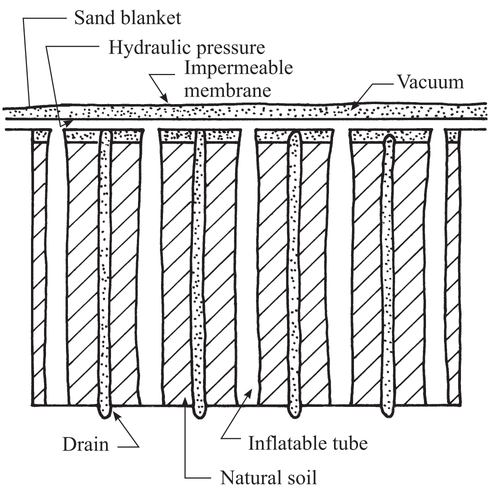

Figure 2.19. Schematic details of preloading by inflatable tubes with sand drain.

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sification of high water content soft clays. Thus, discussion in this treatise is only about dynamic consolidation, also called heavy tamping.

Dynamic consolidation is carried out by repeatedly impacting the soft ground surface by dropping a heavy weight (Pounder), as heavy as 200 tons, from great heights up to 40 m. This is done by a heavy duty crane, at the rate of one blow every 1 to 3 min. Usually the blows are concentrated at specific locations, the distances between the centers of tamping ranging from 4 to 20 m, set out in a grid pattern. The energy per blow is chosen to maximize the zone of influence.

2.7.1 *Principle*

Heavy tamping causes longitudinal waves of high velocity, up to 3 km/sec, along with surface and transverse waves (Fig. 2.20). Longitudinal waves are primarily responsible for the rupture of the skeletal structure and breakup at the surface of the soil layer. Densification of deeper layers is aided by the transverse waves. The permeability of the soil in the upper zone increases markedly thereby enabling fast dissipation of pore water pressure.

The densification of soft clay by heavy tamping is attributed to (Menard & Broise 1975):

1. the compressibility of saturated soil due to the presence of microbubbles,
2. the gradual transition to liquefaction under repeated impacts,
3. the rapid dissipation of pore pressure due to high permeability after fissuring of soil, and
4. thixotropic recovery.

In fully saturated fine-grained soil, this technique is effective provided the formation is varied and contains fairly continuous sand partings, which help excess pore water pressure dissipation. Figure 2.21 illustrates the changes in the soil state after a single pass. Curve 1 shows the energy applied to the soil by a series of impacts

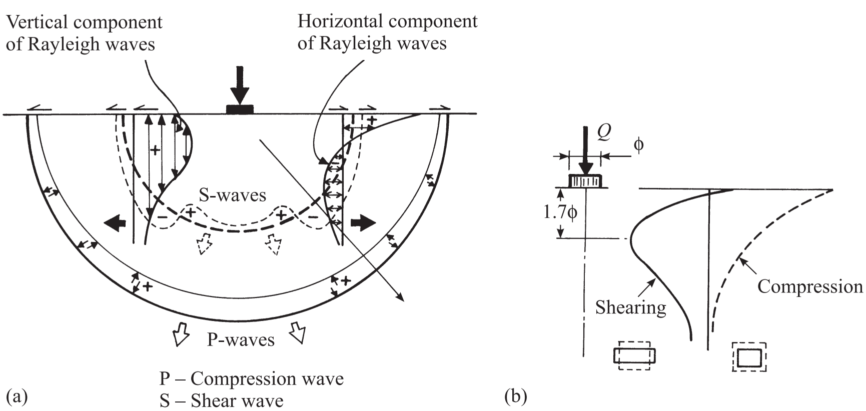

Figure 2.20. Wave propagation during heavy tamping of the ground.

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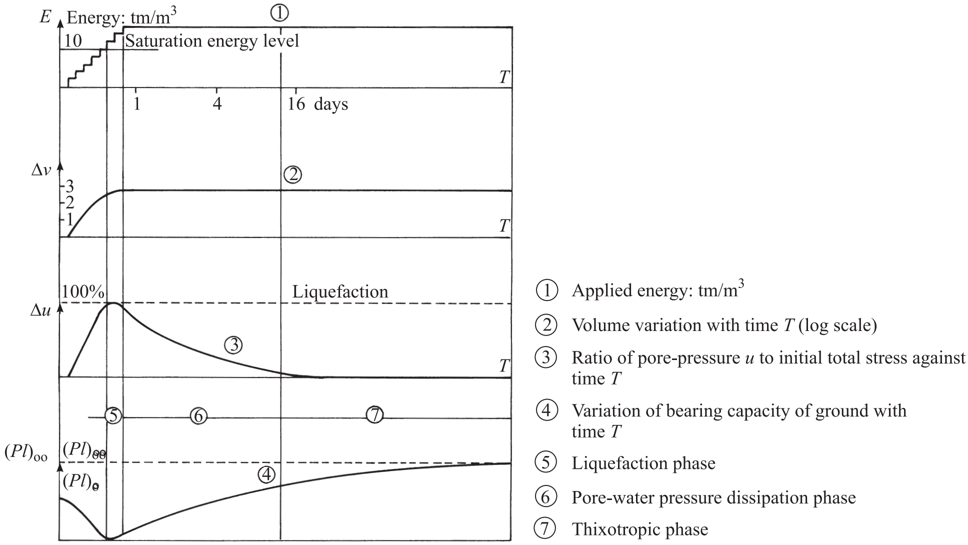

Figure 2.21. Change in soil state with time during the consolidation phase after a single pass (Menard & Broise 1975).

at the same spot. Curve 2 depicts the corresponding volume variation of the soil. Curve 3 illustrates the resulting porewater pressure in relation to the liquefaction pressure and curve 4 shows increase in bearing capacity with time. Figure 2.22 relates to the same parameters as in Figure 2.21, but for a series of passes. The imprints, which are the craters formed by the weight at the tamping locations, are finally ironed out by use of additional material.

2.7.2 *Analysis*

The depth of influence as a function of impact energy for dynamic consolidation has been analyzed by Mitchell (1981) (see Fig. 2.23) and Mayne et al. (1984). The treatment depths vary with initial strength, soil type and energy input. A simple rule of thumb suggests that the depth D in metres, to which heavy tamping is effective can be estimated conservatively by the relation

$$D = 0.5\left(wh\right)^{0.5} \tag{2.9}$$

where w, is the mass of the falling weight in metric tons and h is the height of fall in metres. According to Mayne et al. (1984) the degree of soil improvement peaks at a 'critical depth' which is roughly one half of the maximum depth of influence D. Even during the initial stages of development of the dynamic consolidation technique, attention was paid to general design guidelines (Gambin 1984). The main points to be considered are (1) the applicability of the technique, (2) design parameters, and (3) effects on the environment.

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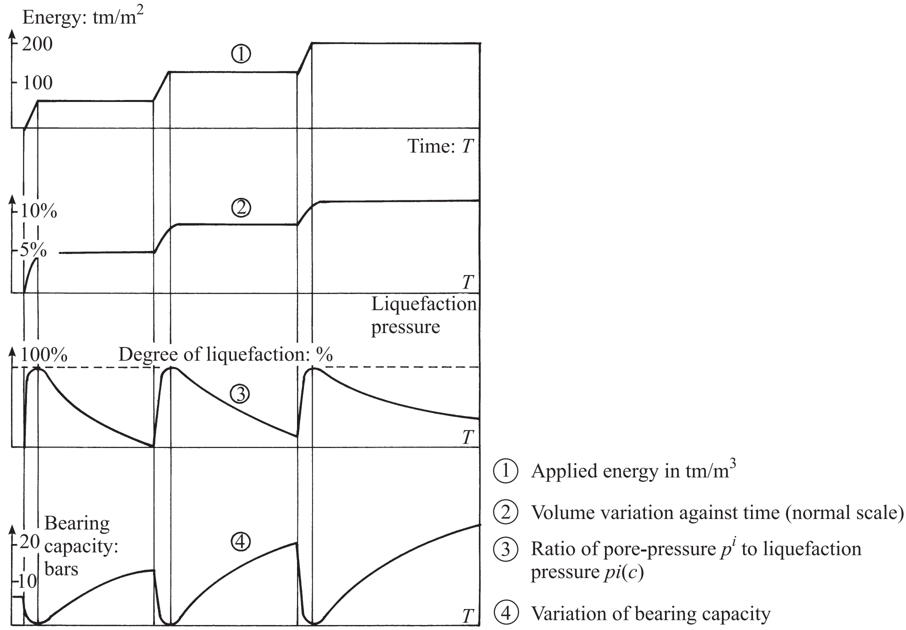

Figure 2.22. Variation of soil state subjected to a number of passes (Menard & Broise 1975).

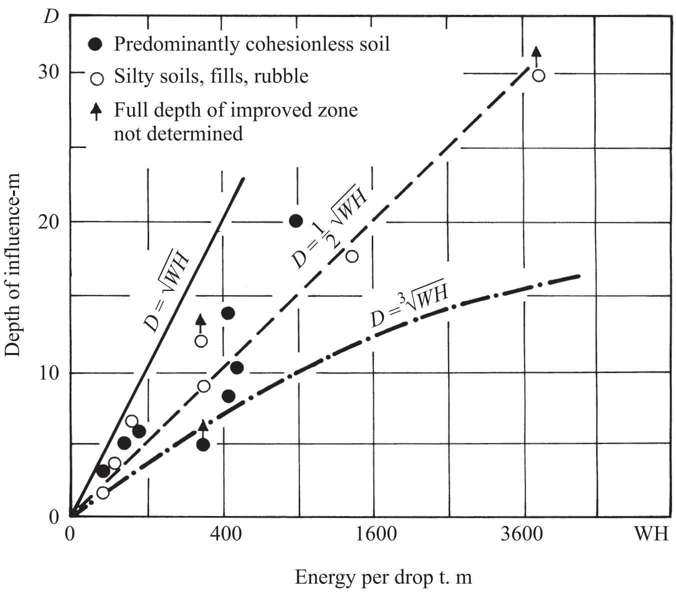

Figure 2.23. Depth of influence of surface tampings with impact energy (Mitchell 1981).

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Experience indicates that the method is less effective when the percentage of clay content is larger than 15%, as permeability would be too low to allow for rapid dissipation of excess pore water pressures (Van Impe & Madhav 1995) and hence the applicability of the method is limited. The spacing of the impact points, the number of blows, and the number of passes are additional variables available to the engineer at the site. The main design parameters to encompass all the above parameters are (1) the energy per impact (2) the impact grid centres, and (3) the total energy per unit surface area. Like any other method of ground treatment involving vibrations, the effects on the environment (apart from being a nuisance to the neighbourhood) and on adjoining structures merit consideration. An additional concern is whether any damage may occur to structures located beyond the area of densification.

2.8 SOIL REINFORCEMENT

The concept of strengthening soil by reinforcement is not new. It can be observed that in nature the stability of trees is primarily due to the physical interaction between soil and roots. Roots are much stronger than the soil in its particulate state. Interfacial shearing resistance between the soil and the roots converts the soil-root system into a coherent gravity mass. The bulb of entangled roots of an uprooted tree is a testimony to this mode of interaction. Two modes of earth reinforcement that can be visualized are as follows.

1. *Compression elements*: Columnar inclusions such as granular piles and stone columns, although weak in tension, have both compressive strength and stiffness which are far higher than the surrounding soil. Hence they impart overall strength and stiffness to the composite ground.
2. *Tension elements*: Soils are weak in tension. Hence the inclusion of reinforcements such as bars, strips, grids and nets which possess far higher tensile strength would provide an overall integrity to the system. The interaction between the soil and the reinforcement are through interfacial shearing resistance. This restricts and practically arrests lateral deformation. This mode of construction remained as a craft until 1966, when Henri Vidal (Vidal 1966) quantified and patented it with reference to its usefulness to practical problems.

2.8.1 *Granular piles and stone columns*

Granular piles including stone columns as ground improvement methods have been used for over three decades. These compression elements introduced into the ground, although they cannot resist tensile stresses, possess high compressive strength and stiffness. Thus such columnar inclusions carry a substantially greater proportion of applied loads with significantly smaller deformation, compared to the in-situ soft clay. They not only serve the primary functions of reinforcement and drainage but also enhance the bearing capacity and reduce settlement of the

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composite ground. Also, as a consequence of the installation processes, the lateral stresses in the original ground conditions around the inclusions tend to be higher than the at rest values. Bergado et al. (1991) present a comprehensive state-of-the-art report on this versatile ground improvement method.

Installation: A cylindrical hole is made by means of a vibrofloat tool. In recent years, to ensure continuity in the body of granular material, particularly in soft soil, a steel casing is used to form the hole. The granular material is filled into the hole formed. Filling and densification is done by various methods depending upon their proven applicability and availability of equipment. The methods explored are (Bergado & Miura 1995):

1. Vibro-compaction method using a vibrofloat.
2. Vibro replacement – wet or dry method.
3. Vibro-composer using a casing pipe.
4. Ramming by dropping 15 to 20 kN weight from heights of 1 to 1.5 m.

Of the above methods the vibro-composer method, which was originally developed by Murayama (1958) as early as 1958, is suitable for soft clay with a high ground water table. The method, in brief, consists of driving a casing pipe to the desired depth by a vibrator at the top. A sand charge is introduced into the pipe, the pipe then being withdrawn incrementally while compressed air is blown down inside the casing to hold the sand in place. The pipe is vibrated down to compact the sand pile and enlarge its diameter. The process is repeated till the casing is completely withdrawn. The resulting pile is usually 60 to 80 cms diameter. These inclusions normally penetrate the soft layer fully, if its thickness is in the range of 12 to 15 m, and end on a bearing stratum; or they penetrate partially and act as a floating pile in a deformable medium. It has been elucidated (Madhav et al. 1996) that as confinement increases with depth, granular piles develop nonhomogeneity as the modulus of deformation increases with depth.

Another distinctly different mode of installation in uncased boreholes is to build up the stone column above ground within an enclosure of strips and spiral winding of fibre such as coir or jute (Datye & Nagaraju 1981). The whole assembly is then lowered into the bore hole. This method of installation is effective only when density is close to optimal value, which can only be attained by prepackaging.

Analysis and design: What is formed by this mode of ground improvement is a composite ground. For a large group of inclusions the response of the composite ground is predominantly one-dimensional, except near the edges of the load or the composite ground. The loads are carried essentially by these columns (see Fig. 2.24). The performance of composite ground is best investigated in terms of ultimate bearing capacity, settlement and general stability.

The tributary area of the soil, A_c surrounding each granular pile is closely approximated by an equivalent circular area. The diameter, d_e, with spacing of the columns, s_c, is related by the expression (Balaam & Brooker 1981),

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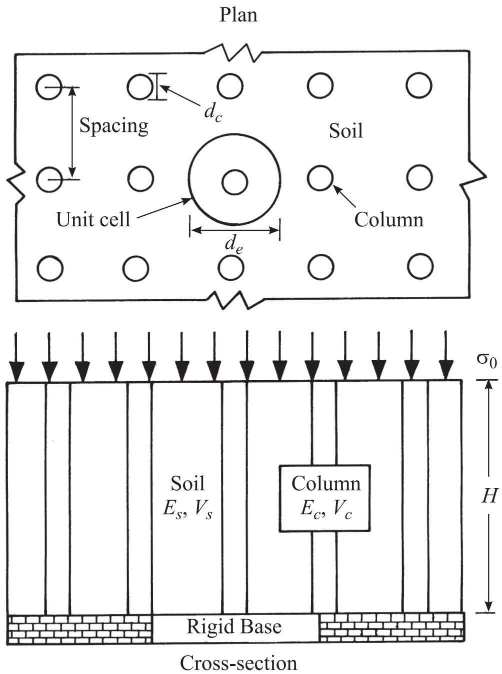

Figure 2.24. Typical column reinforced ground and column soil unit.

$$d_e = c_g\, s_c \tag{2.10}$$

where c_g is a geometry-dependent constant equal to 1.05, 1.13 and 1.29 for triangular, square and hexagonal arrangements respectively. The area replacement ratio, a_s is defined as the ratio of the granular pile area, A_s over the whole area of the equivalent cylindrical unit cell and expressed as (see Fig. 2.24)

$$a_s = \frac{A_s}{A_s + A_c} \tag{2.11}$$

This area replacement ratio can also be expressed in terms of the diameter, D and spacing, S of the columnar inclusion as:

$$a_s = c_1 \left(\frac{D}{S} \right)^2 \tag{2.12}$$

where c_1, is a constant depending upon the pattern of granular pile used; for a square pattern $c_1 = \pi/4$ and for an equilateral triangular pattern $c_1 = \pi/[2(3)^{0.5}]$.

The distribution of vertical stress within the unit cell can be expressed by a stress concentration factor n defined as

$$n = \frac{\sigma_s}{\sigma_c} \tag{2.13}$$

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where σ_s is the stress in the granular pile, and σ_c is the stress in the surrounding cohesive soil. Generally, granular piles and stone columns are founded on a rigid layer of adequate strength underneath the soft soil. If this is not feasible then they are regarded as floating columns. In either of the above cases the soft ground may be stratified.

The possible modes of failure of granular piles are (1) bulging (2) general shear and pile failure, and (3) punching (see Fig. 2.25). Generally granular piles are long enough to preclude punching failure. With the usual practice of providing a foundation bridging granular piles in the composite ground, general shear failure is averted. Hence the most probable failure of granular columns is by bulging. As pointed out by Madhav & Miura (1994), while the tendency for bulging failure is predominant, it occurs in conjunction with the pile action since the applied loads are transmitted to the surrounding soil through the resistance mobilized around the perimeter and the base of the granular pile.

Bearing capacity: The rational design of columnar inclusions such as gravel piles must take into account performance as a whole, i.e., it must consider such processes as consolidation, dilation, settlement and load sharing which occur simultaneously in any loading process (Poorooshasb et al. 1991, 1996). When the reinforced composite ground is being loaded, the columns deform laterally into the surrounding soil strata while distributing the stresses at the upper portion of the soil layer. For most granular columns whose lengths are greater than the critical length, it is recognized that bulging failure governs the load carrying capacity,

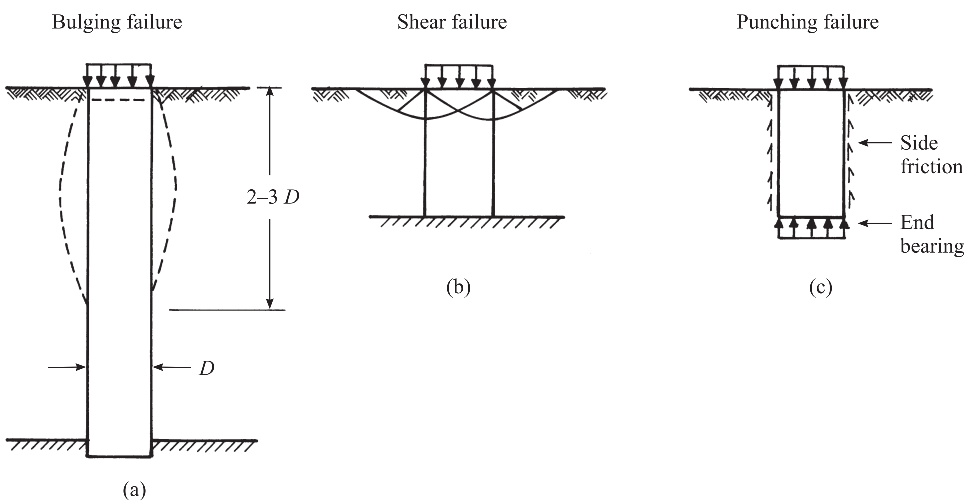

Figure 2.25. Failure mechanisms of granular pile in soft clay (Barksdale & Bachus 1983). a) Long stone column with firm or floating support, b) Short column with rigid base, c) Short floating column. *Note: Shear failure could also occur.*

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whether they bear on a stiff layer or partially into the medium stiff soil. While the tendency for bulging is predominant, it occurs in conjunction with the pile action since the applied loads are transmitted to the surrounding soil through the resistance mobilized around the perimeter and the base of the column. For a granular pile at the verge of bulging, not only a lateral confining stress but shear stress also acts (Madhav & Miura 1994).

The performance of the conventional granular column can be improved by stiffening the top portion to the extent of two to three times the diameter by any of the following methods. Some of the specific methods are: skirting the top portion (Rao & Bhandari 1979), concrete plug (Madhav 1982), rigid granular column (Barksdale & Bachus 1983), and wrapping round with a geosynthetic membrane (Alamgir 1989, Ayadat & Hanna 1991, and Madhav et al. 1994). Both the ultimate load-carrying capacity and the stiffness of the columnar inclusion are markedly enhanced.

In considering the bearing capacity and load settlement behaviour of stone column foundations, the load transfer mechanism involved is complex. It includes the increase in the rates of dissipation of excess pore water pressures and the associated gain in shear strength due to improved drainage conditions in the surrounding soil. The analysis is therefore simplified by using assumptions, for instance that all foundation loads are carried by stone columns (Thorburn 1975). Another approach has been that the stresses are assumed to be shared between them and the surrounding soil in proportion to the relative stiffness of the two materials (Aboshi et al. 1979, Goughnour & Bayuck 1979).

The bearing capacity of a single stone column or sand pile is controlled by the passive resistance of the surrounding soil and by the frictional resistance of the compacted material, and can be assessed by the relation:

$$\sigma_v = \sigma_{hs} \tan^2\left(45 + \frac{\phi'_c}{2}\right) \tag{2.14}$$

where: σ_v is the vertical stress in the column, σ_{hs} is the passive resistance of the surrounding soil, and ϕ'_c is the effective friction angle of compacted material. With a factor of safety of three the allowable vertical stress can be calculated.

Another approach to analyzing the stability of soils reinforced by a large number of columnar inclusions is by the principle of homogenization to arrive at the equivalent properties of the composite ground (Enoki et al. 1991). Madhav & Nagpure (1996) analyse the stability of embankments resting on soft soils reinforced with inclusions using the strength parameters derived from Enoki's approach. Such an analysis reveals that the increase in the factor of safety is relatively less significant for reinforcing zones beyond the toe of the embankment.

Settlement: Apart from bearing capacity, settlement considerations also dictate the overall performance. For a floating or end-bearing granular pile with constant modulus of deformation, settlement can be estimated either by the rigorous con-

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tinuum approach (Mattes & Poulos 1969) or the simpler shear layer concept (Randolph & Wroth 1978). Dividing the granular pile length into a number of layers, Hughes et al. (1975) obtain settlement as the sum of settlement of each layer calculated from the radial stress-strain properties of the clay. Balaam et al. (1977) employ a finite element approach for prediction of the load-settlement response of the granular pile. Madhav et al. (1996) advance a method of settlement analyses which incorporates a practical situation of linear increase in pile modulus with depth.

On the contrary, the deformations of the whole composite ground would be of the same order, if the average stress is realized for the load applied through a relatively stiff structural member at the interface, and stress on the column would be different from that transferred to the soil. The magnitude of the stress concentration depends on the relative stiffness of the granular pile and the surrounding soil. Then, according to the law of mixtures

$$\left(A_s + A_c\right)\sigma = \sigma_s\, A_s + \sigma_c\, A_c \tag{2.15}$$

where A_s is the cross-sectional area of the column and A_c is the area of the surrounding soil. From considerations of 'n', the stress concentration factor and a_s, the area replacement ratio in the above relation, the fraction of loading shared by the column and the soil can be assessed by the following expressions:

$$\sigma_s = \mu_s\sigma = \left\{\frac{n}{1+\left[n-1\right]a_s}\right\}\sigma$$

$$\sigma_c = \mu_c\sigma = \left(\frac{1}{1+\left[n-1\right]a_s}\right)\sigma \tag{2.16}$$

where: μ_s is the coefficient of stress concentration, μ_c is the coefficient of stress decrease, a_s is the area replacement ratio.

The consolidation settlement S_s of the unreinforced soft ground would be in accordance with the volume compressibility of the soft clay and H the thickness of the soft layer. The magnitude can be computed by the relation

$$S_s = m_v\, \sigma\, H \tag{2.17}$$

The consolidation settlement of the composite ground would be

$$S_c = m_v\, \sigma_c H = m_v\, \mu_c\, \sigma\, H \tag{2.18}$$

This results in the computation of the settlement reduction ratio R, which is the ratio of S_c to S_s as given by the value of μ_c

$$\mu_c = \left(\frac{1}{1+\left[n-1\right]a_s}\right) \tag{2.19}$$

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The higher the values of the stress concentration ratio, n, and the area replacement ratio, a_s, the greater will be the reduction in settlement.

Untill recently in the development of the appropriate methods of analysis (Priebe 1976, Aboshi et al. 1979, Balaam & Booker 1981 and others) the emphasis has been mainly to consider the geometrical parameters of the composite ground, relative stiffness of the columnar inclusion and of the soft ground around, with the tacit assumption that the settlement of the composite ground is uniform. This equal strain consideration is valid only if the superstructure load is transferred through the rigid base at the interface. In the equal strain analyses it is assumed that the horizontal section of the ground remains horizontal even after application of the load. Even as late as 1994, in the Foundation Engineering book by Hansbo (1994), considering the common contemplated usage, due to the provision of a rigid base, analysis by equal strain theory is further reinforced.

There are many practical situations, such as the transfer of load of flexible airfield and highway pavements and embankment loading on to the composite ground, in which the rigid interface considerations in the analysis are not tenable. In order to widen the use of columnar inclusions in varied situations as cited above, development of simple methods of analysis is desirable. To consider the other extreme situation to equal strain (rigid interface), this is to develop a method to analyze the deformation response of the composite ground for free strain conditions. This has recently been accomplished (Poorooshasb et al. 1996, Alamgir et al. 1996). This simple analytical approach to predict the deformation response of composite ground considers the deformation properties of the column material and the surrounding soft clay. The interaction shear stresses between the column material and the surrounding soil are considered to account for the stress transfer between the column and the soil. The solutions have been obtained by imposing compatibility between the displacements of the column and the soil system. Numerical evaluations have been made for the usual range of parameters to illustrate the influence of various parameters on the predictions made. Finite element analysis lends additional credence to the analysis.

Not withstanding the practical need to incorporate the effects of the rigidity of pile interface to assess settlement of the composite ground, Madhav & van Impe (1994) recognized the necessity of a gravel bed during the construction stage itself, and hence the need for rigorous analysis. The purpose of this layer of granular fill has been identified as (1) to provide a working platform for the machinery, (2) to level the site to increase the elevation, (3) to prevent upheaval during column installation by vibro-displacement technique, (4) to provide a facility for drainage of water, since the granular columns act as drains as well, and (5) to distribute the load from structures on to the soils and the columns to minimize the differential settlements. The analysis provided by Madhav & van Impe (1994) assumes that (1) granular fill is incompressible and can deform only by shear, (2) there is no slip at the column-soil interface. Further, the stress transfer along the column-soil interface is not considered. Subsequently, a model which incorporates the com-

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pressibility of the granular material and considers the possible slip that may occur along the column-soil interface has been developed (Alamgir 1996). The model also takes into account the nonlinearity of the material properties, and the interaction as well as stress transfer between the column and the surrounding soil along the depth. The governing equations have been solved numerically by the finite difference method. The results from the proposed method compare well with those obtained from the finite element analysis and the existing approaches for some specific cases. The predicted results have good agreement with experimental results obtained both from the laboratory and field tests. The parametric analysis made has brought out the influence of various parameters on the predicted behaviour of column-reinforced ground.

Applications: As mentioned already, improvement in the bearing capacity and overall stiffness of the composite ground can be expected by different types of columnar inclusions. This results in the reduction of settlement. Some of the modes of use of stone columns in practical problems are schematically shown in Figure 2.26. Stone columns can be regarded as piles which are laterally supported by the soft soil, being loaded by individual footings (Fig. 2.26a).

The load carrying capacity is governed by the consideration of bulging failure of the columnar inclusion. Another situation is to examine the improvement in soft ground through raft foundations (Fig. 2.26b). Such conditions prevail in the case of industrial structures, for example silos or the pillars of big bridges. Since the strains experienced by stone columns and soft surrounding soil are of the same magnitude, a stiffer reaction of the stone column may be expected. Another practical situation that one might encounter is that of stone columns supporting a tank foundation (Fig. 2.26c). Depending upon the relative stiffness of the tank bottom, the deformation of the composite ground may have to be assessed. It is possible that edge regions may experience non-symmetrical deformations. There are situations where stone columns and sand piles may experience non-symmetrical deformations. There are situations where stone columns and sand piles have been used to strengthen the base and toe of the embankments (Fig. 2.26d). The stress and deformation conditions are still less uniform in such cases due to the flexibility conditions. Of the four situations discussed here, the last one is relatively most complicated. In all the cases described above, stone columns may rest on a rigid layer of far higher strength than the overlying soft soil. If this is not the case, the analysis has to consider these as compression elements introduced as floating columns. In such cases the load carried through skin shear stresses and that transferred through the tip may have to be assessed.

In general, a realistic assessment of the undrained strength of soft clay, as well as the stress-strain-pore pressure/volume change response of soft clay and that of the columnar inclusion, is needed for analysis of the stability and settlement of different structures founded on the composite ground.

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2.8.2 *Reinforced earth*

Reinforced earth is a composite material composed of soil and reinforcing elements of adequate tensile strength. Suppression of the lateral deformation of soil through the mobilization of the interfacial shearing resistance between particulate soil and continuous tensile reinforcement under external load is the primary tenet of the reinforced earth principle proposed by Vidal in 1966 (Vidal 1966). Mohr Coulomb failure theory enables us to study the stability of particulate materials under different vertical and horizontal stresses (Fig. 2.27). Subsequently, Yang (1972) and Schlosser & Long (1973), by in-depth examination, explained the basic concept from enhanced confining pressure and anisotropic cohesion considerations.

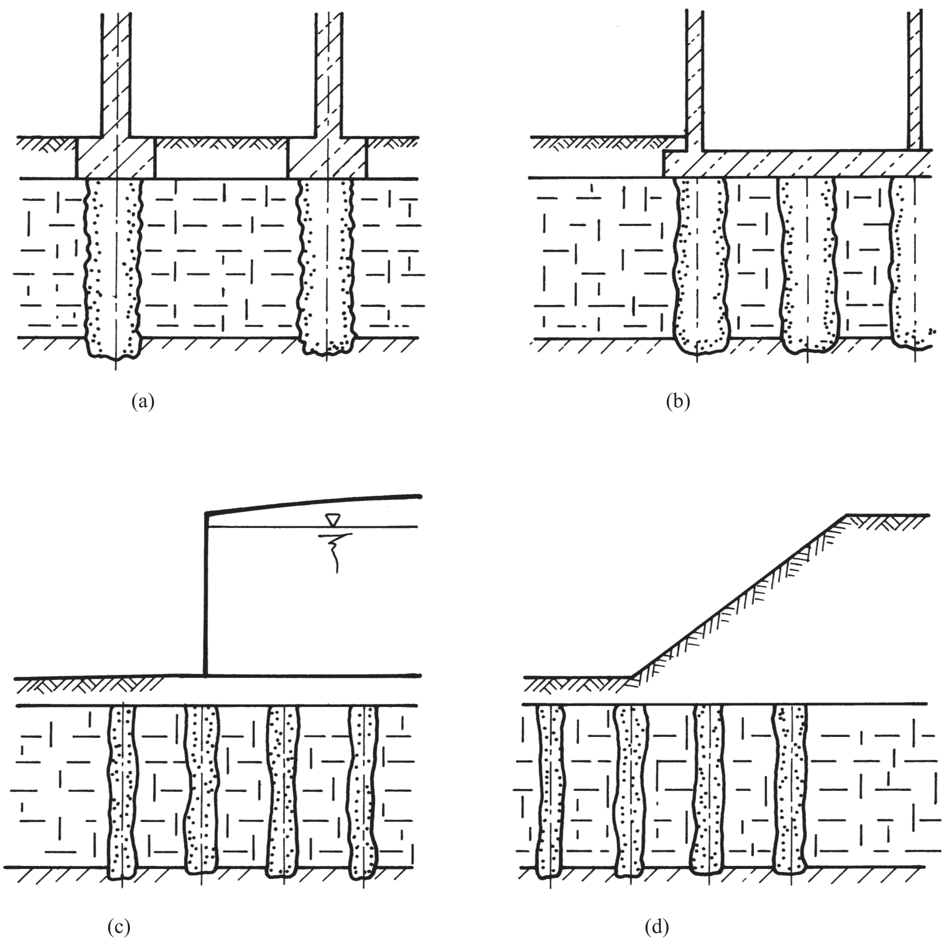

Figure 2.26. Practical situations of use of stone columns. a) Single footings, b) Raft foundation, c) Foundation of a tank, d) Foundation of a dam.

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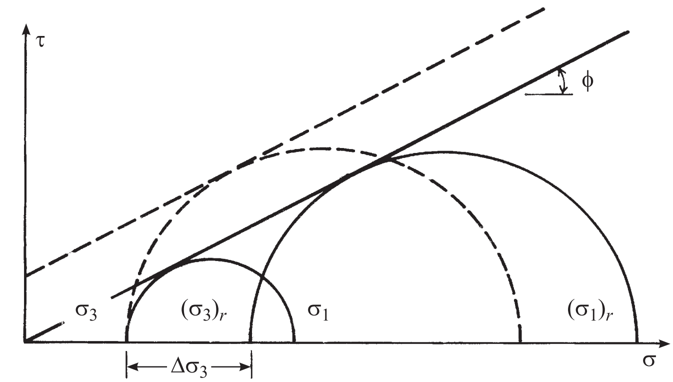
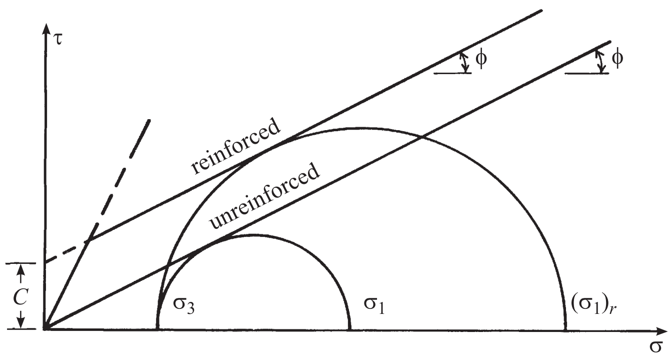


Enhanced confining pressure concept



Anisotropic cohesion concept

Figure 2.27. Concepts of strength mobilization in reinforced earth.

The enhanced confining pressure concept assumes that the vertical and horizontal planes are no longer the principal planes due to shear stresses induced between the soil and the reinforcement. The increase in the strength due to reinforcement is attributed to the confining pressure mobilized due to restraint of the soil deformation in the direction of the reinforcement. The minor principal stress within the reinforced soil sample increases with the increase in major principal stress with the consequent shift in the Mohr's circle of representation of stresses. The failure envelope is the same both for unreinforced and reinforced cases (Yang 1972) (see Fig. 2.27).

According to the anisotropic cohesion concept, at the failure state of a reinforced soil sample the major principal stress is the same as in the unreinforced

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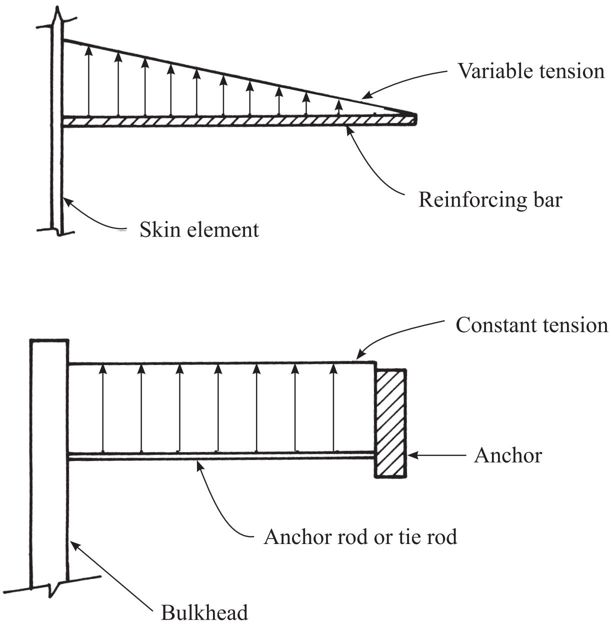

Figure 2.28. Tensile stress distribution along reinforcing bar and anchor tie rod.

state while the minor principal stress is decreased (see Fig. 2.27). Consequently, the failure envelope of the reinforced soil sample would lie above that of the unreinforced situation (Schlosser & Long 1973). Hausmann (1976) pointed out that at low confining stresses, the reinforced samples fail by slippage or loss of adherence (pull-out failure). Hence, no apparent anisotropic cohesion intercept can be noticed. Only the internal friction angle increases. However, at high confining stresses, the reinforced soil fails due to breakage of the reinforcement and has anisotropic cohesion intercept with the angle of shearing resistance being the same both for unreinforced and reinforced cases.

The mere presence of high tensile strength materials in the earth would not form reinforced earth. For example, in the anchored bulkheads the tie rod in between would experience constant tension. This is different from the reinforced earth condition, where the reinforcing elements would experience variable tension along their length (Fig. 2.28). For this to happen the reinforcing element should be in close contact with the soil. Consider an element of the soil in contact with the reinforcement (Fig. 2.29). Let the tensile force transmitted over a length dL be dT. Considering the component of normal stress acting over a width b and length dL of this element is σ. Let f be the coefficient of interfacial shearing resistance which is tan ϕ_μ (ϕ_μ is the angle of friction between the earth and the reinforcement). Then the limiting condition

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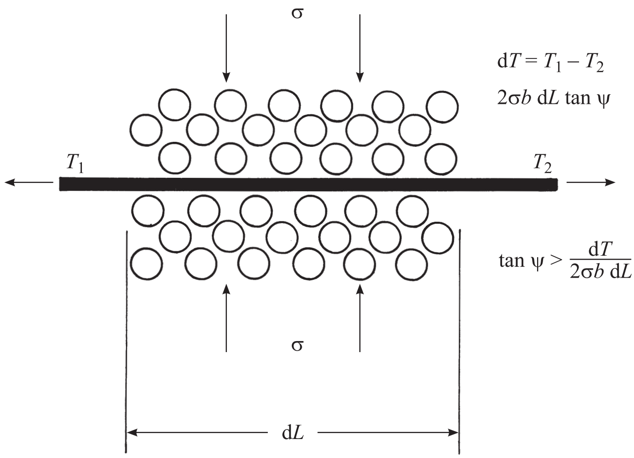

Figure 2.29. Friction between earth and reinforcement.

$$f \geq \frac{dT}{2\sigma b dL} \tag{2.20}$$

would be for inducing lateral deformation. The soil particles can be considered as sacks of earth with a thickness equal to the distance between two reinforcing layers. Due to vertical loading the soil experiences extension with the entire stress field still being compressive. The placement of a reinforcement which can withstand tensile stress is akin to providing additional compressive forces in the soil to reduce extension.

For the common case of a retaining wall with the self-weight of retained fill, the lateral stress on the retaining structure depends upon the movement of the wall. The lateral stresses, from the K_o condition, tend to be active.

$$\sigma_v = \gamma H$$

$$\sigma_h = K_o \gamma H \rightarrow K_a \gamma H$$

This tendency for lateral expansion of soil is restrained by the reinforcement, by interfacial shearing resistance resulting in tensile stresses. The extent to which restraint is provided depends upon the modulus of the reinforcement.

Strain in the reinforcement ε_r of the interfacial area a_r is

$$\varepsilon_r = \frac{K_o \sigma_v}{a_r E_r} \tag{2.21}$$

If $a_r E_r$ is high $\varepsilon_r \rightarrow 0$. On the other hand as stiffness decreases $K_o \rightarrow K_a$.

The above conditions, when satisfied by earth reinforcement, would ensure only the internal stability. It is equally important to ensure global stability of the earth structure against rotation, sliding and such other stability conditions.

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Reinforced earth retaining wall: In order to examine further the basic principles of reinforced earth, an earth-retaining structure is considered (Fig. 2.30). The three basic components of a reinforced earth-retaining structure are the facing elements, the reinforcing strips and the compacted backfill. Even without the facing elements the earth structure is stable. In order to provide an overall integrity to the structure and to prevent local erosion of the soil, semi-elliptical galvanized steel or concrete panels are used as facing elements. If the earth retained has a steep slope instead of being vertical, even grass turfing would do the job of the facing element.

For the first ten years after the introduction of the reinforced earth technique, smooth galvanized-steel strips have been used. At present a wide variety of reinforcing elements are available. They are primarily polymeric materials in the form of uniaxial and biaxial grids and membranes.

In order to ensure adequate development of friction a coarse material is used. The percentage of fines is restricted to be less than 15%. The backfill is placed in the compacted state close to the optimum moisture content. This results in a free-draining backfill with a friction factor not less than 0.3 ($\phi \geq 17^{o}$).

In reinforced earth the essential composite action of the soil and the reinforcement has to be ensured by the development of the adequate friction at the interface without any slip or tie failure. In the design, it is intended to arrive at spacing of strips both horizontal, S_x, and vertical, S_y. Considering one unit of the facing element, the horizontal force to be resisted is

$$F_H = K_a\, \sigma_v\, S_x\, S_y \tag{2.22}$$

This is resisted by the interfacial shearing resistance of magnitude

$$\tau = 2\, \sigma_v\, w\, l_e \tan \phi_\mu \tag{2.23}$$

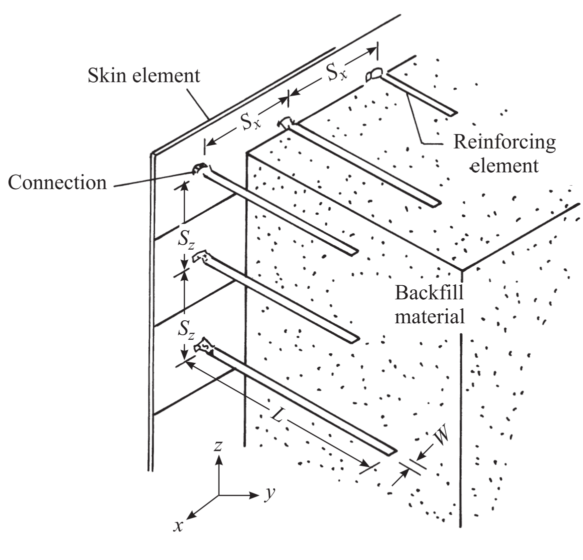

Figure 2.30. Schematic representation of components of reinforced earth retaining wall.

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where w is the width and l_e is the critical length of the reinforcement. This resisting force has to be higher than the lateral pressure developed by the soil

$$2\sigma_v w l_e \tan\phi_\mu \geq K_a \sigma_v S_x S_y$$

$$2wl_e \tan\phi_\mu \geq K_a S_x S_y \tag{2.24}$$

The normal stress $\sigma_v = \gamma H$ influences both the lateral stresses as well as the frictional stresses mobilized. As the height of the retaining wall increases both the stresses components increase. As such, theoretically the retaining wall could be of any height but practically other conditions dictate the permissible height.

The tensile force acting on a reinforcement cross-section of $t\ w$ is $\sigma_y\ tw$. This has to be greater than the tensile force induced by the soil, i.e. $\sigma_y\ tw > w\ \sigma_v\ L_e \tan \phi_\mu$ on the plane of maximum σ_y. Tension failure is dependent on the tensile strength of the reinforcement.

In recent years there have been extensive developments in every aspect of testing, analysis, design and construction of reinforced earth structures. It would be out of place within the framework of this treatise to discuss all the above aspects in great detail. Only a very brief description is provided on some of the specific aspects.

Soil reinforcement interaction mechanisms: Inextensible and extensible reinforcements are the two distinct types widely used in practice. The distinctions mainly concern the strains required to mobilize full strength in the reinforcing elements in relation to the strains required to develop the full strength of the soil. As for the mechanisms of interaction with different reinforcement stiffnesses, the response of the reinforced soil mass in terms of the overall load- deformation is different (McGown et al. 1978) (see Fig. 2.31). When inextensible reinforcements are embedded in soil in the directions of the principal tensile strains, the net effect is to increase the load carrying capacity or reduction of the boundary deformation until the rupture of the reinforcement takes place. Then the system reverts to the behaviour of soil without reinforcement. In the case of a relatively extensible reinforcement, although similar system behaviour is noticed, the maximum force in the reinforcement is controlled by the deformation of the soil. In other words, the imposed deformation of the reinforcement is far less than its rupture deformation. As such, even though the soil loses its strength with deformation the reinforcement imparts additional strength to the system. In either of the above circumstances the strength of the reinforced soil is greater than the soil alone.

The mechanism of interaction between the reinforcement and the soil consists of the mobilization of soil-reinforcement friction resistance, soil bearing resistance on transverse members and the bending resistance in the reinforcement. The influence of bending resistance on the behaviour of the reinforced soil structure, being relatively less, is neglected in most of the cases (Schlossser & Buhan 1990). In a typical reinforced soil structure, limit equilibrium analysis postulates two failure mechanisms:

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1. Tensile stress generated in the reinforcement and resisted by pull-out resistance of the embedded reinforcement length in the stable soil mass dictates the pull-out interaction.
2. Sliding resistance along the soil reinforcement interface – direct shear interaction. Figure 2.32 shows a typical reinforced soil structure where both the mechanisms are operative at different locations (Jewell 1992).

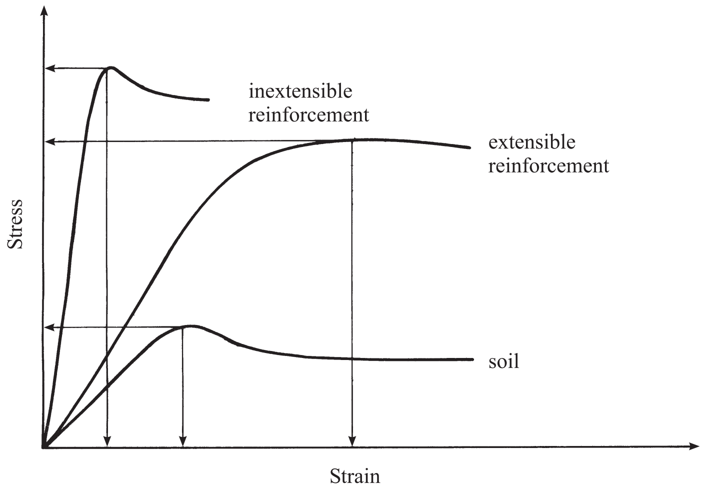

Figure 2.31. Load-deformation response with inextensible and extensible reinforcements.

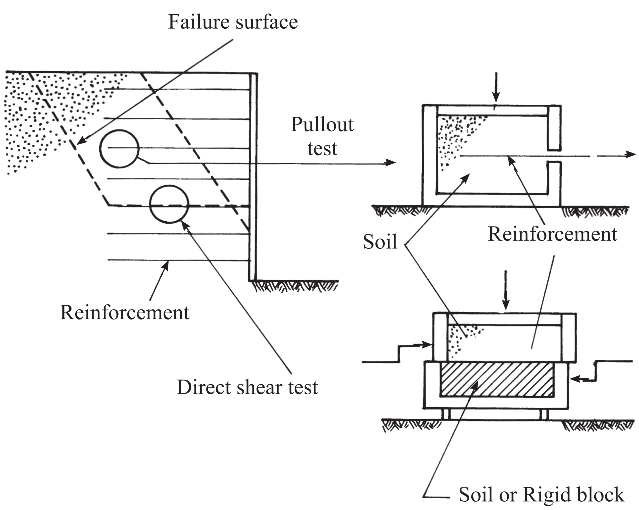

Figure 2.32. Reinforced soil structure with both pull-out and direct shear interaction mechanisms (Jewell 1992).

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Testing: The pull-out and direct shear tests provide both the above interaction parameters. Choice of any one test to measure the interaction parameter is not possible, since these tests yield different values. Hence both tests have to be conducted and used appropriately. From a detailed experimental investigation, using test equipment capable of performing both the pull-out and direct shear tests, Alfaro et al. (1995) show that local mobilization of pull-out interaction resistance can be established through pull-out tests on short geogrid specimens (element test). These are validated by good agreement between the simulated and measured pull-out response of long geogrid specimens (model test) and could therefore be used in the design and analysis of reinforced soil structures. It is also necessary to consider the type of reinforcement used viz., sheet or strip type, in evaluating the pull-out resistance since the non-dilating zone in the backfill soil surrounding the strip reinforcements imposes restraint against soil dilatancy in the dilatancy zone. The primary pull-out interaction mechanism of sheet reinforcement is the classical soil-reinforcement interface which has been designated as two-dimensional (2-D) interaction mechanism (Alfro et al. 1995a). For strip reinforcement, a three-dimensional interaction mechanism develops at both the edges while its middle section experiences 2-D behavior. As the width of the reinforcement reduces, the influence of restrained dilation results in the development of a purely three-dimensional (3-D) interaction mechanism. A methodology to determine the pull-out resistance due to the combined effects of both the modes of interaction has been developed by Alfro et al. (1995a).

Model plate load tests have shown that the shape of the foundation does not affect the behaviour of a reinforced bed, while the size of footing has a similar effect on a reinforced sand bed as in the case of the unreinforced condition (Sridharan et al. 1988). Scale and other factors affecting the results of pull-out tests on grids buried in sand have been examined in detail by Palmeira & Milligan (1989). In the light of three distinct mechanisms – sliding, bonding and bearing – for interfacial friction mobilization, the effects of variation in reinforcement parameters, such as extensibility, flexibility and hardness have been examined (Srinivasa Murthy et al. 1993).

Design: The parametric identification in the design would be to fix the horizontal and vertical spacing of the reinforcement and to arrive at the required length of the reinforcement. The geometrical parameters of reinforcement such as width and thickness are selected by considering the maximum tensile stress to be withstood by the reinforcement without the risk of failure. In practical design, safety factors are applied to the calculated values for each failure mode (Verge & Reid 1976) as cited below.

1. The tensile stress in the reinforcement is limited to two-third of its yield stress.
2. The length of reinforcement provided is at least twice that of the critical value.

In the case of reinforced earth foundations on soft ground the earlier work by Binquet & Lee (1975) has provided a basis for detailed design by considering different modes of failure for the relative disposition of reinforcement layers in relation to the

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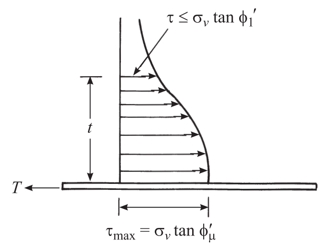
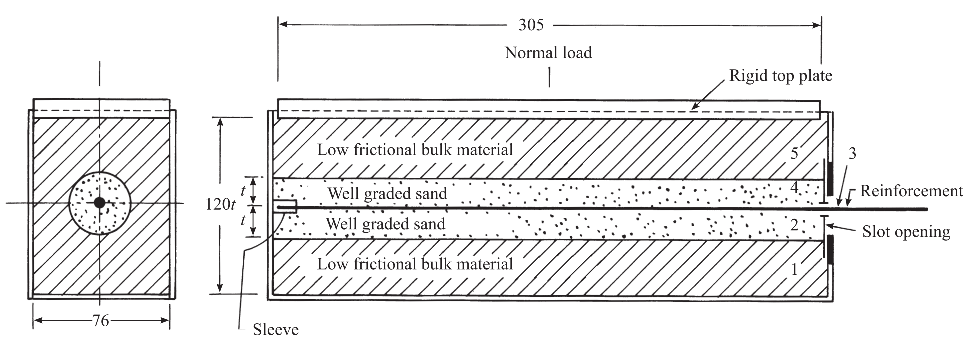
width of the foundation, B. For enhancement of bearing capacity ratios (BCR) the first layer should be located within a depth of 0.25 B. The tensile strength has to be adequate to avert tension failure and the length of the reinforcement should be such as to possess adequate resistance against failure. The results obtained from a parametric study using the elastic continuum approach (Madhav & Pitchumani 1993) indicate that for maximum reduction in surface settlements due to shear stresses, the strips should be placed at depths ranging from $0.75B_f$ to B_f (B_f is half the width of the loaded area for strips of length equal to $2B_f$).

Construction: Mobilization of interfacial shearing resistance is ensured by specifying a highly frictional and well-graded coarse-grained soil as the backfill material. For structures of critical importance and with long service life requirements, fine-grained soil is not advocated due to its low interfacial shearing resistance, loss of adhesion under large strains, excess buildup of pore water pressure and large post-construction settlement. Laboratory studies by Sridharan et al. (1991) indicate the possibility of using fine-grained soils in reinforced earth construction. This is based on the distribution and reduction of induced shear stress in the soil away from the interfacial zone (Fig. 2.33). Fine-grained soils with low interfacial



(a)



(b)

Figure 2.33. a) Shear stress distribution around reinforcement, and b) sandwich layer of sand between low frictional bulk material and reinforcement in sand layer (Sridharan et al. 1991).

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friction angles can be used as backfill material by providing a layer of material with a high frictional strength immediately adjacent to the reinforcement where dilatancy has a role to play in the mobilization of interfacial shearing resistance. The required thickness of the high frictional strength materials depends to a great extent on the surface roughness of the reinforcement and the strength of the fine grained soil and to a lesser extent on the shape and size of the reinforcement. The pull-out resistance of such a sandwiched system will remain essentially similar to the case where high frictional-strength material is used throughout between the reinforcement layers. A thickness of about 15 mm of high frictional strength material as bulk material is sufficient to increase the interfacial frictional angles close to those of the full thickness, high-strength material itself. In a sandwiched system the interfacial friction angles both at peak and at large pullouts are almost equal. All the above concepts are, at present, from laboratory experiments. Their applicability to actual full-scale field problems is still not well established.

2.8.3 *Soil nailing – in-situ reinforced earth*

Most of the earlier adoptions of the reinforced earth principle essentially involved using placement method. In this mode, the reinforcing strips, grids and membranes are placed on the grade and then the layer of soil is spread and roller compacted. The next layer of reinforcement is placed after creating a sufficient thickness of the compacted soil layer to comply with the predetermined vertical spacing.

Another, later technique is to insert reinforcement into in-situ soil. This is known as in-situ reinforced earth (Gassler 1977) or nailing. This technique of reinforcing in-situ soil by stiff linear inclusions has been designated 'soil nailing' (Gassler & Gudehus 1981). In Japan since 1960, this technique has been used under the name 'nuiji (sewing earth)' or 'sashikin (injecting steel bars)' for the purpose of stabilizing natural slopes and at the entrances of tunnels (Hayashi et al. 1992). In this mode, relatively small diameter bars are directly inserted into the ground in a regular array or by drilling and grouting so that there would be adequate synergy between the reinforcement and the soil. This results in a situation that the reinforcement can develop the axial tensile force required to ensure internal stability. The basic principle of this mode of earth reinforcement is illustrated in Figure 2.34.

Construction technique: The excavation is performed in steps usually of 1 to 2 m in depth. The nails inserted are steel bars of 20 mm to 50 mm maximum diameter, with lengths of 5 to 10 m. The spacing is usually such that a nail density of 0.5 to 1 nail per square meter is attained. Figure 2.35(a to e) illustrates the general construction sequence. Installation of the nail, is done either by drilling and grouting or by driving. The most widely used method is to predrill the hole, insert the bar and grout the annular space. In the driven method nails are typically solid circular or tubular bars. The major advantage of this technique is rapid installation, especially in soils where drilling requires casing of the borehole.

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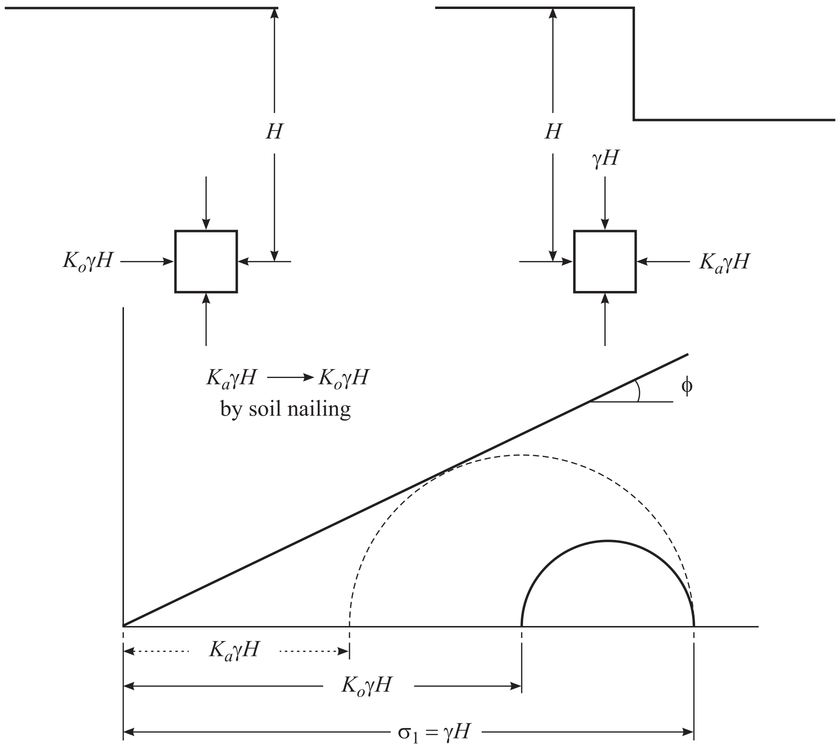

Figure 2.34. The basic principle of the nailing method.

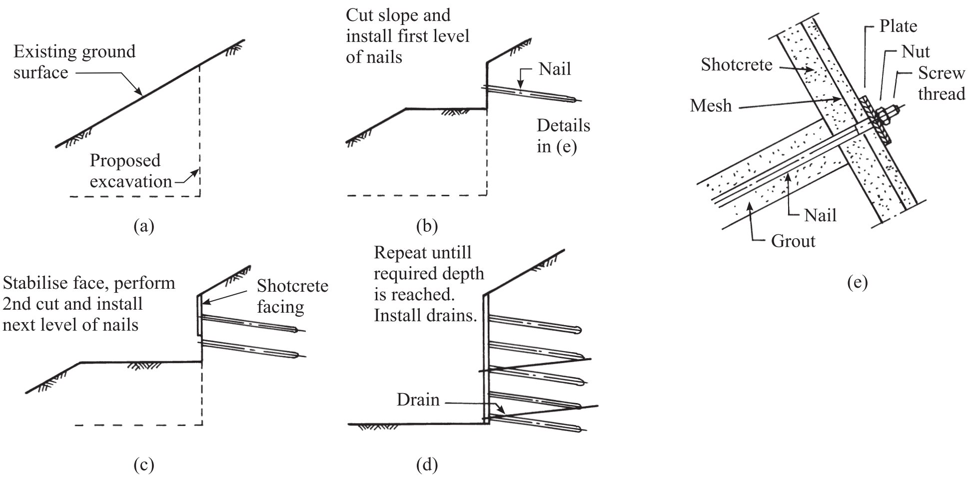

Figure 2.35. Construction sequence of the nailing method.

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Design: Just as in the design of a gravity retaining wall, the stability of a nailed structure must be checked for both global and internal stability. Regarding global stability, it is to ensure that the reinforced zone:

1. must not overturn,
2. will resist outward thrust from the unreinforced zone without the risk of sliding,
3. must resist the lateral thrust without the bearing failure for the combined loading, and
4. will be stable against deep-seated overall failure.

Internal stability is examined for each individual reinforcement for its local stability. The reinforcement elements must have sufficient length and capacity to ensure stable reinforced zone. Overall slip failure need be considered and examined against insufficient bond or breaking of the reinforcement.

Applications: Nailing has been applied to many cases where the soil mass needs to be strengthened. It may be used to improve the bearing capacity and to reduce settlements of soft clays under roadway or railway embankments and industrial structures like storage tanks. Some of the practical cases are illustrated in the Figure 2.36(a to d). Even before any rigorous analysis and design was attempted insitu reinforced earth principle was applied to a deep excavation in a sheet metal hangar of an aircraft industry at Bangalore, India. Another condition imposed was that this deep excavation had to be carried out free from noise and vibrations and without any dislocation in the schedule of work in the factory. It was needed to ensure the safety of the load-bearing wall right next to the excavation. The details are reported in Figure 2.37 (Nagaraj et al. 1982). Wooden planks were the skin

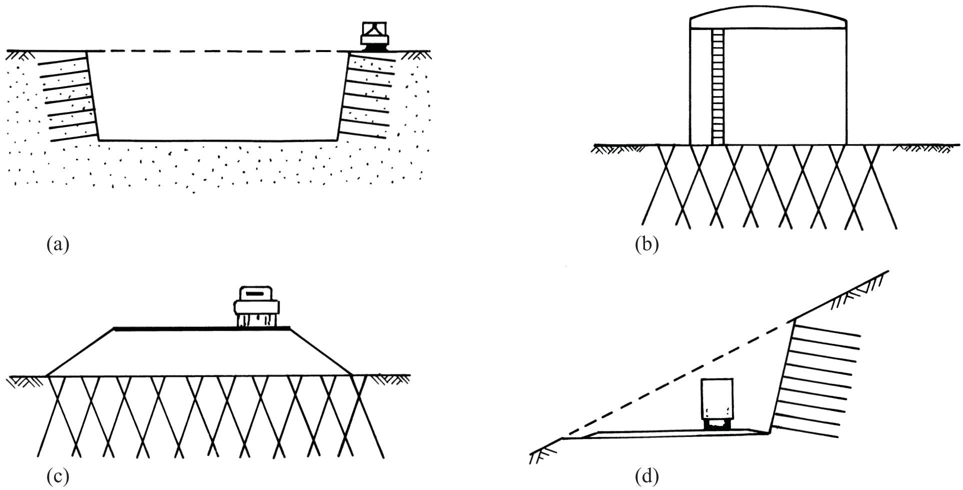

Figure 2.36. Examples of soil nailing.

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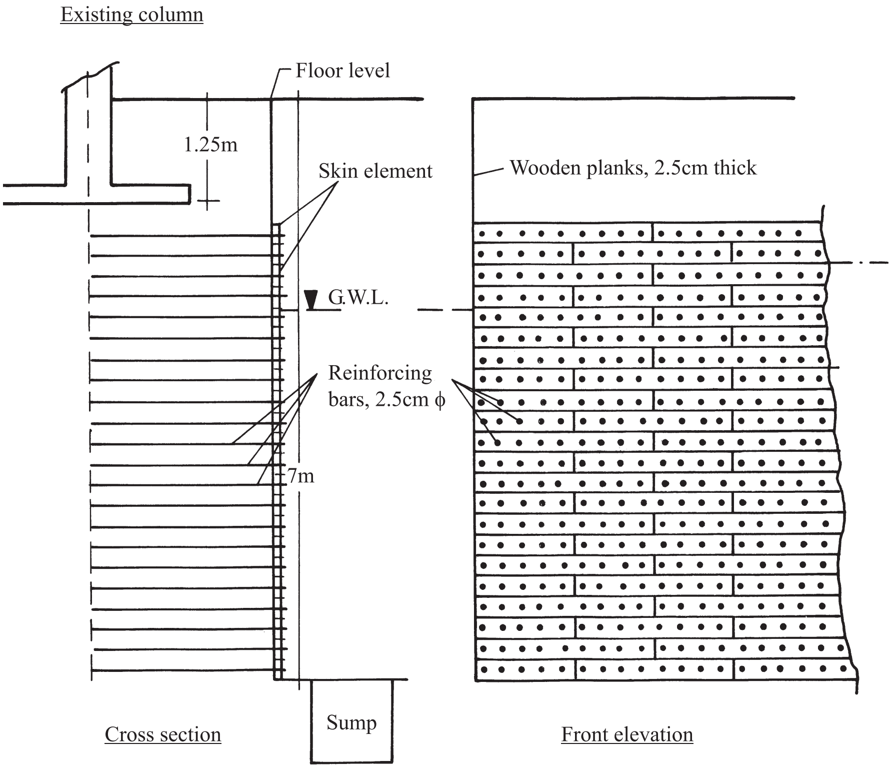

Figure 2.37. Deep excavation adjoining a load-bearing wall in an industrial building (Nagaraj et al. 1982).

elements. Torsteel rods of 25 mm in diameter and 3 m in length were driven into in-situ soil through the holes drilled in the wooden facing elements. By resorting to this techniques deep excavation could be carried out below ground water level, simultaneously ensuring the safety of the adjoining structure.

2.8.4 *Reinforced earth applications in soft ground*

During the past three decades the reinforced earth technique has been in use to solve too wide a spectrum of practical problems on soft ground. It is generally accepted that the provision of reinforcement located near the base of an embankment can substantially improve the stability of embankments or dikes constructed on soft ground. Detailed scanning of available information has revealed that reinforced earth structures could tolerate considerable overall settlements, even greater than one metre with major differential settlements of the order of two percent of span without any serious problems. This essential characteristic soon interested designers who wished to build structures on soft soils, to develop design

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methods and appropriate construction techniques. Geosynthetics, mainly in the form of sheet reinforcements, are used extensively at the bottom of embankments, below pavements, dykes in maritime applications (breakwaters) and landfill basins (Gourc 1993).

Analysis: The behaviour of reinforced embankments involves an interaction between the embankment, the reinforcement and the underlying foundation and consequently the analysis and design of these embankments require consideration of soil-structure (reinforcement) interaction. A number of idealized failure mechanisms for reinforced embankments envisaged are embankment splitting, foundation extrusion, rotational slip and excessive displacement (Rowe 1996) (see Fig. 2.38). Various methods of analysis include limit equilibrium methods, plasticity (bearing capacity) solutions and sophisticated finite element methods. For

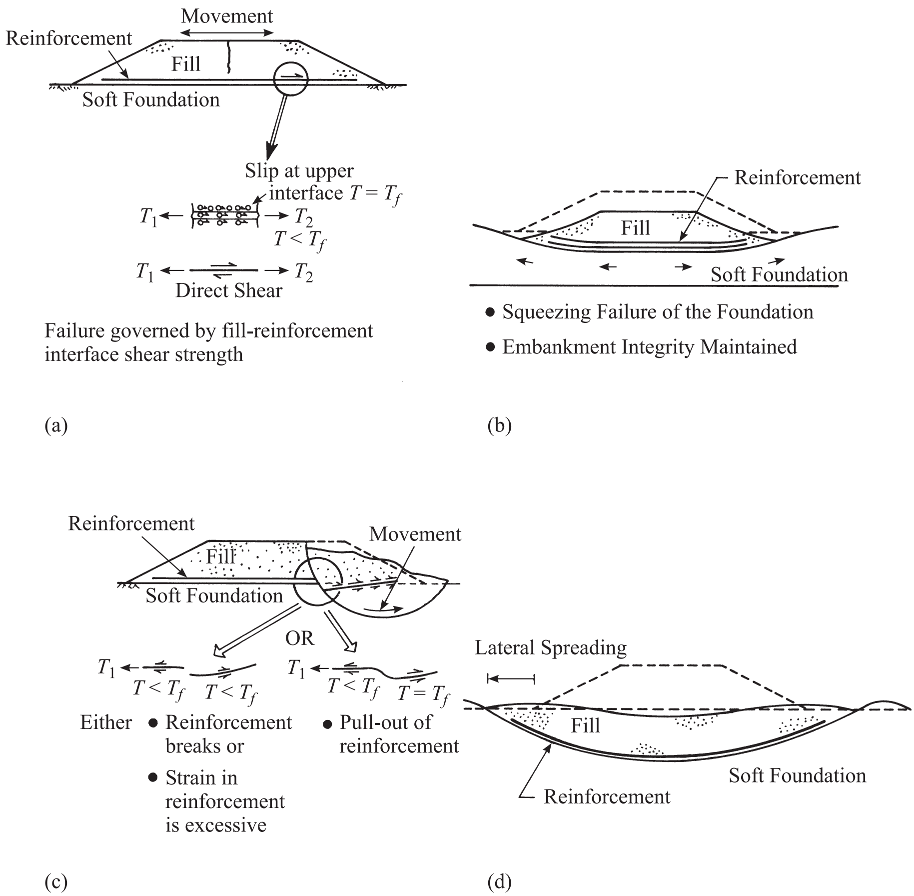

Figure. 2.38. Failure mechanisms of reinforced embankments on soft clay (Rowe 1996). a) Embankment splitting foundation, b) Extrusion of soft material, c) Rotational foundation failure, d) Excessive displacement.

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employment of any of the methods of analysis *the relative stress-strain characteristics of the reinforcement, the embankment fill and the underlying soft clay must be realistically assessed.*

Rowe & Soderman (1987) have proposed a simple method of estimating stability, taking into account the increase in undrained shear strength with depth as well as the relative thickness of the underlying soft clay deposit. Limit equilibrium methods have been used extensively to assess the short term (undrained) stability of reinforced embankments on soft ground (Rowe & Soderman 1985). The finite element method is now well recognized as a powerful analytical tool which can be used to model and analyze reinforced embankments constructed on soft foundations.

A new foundation model element, rough membrane, has been proposed by Madhav & Poorooshasb (1988) to analyze the load-settlement response of geosynthetic reinforced soft soil systems. Analysis of results by this method indicates that, at small displacements, the contribution of shear layers far outweigh the effect of the rough membrane in reducing settlements of the reinforced soft soil. Subsequently, with the Pasternak-type foundation model, the effect of the membrane in increasing the confining stress in the granular material with consequent increase of shear modulus with distance has been investigated (Madhav & Poorooshasb 1989).

It is now recognized that a reinforcement in a sheet or grid form at the base of the embankment built on soft clay improves its rotational stability. A simple closed form solution, for the analysis of the rotational stability of reinforced embankments, has been developed by Kaniraj & Abdullah (1992).

A number of case histories have been reported (Rowe et al. 1995, 1996) to illustrate that finite element analysis can be used effectively to understand how and under what conditions reinforcement contributes to embankment stability. For detailed treatment of the theory and practice of geosynthetic reinforcement of fills over soft ground the reader is referred to the recent paper by Ochiai et al. (1996).

2.9 INDUCED CEMENTATION

Improving the engineering properties of a soil by admixtures such as lime and cement is generally referred to as soil stabilization. Traditional surface stabilization begins by excavating and breaking up the clods of the soil. Then the stabilizer is added. Soil and additives are mixed thoroughly with known quantities of water, then roller compacted and allowed to develop cementation in that densified state (see Fig. 2.39). By this method depths of the order of 150 to 250 mm can be strengthened. Using heavy equipment with appropriate modifications, the depth of the stabilized and strengthened zone may be increased up to one metre. By this means the entire volume of soil up to shallow depths can be altered and strengthened, provided the soil encountered is in the dry stage, and where only addition of

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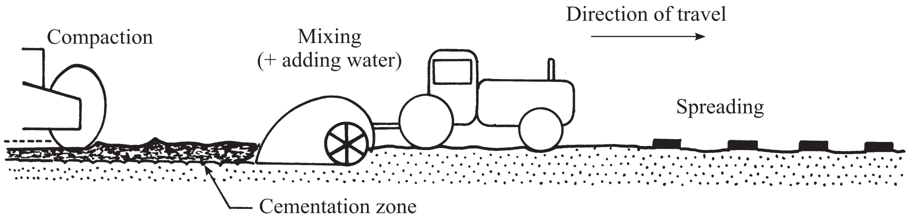

Figure 2.39. Shallow depth admixture cementation.

moisture is resorted to. These methods are used extensively to strengthen bases and sub-bases of highways and airfield pavements, particularly when the natural water contents are in the optimum range for roller compaction.

The above requirement cannot be met in the case of in-situ soft clays since the natural water contents are generally in the range of the liquid limit state or higher. It is not practicable to reduce the water content to the low levels needed for conventional methods of incorporating binding agents and then compacting. Further the depths of strengthened zones required would be far greater than could be realized by conventional methods. Hence to meet the practical requirements of enhancement of load-carrying capacity and reduction in settlements to acceptable levels after construction, strengthening of the soft ground up to appreciable depths at a natural high water content is done by in-situ deep mixing and grouting. It is, in a way, an attempt to redo and even exceed what happens in naturally cemented soft sensitive clays over a geological time scale. It is quite obvious that by such methods the entire volume of the soft clay cannot be transformed into cemented material. What can be attempted is to create a composite ground with vertical cemented zones of soft clay, leaving a considerable volume of soft clay untouched.

This alternative method of soft ground improvement, by mixing soft clay with cementing binders like lime and cement, was developed simultaneously in the 1970s both in Sweden and Japan (Broms & Boman 1975, 1977, Okamura & Terashi 1975, Kawasaki et al. 1981, Saitoh et al. 1985). The strengthened ground produced by in-situ mixing with cementing binders is a composite ground with columnar inclusions although block, wall, and grid types are also widely used to meet various practical needs (Yonekura et al. 1996). It can be done in a grid pattern with spacing and depth dictated by the load to be transferred. It can be done so as to form a barrier in specified zones by having cemented columns with varying degrees of overlapping, or creating a block of cemented soft clay to meet specific needs.

2.9.1 *Methods to incorporate the admixture*

Mechanical mixing and high pressure grout mixing are two methods employed in deep soil conditions. In the mechanical method, the admixtures are incorporated

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by either a dry process or wet slurry method. In this method no water is added to the ground. The cement or quicklime powder is injected into the deep ground through a pipe with the aid of compressed air and then mechanically mixed by rotary wings. Figure 2.40 provides the schematic details. The tool is screwed into the soil down to a depth which corresponds to the desired depth of improvement. The direction of the rotation is then reversed and the tool is slowly pulled out of the ground (2.5 cm/revolution). The cementing agents are forced out into the soil with compressed air through holes located just above or below the horizontal blades of the tool during withdrawal. With rest period, the soil is strengthened. For example, stabilizing with lime increased the strength of soft clay by as much as 10 times in just a few hours. With a two months rest period, lime admixted clay attained half of its final strength, which can be up to 50 times the strength of clay without any admixture. In the dry mix method, since no water is added to the ground, the improvement in strength and stiffness are higher than with slurry.

Nishida et al. (1994) have examined the level of improvement in relation to the degree of mixing between solidification material and clay. To verify this point, coloured plastic pieces were mixed into three types of clay, and the uniformity of the mixing was investigated. The test method is as follows. Add 1200 plastic pieces to 1000 cm^3 soil, and mix them with a hovered soil-mixer for a pre-designated time, then transfer the sample into 12 containers equally and finally count the number of the plastic pieces in each container. The efficiency of the mixing is evaluated by the degree of mixing, which is defined by the deviation of the number of plastic pieces in the containers (Japan Powder Industry Association 1980). It has been observed that with the same mixing time the degree of mixing for Ariake clay is the highest.

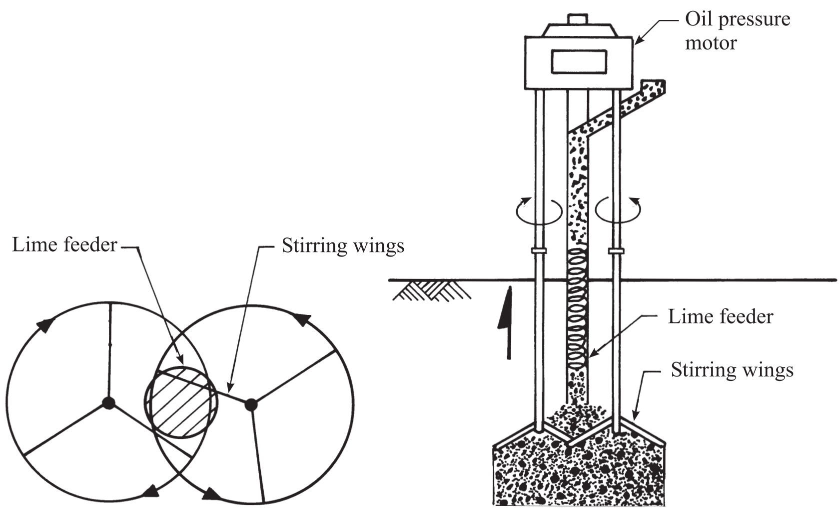

Figure 2.40. Mechanical admixture feeding and mixing arrangement.

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The higher the sensitivity of clay, the higher has been the degree of mixing, which is regarded as one of the reasons for the effectiveness of lime improvement for Ariake clay deposits (Japan DJM Method Research Committee 1993).

The mixing energy (E) per cubic metre of soil (1 m^3) is calculated as the product of time increment, t_i, electric current, A_i, and voltage, V_i. It has been observed (Miura 1996) that the point at which the increase in strength attains a constant level corresponds to 3 minutes mixing time. That is, for the laboratory mixing test, mixing for 3 minutes yields almost the same effect of a mixing time of 10 minutes, as usually used in practice.

Another mode of strengthening the ground by lime is to fill and compact unslaked lime in predrilled holes. The volume expansion that takes place during the slaking of lime consolidates the soil between boreholes. The volume increase of quicklime is about 85% which correspondingly reduces the water content of soft soil around the holes. Instead of unhydrated lime, to stabilize in-situ Bangkok clay, lime slurry has been adopted. The increase in bearing capacity realized has been in the range of 20 kN/m^2 to 60 kN/m^2 after 1 and 6 months respectively (Petchgate & Tunbouterm 1990). Recent laboratory studies by Rajasekaran & Narasimha Rao (1996) and their analysis of lime column and lime injection studies on marine clays, in relation to practical problems, reveal that for high water content clays under large depths of water, the lime injection technique is better suited to improve soft ground.

Jet grout method: This is a method by which a slit is cut underground in a soil deposit with a high speed water jet surrounded by an air jet and a grouting solution is injected into the slit to form a cutoff wall. The various stages of operation are illustrated in Figure 2.41. Guide holes are sunk by auger or boring machine. A monitor is lowered to the predetermined depth and the jetting direction is set; while the water-air jet is shot and the grouting solution is injected, the monitor is lifted.

Column jet pile method: This is a method to form a column of consolidated soil in the ground by fracturing and churning soil and injecting cementing agents. The following, in brief, is the procedure (Fig. 2.42). A guide hole of 15 to 20 cm in diameter is sunk to the specified depths. The monitor is placed in the guide hole. Rotation jetting at high speed has enabled to have a consolidated column as large as 2 to 3 m in soft soil. The compressive strength of the cemented material has been in the range of 30-150 kN/m^2 in sandy soil and 10-50 kN/m^2 in cohesive soil.

As there has been an extensive record of successful applications of this mode of ground improvement, these techniques have proven highly reliable for improvement of soft clayey or sandy soil deposits. The basic technique has undergone rapid changes to enhance its scope and versatility. For example, by providing vertical mixing vanes and a gearbox in addition to the usual horizontal blades in the mixing units, a monolithic rectangular mixing zone can be realized (Watanabe et al. 1996). By use of a superjet, a large diameter (up to 5 m) soil improvement zone, extending to the desired depths, is possible using a small bore hole (Yoshida et al. 1996). A

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new system, JACSMAN (Jet And Churning System MANagement), which combines a mechanical mixing (churning) system and a jet mixing system has been developed (Miyoshi & Hirayama 1996). It uses a cross jetting system with the distinct advantage of creating uniform diameter column of mixing, irrespective of the relative strength and stiffness of the soil layers with depth.

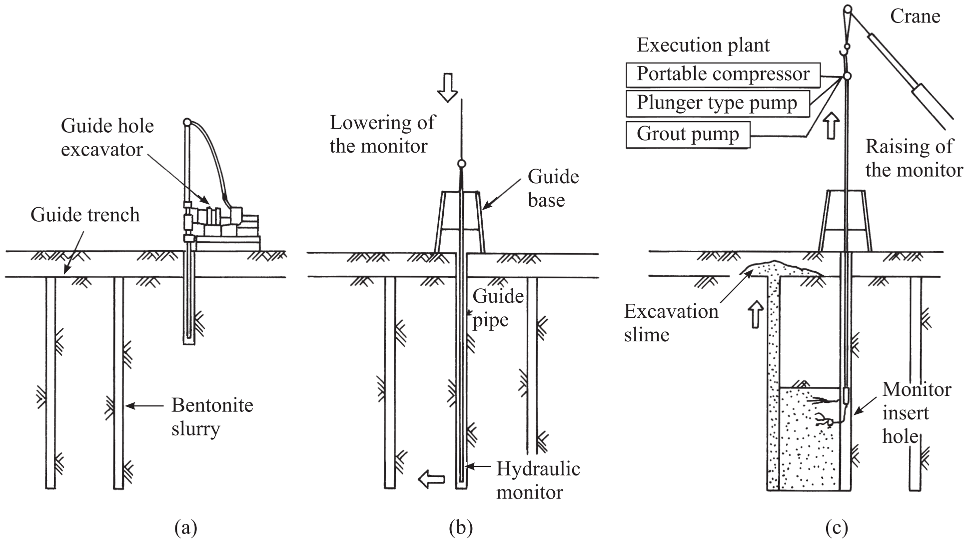

Figure 2.41. Procedure in carrying out the jet grout method.

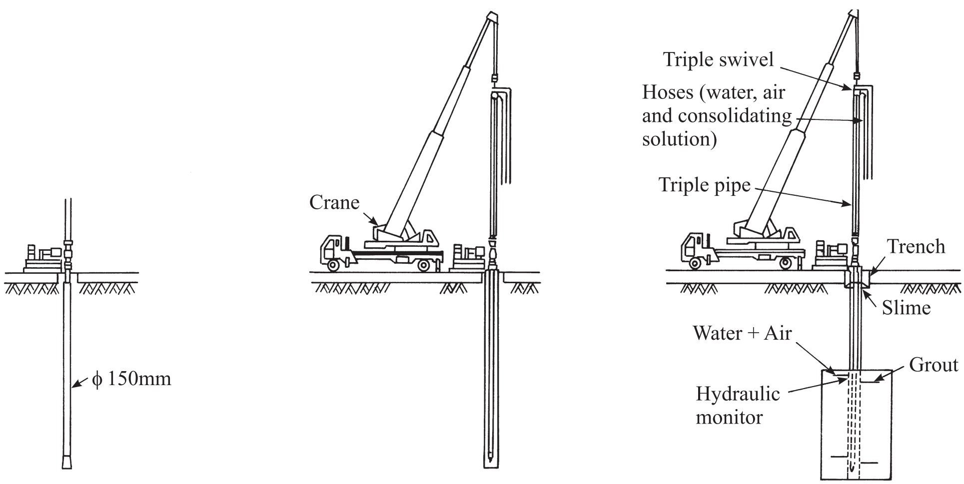

Figure 2.42. Procedure for carrying out the column jet pile method.

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2.9.2 *Bearing capacity*

The ultimate strength of a single column is governed either by the shear strength of the surrounding clay (soil failure) or by the shear strength of the column material (column failure). At soil failure, the ultimate bearing capacity of a single column depends on both the skin friction resistance along the surface of the column and on the point resistance. At column failure, the bearing capacity depends upon the shear strength of the column material. Load tests on excavated columns indicate that failure takes place along the relatively weak joint planes in the columns. The behaviour of the column material is thus similar to that of a stiff fissured clay.

The ultimate bearing capacity of a group is governed either by the bearing capacity of the block as shown in Figure 2.43a or by the local bearing capacity along the edge of the block as in Figure 2.43b. The bearing capacity of a column group can be computed by the expression (Broms & Boman 1978, Bergado et al. 1996).

$$Q_{\text{ult}} = 2\, C_u\, H\,(B + L) + (6 \text{ to } 9)\, C_u\, BL \qquad (2.25)$$

where: B is the width and L is the length of the block, H is the height of the group, C_u is the average undrained shear strength of the soil.

2.9.3 *Settlement analysis*

The columns will reduce both the total and differential settlements. The maximum total settlement is equal to the sum of the local settlement of the composite block,

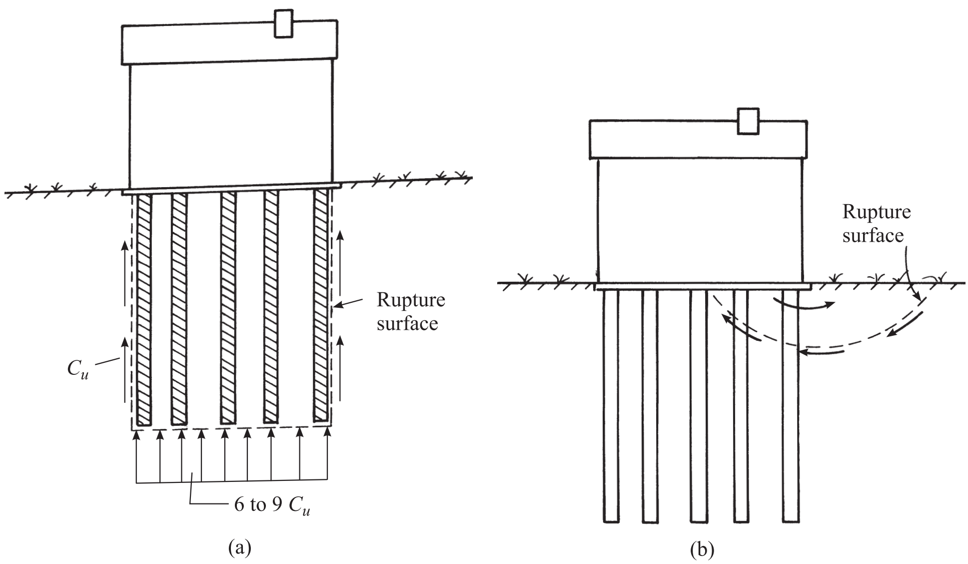

Figure 2.43. a) Bearing capacity failure, and b) local shear failure of composite ground (Broms & Boman 1978).

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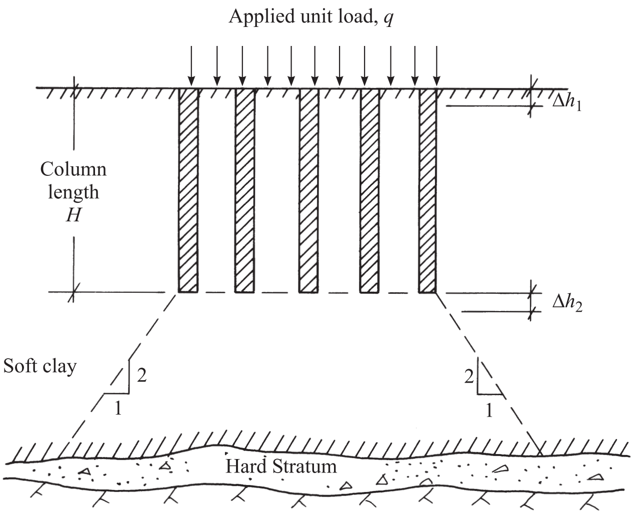

Figure 2.44. Components of settlement of foundation on composite ground (Broms & Boman 1978).

Δh_1 and the local settlement of the unstabilized soil below the block, Δh_2 (see Fig. 2.44). The settlement of the composite block is affected by stress distribution in the block and thus by the interaction of the columns with the surrounding unstabilized soil. Test data indicate that the relative deformation of the columns will be about the same as that of the unstabilized soil between the columns (Bergado et al. 1996). The load due to the structure or that of the fill, q, is partly carried by the unstabilized soil between the columns. The resulting settlements (Δh_1) can thus be estimated from the expression (Bergado et al. 1996).

$$\Delta h_1 = \frac{qH}{a M_{\text{col}} + (1-a) M_{\text{soil}}} \tag{2.26}$$

where: a is the relative column area, M_{col} is the confined modulus of column material, M_{soil} is the modulus of the soil.

The local settlement Δh_2 below the block can be estimated by dividing the soil below the block into layers and by calculating separately the compression of each layer. The estimated cumulative settlement is the sum of settlements of each of the layers.

Damage to the structures is primarily due to large differential settlements. This is mainly dictated by the shear deformation in the unstabilized clay. The angular rotation, α, between two column rows will then be proportional to the average shear modulus G_{soil} of the soft clay resulting in the relation (Fig. 2.45)

$$\alpha = \frac{\tau_{\text{av}}}{G_{\text{soil}}} \tag{2.27}$$

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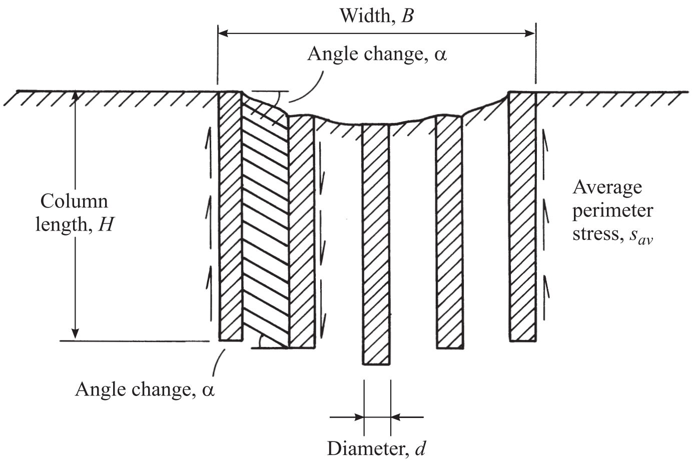

Figure 2.45. Differential settlement due to shear deformations of soft clay below composite ground (Broms & Boman 1978).

2.9.4 *Applications*

Lime columns can be used in soft clay as a foundation
1. to reduce the total and differential settlements,
2. to increase settlement rate,
3. to improve the stability of trenches and deep excavations, and
4. to decrease differential settlements and to reduce the negative drag on structural piles caused by falling ground water levels and the risk of rupturing sewer or water lines.

Some of the specific applications of jet grouting are:
(a) improvement of the ground under an existing structure,
(b) improvement of the resistance capacity of piers and existing piles against a horizontal force,
(c) prevention of slope failure, and
(d) soil improvement below excavated bottom.

Figures 2.46 and 2.47(a to d) illustrate some of the applications.

2.10 CONCLUDING REMARKS

Recent developments in ground improvement techniques have been so many that our discussion in this chapter has been very limited. It has not been possible to cover more than the innovations and methodology. Only those methods which have a bearing on the improvement of soft ground have been dealt with. Even with each of the techniques, many aspects have not been addressed. The discussions provide only a general introduction to the principles, methodology and some

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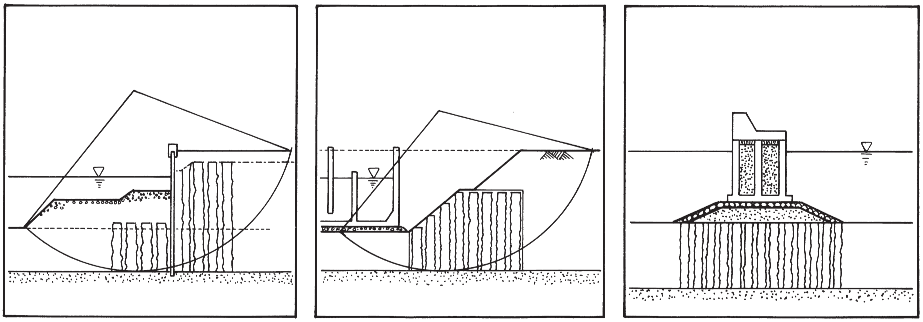

Figure 2.46. Illustrations of practical applications of the jet grout method of ground improvement.

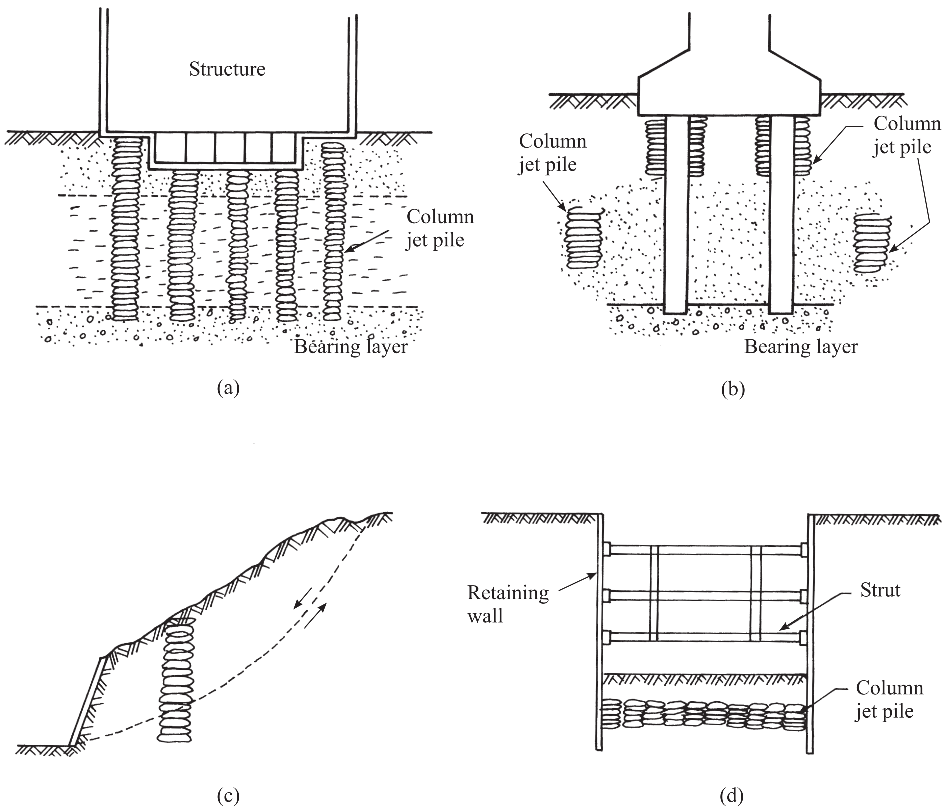

Figure 2.47. Illustrations of practical applications of the jet pile of method of ground improvement.

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applications. For effective doctoring of soft ground, both diagnostic and treatment, with observational approach for monitoring the improvements realized and to take the unavoidable calculated risks, a realistic assessment of the engineering properties of the vast soft ground involved is imperative. Such an assessment would also aid parametric evaluation of various factors involved in the implementation of the appropriate ground improvement method identified for a specific practical situation. To reiterate, some of the specific needs for analysis and assessment of engineering properties, apart from those cited and discussed at different locations in the text, are:

1. In preloading by fill or by vacuum, the level of loading required to effect a particular change in water content may have to be assessed. If the loading level is decided, then the height of fill or suction level can be decided. By determination of the water content at different depths at different time intervals, the possibility of assessing the degree of improvement to engineering properties merits examination.
2. Assessment of the coefficient of consolidation, C_v, of a soft clay layer would help in deciding the spacings of sand drains and geo-drains.
3. In electro-osmotic drainage, assessment of the hydraulic conductivity of natural clay would help in deciding the level of electrical gradient required to achieve a certain level of drainage.
4. In order to compute the bearing capacity and settlement of composite ground, assessment of the undrained strength, and the compression index of soft clay would be helpful.
5. In the induced cementation method of improving soft ground, assessment of the level of admixture to be used and the rest period required to achieve a particular level of improvement of the strength of soft ground, using simple laboratory experiments, would go a long way in the implementation of the insitu deep mixing methods of ground improvement.

In the subsequent chapters the development of the subject would be to analyze the behaviour of uncemented, naturally cemented and induced cemented states of soft clays in order to evolve simple methods to assess the behaviour of soft clays. These methods involve minimum input parameters normally determined in routine investigations.

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CHAPTER 3

Development of the basic framework for analysis

3.1 INTRODUCTION

Soils primarily being particulate media, the stresses to which they have been subjected, the environment in which they are formed and the time that has elapsed, in the geological time scale, at different stages of their formation, have all been recognized as potential factors imparting their effects to the soft clay deposits encountered. It is very well known that the present structure and the consequent mechanical behaviour are the combined effect of different phenomena such as deposition environment, leaching, aging, thioxotropic hardening, and cementation (Bjerrum & Lo 1963, Bjerrum 1967, Leroueil et al. 1979, Hanzawa & Kishida 1981, Nagaraj & Srinivasa Murthy 1983, and others).

The relationship between void ratio and effective pressure was analyzed by Terzaghi (1941), as early as 1941, in relation to the sedimentation compression curve. Terzaghi defined a clay as normally consolidated when it had never experienced a pressure greater than the pressure due to the existing overburden. Figure 3.1 shows a schematic representation of the sedimentation curve. Skempton (1970) presented such curves for a large variety of clayey deposits from very recent to the Pliocene age. Clays in this condition would be represented by any point such as B on the curve A to C. If the existing effective overburden pressure is less than the maximum effective pressure to which the clay has been subjected in the past, the clay is said to be in the overconsolidated state and it would be represented by a point such as D in Figure 3.1. Thus the state of clay at two points B and D at the same effective pressure due to consolidation stress history will have different void ratios.

Skempton (1970) proposed the following two methods to categorize the nature of the in-situ stress state of soft clay deposits.

1. In general, the undisturbed sample is to be tested in the oedometer to obtain its compression path (see Fig. 3.2). From Casagrande's graphical procedure (Casagrande 1936a) determine the preconsolidation pressure σ'_{vc}. The lower limit of $\sigma'_{vc(\min)}$ can be obtained by extending the linear virgin portion of the $e - \log \sigma'_v$ curve to intersect with the horizontal line passing through the in-situ void ratio e_o. If the clay is normally consolidated, σ'_{vc} and $\sigma'_{vc(\min)}$ will straddle the value of σ'_{vo}.

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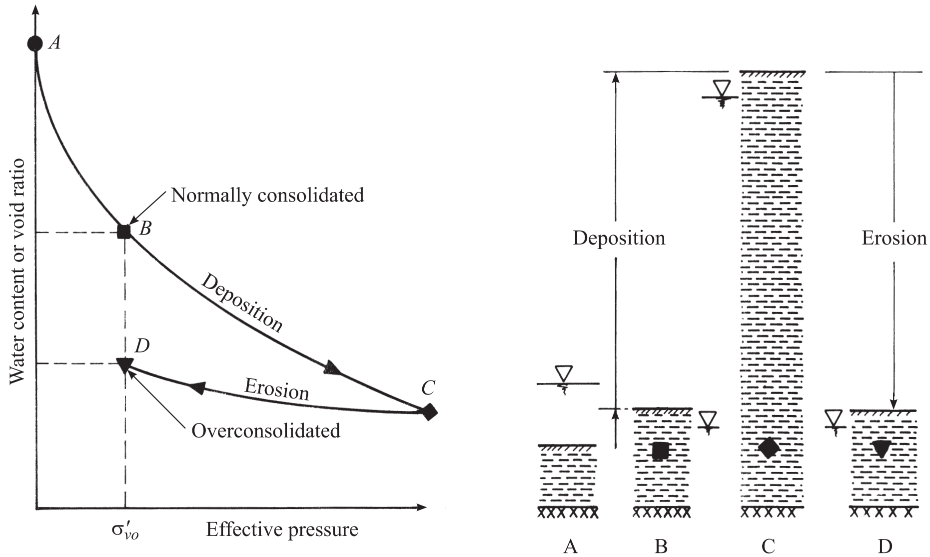

Figure 3.1. Normally- and over-consolidated soft clay deposits (Skempton 1970).

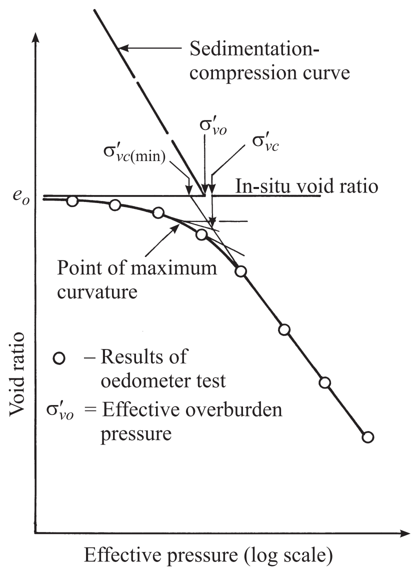

Figure 3.2. Analysis of a typical oedometer test compression path of clay (Skempton 1970).

2. From the plot of the ratio of S_u/σ'_{vo} versus depth, where S_u is the undrained shear strength, the clay is normally consolidated if the undrained strength increases in proportion to the effective overburden pressure.

In the above postulations the construction of the sedimentation curve using the curvature of the $e - \log \sigma'_{vo}$ is done only by the stress considerations, with the tacit assumption that the present state of every sedimentary, normally consolidated clay

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has been preceded by a gradual increase of the pressure from zero to the present overburden pressure, and the sedimentation curve has been built up from the observed compression path obtained from oedometer test on an undisturbed sample.

3.2 RE-EXAMINATION OF THE CLASSIFICATION OF SOFT CLAY DEPOSITS

It is well known that the equilibrium state of the in-situ deposits is influenced by stress, time and environment (see Table 1.2). These are not mutually exclusive processes. It is very likely that the effects of all these factors are not to the same degree. In this case the sedimentation curve by extrapolation tends to be a hypothetical one, since it cannot represent both the decrease in void ratio of the clay and the increase in overburden pressure with time, which could have deviated from monotonic in nature. Is consideration of only the in-situ stress state enough to take care of the geological history which the clay deposit has been subjected to, in the classification of clay deposits? This aspect merits examination.

In fact the effects of time at the same level of overburden stress have been stuidied by Leonards & Ramiah (1960). Subsequently Bjerrum and Lo (1963), Bjerrum (1967) termed the reduction of void ratio under unchanged effective stress, as delayed compression. Figure 2.2 illustrates the effects of time (geological age) on the compressibility characteristics of a normally consolidated soft clay. Delayed compression causes a decrease in void ratio (path AB in Figure 2.2) with a consequent enhanced resistance to further compression with an increasing overburden load. This distinction from the original consideration of stress reduction and again stress increase to regard the deposit in its overconsolidated state was overcome by designating the pressure σ'_{vc} (or p_c) as 'Critical Pressure or Apparent Preconsolidation Pressure'. Whether it is stress dependent overconsolidation or delayed consolidation the $e - \log \sigma'_{vo}$ path does not cut across the sedimentation curve where the time effects are negligible.

The reference path cannot be obtained from the compression paths of undisturbed clay as the effect of time is always masked by the stress history effects. The effect of the chemical environment further complicates the demarcation of the effects of stress and time on the compression and shear behaviour of soft clays. This predicament was recognized by Leonards (1972) as early as 1972. Without necessarily linking this to any stress history, the stress levels which mark the beginning of yielding were termed as 'Apparent or quasi preconsolidation pressure' (Leonards 1972). All the above efforts were directed to find reasons for the apparent experimental observation of a bilinear $e - \log \sigma'_v$ trend in the laboratory compression path in an oedometer test on an undisturbed soft clay sample.

While examining the fundamental considerations of the undrained strength characteristics of alluvial marine clays, Hanzawa & Kishida (1981) and Hanzawa (1983) categorically state that naturally deposited marine clays are usually in an

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overconsolidated state even though they have been subjected to release of the overburden because of the additional strength developed by the aging effect, such as due to secondary compression and chemical bonding. The need to recognize behaviour different from that of normally consolidated situation arose since these natural clays exhibited brittle behaviour to a higher degree within the yield stress level.

In the twenty-second Terzaghi Lecture on 'Stability evaluation during staged construction', Ladd categorically comments on preconsolidation pressure. To quote (Ladd 1986 – page 564) on 'Preconsolidation pressure: Significance – although originally considered, and still often called, the maximum past pressure that acted on clay, the profession generally views this as representing a yield stress that separates small strain elastic behaviour from strains accompanied by plastic (irrecoverable) deformation during one dimensional compression. This distinction has practical significance when attempting to select a profile consistent with the geologic history of the deposit and in estimating the in-situ state of stress'. On page 569 (Ladd 1986), to quote again, 'SHANSEP' (Stress history and normalized soil engineering properties) assumes mechanically overconsolidated behaviour to represent all preconsolidation mechanisms, and hence involves obvious errors with highly structured, sensitive clays and with naturally cemented deposits'.

Earlier attempts to examine the grouping of natural soft clay deposits are due to the efforts of Bjerrum (1973), Janbu (1977) and Hanzawa & Kishida (1982). The soft clay deposits accordingly have been categorized into four groups. State of clays dependent only on the present and past stress history were identified as normally consolidated and overconsolidated young clays. Two additional types introduced to consider the effects of time and cementation were normally consolidated and overconsolidated aged clays, which still had a bias towards the stress state and stress history. This regrouping still relied heavily on the present stress state and its stress history.

The question that arises now is: Is it necessary to have prefixes such as normally consolidated and overconsolidated, which are relevant only for mechanically stressed systems, even though we are fully aware that the soft clay deposits are formed due to the combined effects of stress, aging and cementation, with their response primarily dicated by any one of them having overriding influence on the overall engineering behaviour? Can we still manage with the present practice of assessment of preconsolidation pressure and analyze soft clay behaviour within the general framework already established? The answer is not total affirmation. Some of the specific cases in the analysis of soft clay behaviour where difficulties arise from such practices are;

1. Assessment and quantification of sampling disturbances and accounting for the same in the modifications of soft clay engineering properties.
2. In the development of appropriate constitutive relationships needed for application of numerical methods to solve practical problems encountered in soft clay engineering. For example, plasticity models are not directly applicable to cemented sensitive soils, since their post-peak behaviour is totally different from

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that of overconsoldiated soils in the sense that softening in these soils is associated with continued volumetric compression or positive pore water pressure (see Fig. 3.3).

The logical step to re-examine the inherent in-situ characteristics of soft clays due to the effects of past stress history, elapsed time and cementation in their formation

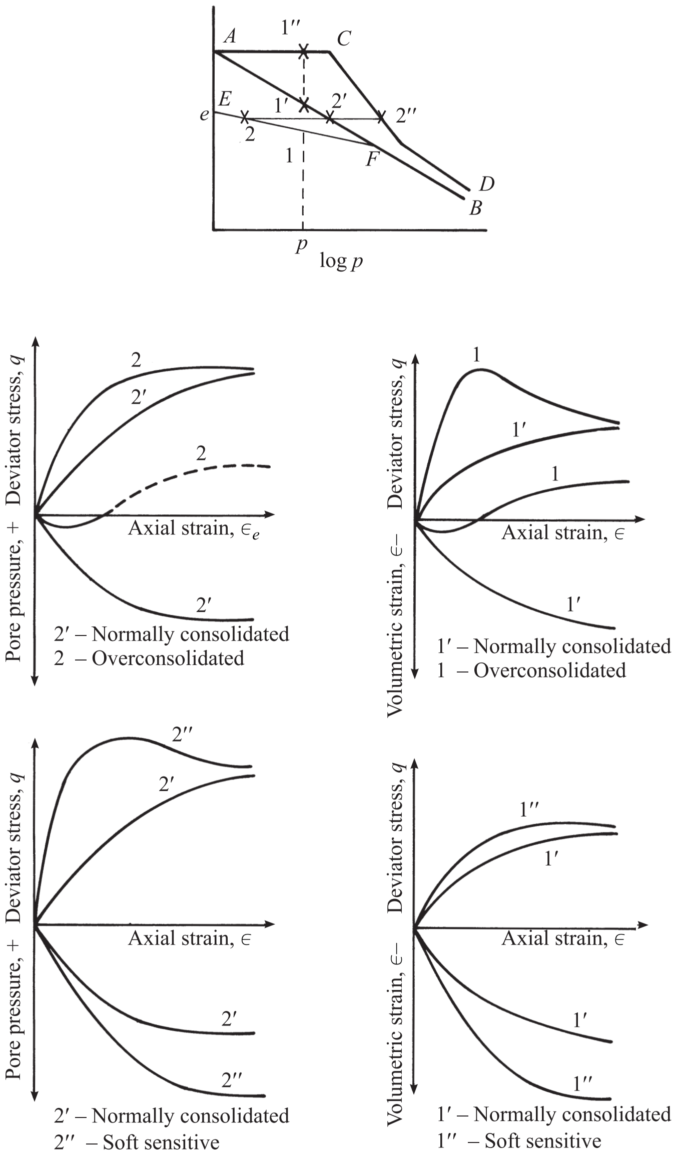

Figure 3.3. Schematic representation of stress-strain-porewater pressure or volumetric strain response of uncemented overconsolidated and soft sensitive clays.

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is to examine the in-situ data from the published literature. Table 3.1 shows the details of natural water content, liquid limit water content of the clay, in-situ vane strength, overburden pressure and the stress level at which the $e - \log \sigma'_v$ path exhibits a bi-linear path/shows a change in curvature in its compression path. One hundred soft clay data from different regions of the world, collected and collated, are tabulated (Nagaraj et al. 1997a). A cursory glance reveals that no specific pattern is apparent between any of the above parameters. It is also not possible to make any further examination unless the effects of stress alone on the state of clay are characterized, since the effects of time and cementation are subdued in the natural water content or void ratio, itself a state parameter of soft clays. Hence systematic characterization of the individual factors merits examination.

Soils being primarily particulate media, their engineering properties at all stages are dictated by the effective stresses to which they are subjected. All measurable changes in volume, deformation and mobilization of shearing resistance are exclusively due to changes in the effective stress as enunciated by Terzaghi (1936). The soils would exhibit engineering properties only when they acquire a small but measurable effective stress. The compressibility characteristics of clay-water systems devoid of any past stress history, time and cementation effects, merit examination to build up the basic framework for systematic analysis of the behaviour of natural soft clay deposits with the effects of unknown past stress history, aging effects over a geological time scale, with or without cementation. This precludes detailed examination of the fundamentals of soil behaviour.

3.3 FUNDAMENTALS OF CLAY BEHAVIOUR

Broadly, to understand, generalize and assess soil behaviour there are three levels of investigation, the molecular level, the structural level and phenomenological level (Klausner 1991).

The subtlest level is the molecular level in which the behaviour of each individual molecule is predicted from well-defined physical laws and their cumulative effect is computed to get the total response of the material. In principle, although this is the most exact level, the difficulty in identifying and accounting for all the interactions at the submicroscopic level is often insurmountable. The mathematical formulations to describe the behaviour in totality would normally tend to be involved and complex. Measurements and observations at this level of probing would be highly sensitive and refined, which would be out of bounds for routine investigations. The adoption of the Gouy-Chapman diffuse double layer theory for assessment of compressibility of clays (Sridharan & Jayadeva 1982) is an example. The rate process theory which has been used to analyze shear and creep behaviour of soils (Mitchell et al. 1968) is another example. By this approach the activation energy required to displace each flow unit at molecular level is related to the macro level response.

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Table 3.1. Soft clay deposits – data from published literature.

Sl. No	Soil	w_n	w_L	S_u kPa	σ_o kPa	σ_y kPa	Reference
	JAPAN						
1	Ariake clay – – at 5 m	133	121	12	30	40	Tanaka et al. (1996)
2	– at 10 m	133	110	20	50	70	
3	– at 15 m	100	90	26	70	95	
4	Kinkai site – – at 10 m	110	110	20	45	65	
5	– at 15 m	120	115	30	60	75	
6	– at 20 m	120	115	37	75	105	
7	– at 25 m	50	50	40	90	110	
8	Ariake Bay – Kyushu at 5 m	95	105	10	20	25	Hanzawa, et al. (1990)
9	– at 10 m	80	105	13	40	40	
10	– at 15 m	65	80	23	60	72	
11	Ariake Bay – Location 1	87	87	–	46	70	Onitsuka, et al. (1995)
12	– Location 3	73	77	–	85	90	
13	– Location 6	115	90	–	18	50	
14	– Location 7	121	85	–	16	80	
15	Shimbara clay – Location 7	60	54	–	91	200	
16	Osaka clay	69	72	–	50	94	Adachi et al. (1995)
17	Kuwana clay – Nagoya	76	91		156	240	
18	Komatsugawa clay-Tokyo at 10.4 m	35	45	92	100	200	
19	at 22.5 m	30	35	135	165	390	
	Natsushima clay–Tokyo						
20	at 5 m	78	86	32	34	122	Hanzawa (1979)
21	at 14.5 m	75	79	55	88	165	
22	at 25	89	93	70	195	240	
	SE – ASIA						
23	Changi-Singapore – 5 m	70	70	20	20	80	Choa (1995)
24	– Stiff silty clay at 30 m	55	80	50	160	200	
25	Bangakok clay	79	95	–	52	61	Kuwano & Bhattarai (1989)

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Table 3.1. Continued.

Sl. No	Soil	w_n	w_L	S_u kPa	σ_o kPa	σ_y kPa	Reference
26	Sungham – Taipei, K1T2						
	– 5 m	40	30		20	120	Chin et al. (1994)
27	– 25 m	30	26		160	220	
	CANADA						Capozio et al.
28	Noranda mines	65	58	7	24	27	(1982)
29	Noranda Qubec	89	72	25	65	85	Roy et al.
30	St. Alban	95	52	10	15	32	(1982)
31	St. Alban	60	40	19	35	80	Roy et al.
32	St. Alban	40	28	26	42	100	(1982)
33	Champlain						Silvesteri
	clay – rang east	55	50	55	135	200	(1980)
34	Champlain						Silvesteri
	clay – rang east	83	60	35	77	105	(1980)
							Yong et al.
35	Orleans	69	76	60	20	276	(1980)
36	St. Alban	60	40	18	–	72	Roy (1979)
37	St. Alban (3 m)	90	51	60	48	244	Lo & Becker (1979)
38	Silty clay	40	50	–	22	50	Leroueil et al.
39	Berthierville	72	59	14	21	47	(1979, 1983)
40	St.Cesaire	89	68	25	55	80	
41	St. Cesaire	85	70	27	68	90	Leroueil et al.
42	Gloucester	88	52	20	35	65	(1979, 1983)
43	Gloucester	76	53	20	38	67	
44	Gloucester	93	53	25	58	87	
45	Varennes	62	65	60	64	216	
46	Joliette	65	41	29	40	110	
47	St. Catherine	88	60	18	20	60	
48	Masauche	65	55	70	34	270	
49	St. Alban	60	40	13	25	55	Leroueil et al.
50	Fort Lenox	60	45	30	54	105	(1983)
51	Louiseville	75	70	45	58	160	Leroueil et al.
52	Batiscan	80	43	25	60	88	(1983)
53	St. Louis	69	50	–	40	95	Morin (1975)
54	St. Vallier	59	60	–	34	85	
55	Breckenridge	79	65	36	67	120	Crawford & Eden
56	Sewage-plant	55	36	100	38	300	(1965)
57	Green C-fill	66	70	198	238	490	
58	Green C-fill	69	47	102	202	300	
59	Nat. Research	68	38	55	83	180	
60	Main street	51	35	57	52	230	

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Table 3.1. Continued.

Sl. No	Soil	w_n	w_L	S_u kPa	σ_o kPa	σ_y kPa	Reference
61	Gloucester	72	49	21	16	60	
62	Kars	60	40	29	38	120	
63	Ottawa sewage Plant	60	49	136	145	439	Gillot (1970)
64	–ditto–	45	31	146	200	488	
65	Walksy Road	58	53	58	107	234	Gillot (1970)
66	HMSC-Glouc.	70	43	21	48	68	
67	St. J. de T-Q	22	29	146	105	488	
68	Batican	79	43	25	–	88	Leroueil et al.
69	Joliette	65	41	29	–	115	(1985)
70	Louiseville	76	70	45	–	160	
71	Mascouche	67	55	70	–	270	
72	St. Ceasaire	85	70	27	–	90	
73	Rigaud clay	75	60	45	40	160	Silverstri (1984)
74	Champlain clay	65	45	17.5	30	65	Tavenas et al. (1974)
75	–ditto–	70	42	12	22	46	Delage & Lafebvre (1984)
76	St. Marcel clay	80	60	15	23	54	Lo (1972)
77	Lonsille clay	83	63	30	50	90	Bozozuk (1984)
78	Gloucester clay	80	50	15	15	44	Lefebvre et al. (1994)
79	–ditto–	66	50	12	18	60	
80	St. Alban, at 2m	96	52	10	17	40	
81	–ditto– at 8m	48	28	27	45	108	
82	Batiscan	86	49	–	30	50	Mesri et al.
83	St. Hilaire	84	55		52	75	(1995)
	NORTH ATLANTIC REF. USA						
84	S-Nares Abys.	105	104	7	27	27	Zizza &
	Plain(Core66)	72	104	26	114	137	Silva (1988)
85	–ditto–	100	70	2.8	10	12	
86	Bermuda Rise	102	97	8	12	36	
87	Gulf Stream						
88	SohmAbyssal Plain	100	66	1.8	5	7.5	Silva & Booth (1986)
	NORWAY						
89	Drammen clay Plastic clay, at 5m	50	55	10	45	72	Bjerrum (1967)

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Table 3.1. Continued.

Sl. No	Soil	w_n	w_L	S_u kPa	σ_o kPa	σ_y kPa	Reference
90	Lean clay, at 15.5m	30	32	10	125	160	
	NORWAY						
	Skoger Spare						Bjerrum (1967)
91	Bank at 12.5m	45	55	30	120	130	
92	at 15m	43	55	45	140	180	
93	Konnerud site at 10m	36	38	30	80	125	
94	at 20m	36	38	28	160	180	
	Scheitlies site						
95	at 7.5m	52	55	20	75	90	
96	at 24m	30	30	40	200	220	
	IRAQ						
	Fao clay						
97	at 10.5m	50	55	35	60	80	Hanzawa (1977)
98	at 18.5m	38	48	70	100	150	
99	Kohr Al-Zubair, at 9.5m	42	58	55	170	238	Hanzawa
100	at 12.75m	41	55	75	230	320	et al. (1979)

w_n = the natural water content, w_L = the liquid limit water content of the soft clay, s_u = the undrained shear strength of the clay in kPa, σ_o = effective overburden pressure in kPa, and σ_y = the transitional stress in the $e - \log \sigma_v$ compression path.

The next level of investigation is at the microstructural level, in which the interactions between multicomponent and multiphase constituents of the system are considered in the formation of the resultant microstructure under the influence of external stimuli and to link this with the macro-level response. Even in situations where the physico-chemical interactions are not dominant, as in the case of granular media, forces and deformations at each of the grain contacts for defined initial packing are analyzed by discrete particulate mechanics, thereby enabling us to pursue this method.

The most commonly adopted mode of probing to assess material behaviour is at the phenomenological level. In this approach material is considered in its entirety with the observed response forming the basis for property correlations. Models based on continuum mechanics and principles of elasticity and plasticity are some examples. Further, this mode of probing, as is true in any field where practice or application precedes scientific understanding, has enabled to develop em-

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pirical relations for correlating mechanical properties with physical and state parameters.

They have almost become indispensable tools owing to their simplicity. Some of the widely used relations are:

1. Skempton's (1944) compression index equation, and its modified forms, to assess the compressibility of fine-grained soils.
2. Skempton's (1954) c_u/p vs I_p relationship to assess the undrained strength of in-situ soils from data of overburden pressure and index properties.
3. Undrained remoulded strength versus liquidity index, I_L relationship (Houston & Mitchell 1969).

Often, for a complex material like soil, it may not be possible to generalize and assess the soil behaviour, in all its entirety, consistently using only one of the above approaches. Perhaps what is desirable and practicable would be to have a combination of the above approaches. Possibly, micro level considerations might provide a basis for qualitative understanding of the mechanisms involved and as well as to identify appropriate parameters at macro level. By the use of such parameters and by using extensive representative experimental data, functional relationships between engineering properties can be formulated. If the parameters identified happen to be those normally determined in routine investigations, then it would be an added advantage.

It is needless to stress that it is neither possible nor attempted here to develop a unified model to assess soil behaviour encompassing all soils like clays, silts and sands formed under different stress and environmental conditions. Hence, even before considering the effects of stress and environmental factors, it would be necessary to consider the nature of soil solid constituents and their interactions with water.

In retrospect, as discussed in Section 2.2, soils are primarily particulate media. According to Feda (1982), particulate mechanics provides a useful tool to analyze responses of the soils. The particulate materials are those which exhibit dilatancy and contractancy and are sensitive to hydrostatic stresses. They are composed of mutually contacting solid particles or stable clusters of particles in the fluid phase of clay-water systems. The structural unit is that part of the soil fabric which partakes as a single entity during the deformation process. In order to understand clay soil behaviour from micromechanistic considerations, it would be advantageous to examine the fabric and structure of the clay. In order to do that it would be necessary to be appraised of the fundamentals of soil behaviour.

3.1.1 *Nature of soil solid particles*

In retrospect, soils in nature encompass a wide range of particles varying in size over two million times. It so happens that along with this wide range in particle size go a large number of other differences such as particle shape and specific surface, mineralogy and associated physico-chemical properties. Generally particles

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coarser than 0.002 mm are bulky and isometric in shape and exist as individual stable units (see Section 1.3). The common minerals constituting these are silicates (feldspars), oxides (silica and iron), carbonates (calcium and magnesium) and sulphates (calcium). On the other hand, clay minerals are hydrated aluminium silicates and hydrous oxides of aluminium, magnesium and iron in a crystalline form of relatively complicated structure. The clay minerals are broadly of three groups based on their crystal structure viz., kaolins, montmorillonites and illites. From an engineering standpoint, soil devoid of clay and silt fraction are regarded as sands (coarse-grained) and those comprising of broader range of particles with appreciable clay fraction are clays (fine-grained soils).

3.1.2 *Clay-water interactions*

Most of the clay minerals are crystalline in the form of defined submicroscopic structural units. The basic stable unit of clay mineral varies over a wide range right from 500-1000Å in a typical kaolin crystal to 10-100Å in smectites (Grim 1968). The direct implication of particle size variation is in their specific surface (see Table 1.1). As an analogy, the vast specific surface of clays can be visualized from the fact that about 12 gms bentonite clay would suffice to cover a football field when particles are placed side by side. This has a direct bearing on physico-chemical interactions with the pore medium. In sand and silt, since the surface area is relatively small, surface molecules constitute only a fraction of the total molecules of the particle. In clays a large portion of the total molecules constitute the surface molecules. Hence, of the total surface charge it can carry, both, positive and negative, the net charge is negative. This may arise from any one or the combination of the following reasons.

1. isomorphous substitution,
2. surface dissociation of hydroxyl ions,
3. crystal lattice defects and broken bonds,
4. adsorption of anions, and
5. presence of organic matter.

The first two of these reasons have been regarded as dominant in clay minerals while broken bonds occur in coarse-grained soils. Although per unit mass total electrical charge, is directly related to the specific surface, the surface activity of sand does not acquire the level of clays even if ground to clay-size particles. This is so because of the non-chemical nature of their surfaces. In most of the clay mineral structures the chemical aspects are absorbed by full or partial replacement of aluminum ions (Al^{+3}) by other cations of lower valency to form isomorphs of the minerals. This results in increased charge deficiency on the surface. In kaolins, due to their inherently stable structure, the surface charge arises more by broken bonds than by isomorphous substitution.

The ability of a clay to adsorb ions on its surfaces or edges is reflected by its exchange capacity. This depends upon the surface charge and the specific surface.

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In clays, since negative charge dominates, it is mostly a case of cation exchange and hence they possess base exchange capacity. This process is one in which the cation adsorbed on the surface of the clay is exchanged for a cation in the pore medium. It is expressed as milliequivalents per 100 gms. of dry soil. One milliequivalent is one milligram of hydrogen or the amount of any ion which will combine with or displace one milligram of hydrogen. The typical value of base exchange capacity in meq/100 gms of dry soil are in the range of 4, 40 and 80 for kaolinite, illite and montmorillonite clay minerals respectively. Thus it is to be appreciated that the characteristics of a clay particle are not unique. They vary both in their mineralogical composition and the environment.

Since in most of the cases the pore medium is water and is held by the clay particle surfaces it merits examination of the structure of water. In water molecules the centres of positive and negative charges do not coincide and hence they behave like dipoles (Fig. 3.4). The distribution of electrical charges in a water molecule resembles a tetrahedron, with two vertices charged positively and the other two negatively. This specific arrangement of the charges in the water molecule permits it to bond with neighbouring water molecules, so as to form a definite structural pattern similar to the crystalline solids. However, these structural forms are not stable because of weak bonding and local thermal effects.

Clays are strongly influenced by the presence of water because of their high surface activity. The causes of clay-water interactions are:

1. The water held on to the clay particle by hydrogen bonding in which the hydrogen of the water molecule is attracted to the oxygen or hydroxyl units on the surface of the particle.
2. The water molecules are electrostatically attracted to the surface of the clay particle.

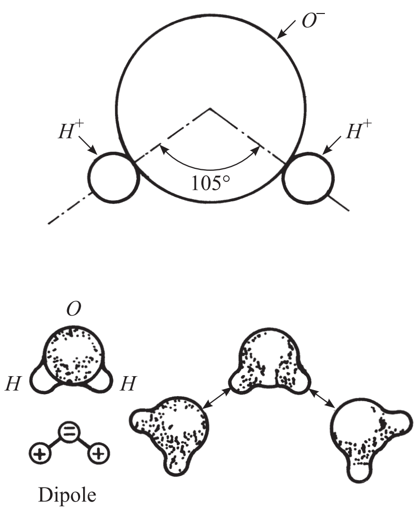

Figure 3.4. Structure and orientation of water molecules.

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3. The hydrated exchangeable cations present in water are electrostatically attracted to the clay surface.

The water of hydration held by the clay surface with a highly oriented structure is the adsorbed water. The clay-water interaction results in a tendency of counter ions and other exchangeable cations in the water to diffuse away from the surface. This will be counterbalanced by the electrostatic attraction. In retrospect (see Section 2.2.2) the charged surface and the strongly held cation at the surface, together with the relatively mobile counter ions adjacent to the surface, are considered to be two layers, and the whole system is referred to as the 'diffuse double layer'.

Interparticle interactions: Between two units of matter, there always exist interaction forces which are both attractive and repulsive in nature. These can be broadly grouped as short and long range forces. Depending upon the dominance of any one type, the effect of the other would be subdued. While the long range forces can act over several hundred angstroms, the action radius of short range forces are only a few angstroms. Some of the typical long range forces are the van der Waals – London attractive and Coulombic repulsive forces while the homopolar, the heteropolar, the born repulsion, hydrogen bonding and close range forces represent the free surface energy brought into action by the interphase boundary, i.e. by the failure of crystal lattices on their surface, and hence are bound to that surface. The magnitude of short range forces varies as the inverse seventh power of the distance (r^{-7}) between two solids.

The long range forces originate between particles of colloidal dimensions. A typical example of this is clay particles in an aqueous medium. Long range attractive forces are inversely proportional to the third power of the distance between the particles (r^{-3}), while the repulsive forces are inversely proportional to the second power of the distance (r^{-2}). Hence, of the long range forces, the repulsive forces are dominant.

3.1.3 *Physical and physico-chemical interactions*

In the case of coarse-grained soils short range forces are dominant. The mutual interactions between particles is predominantly physical as illustrated in Figure 3.5 (Leonards 1962). While the gross macroscopic view of interactions between two particle surfaces appears to be as if between two polished surfaces (Figure 3.5a), in reality, at the sub-microscopic level, the same would be jagged and irregular as depicted in Figure 3.5b. This is so because the asperities of these solid surfaces can be of the order of several thousands of angstroms. Surfaces in contact macroscopically imply that contact zones would primarily be between the apexes of the asperities. At these points, only short range forces are in play and the long range forces are insignificant, since at every other point than in the vicinity of the contact, junctions are formed as shown in Figure 3.5c. These contact junctions offer resistance to sliding and rolling and hence they are regarded as

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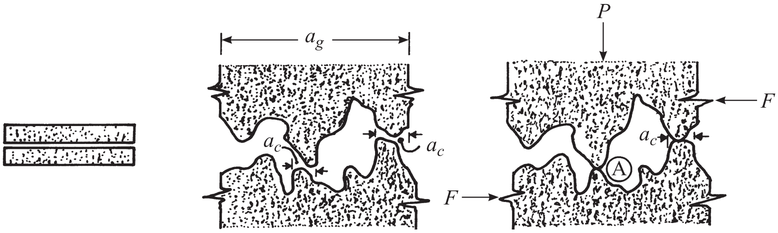

Figure 3.5. Frictional shearing resistance due to contact junctions (Leonards 1962). a) Macroscopic view, b) Microscopic view, c) Microscopic view of contact.

contact or friction bonds. Generally, friction is defined as solid friction between two surfaces with negligible moisture. Since the solid surfaces are generally not clean but masked by an adsorbed layer of hydrated ions, or contaminants, friction at these contact bonds will be different than solid friction. As the physico-chemical interaction between sand and water is very low, the resistance offered by contact bonds is not radically different due to the presence or absence of moisture, provided the response is not influenced by pore water pressure.

In the case of fine-grained soils, the micro-roughness along the basal planes of the clay particles is of the order of 10-100Å. Further, diffuse double layer of thickness greater than that of micro-roughness develops on these surfaces in an appropriate aqueous environment. This inhibits the apex contacts, making the short range forces insignificant. Of the long range forces, the coulomb forces of the electric double layer, repulsive in nature, are dominant over the van der Waals attractive forces. Accordingly, the long range forces can theoretically keep the fine-grained stable particle units separated, preventing them from being in direct contact as in the case of coarse-grained soils.

Net force between particles: As indicated earlier, there exist both attractive and repulsive forces between collodial clay particles in a dilute aqueous suspension containing electrolytes. The net force at a specific separation distance is the algebraic sum of the repulsive and attractive forces mobilized. The repulsive force per unit area can be computed as the difference in osmotic pressure midway between stable particle units relative to that in the equilibrium solution. The osmotic pressure difference depends directly on the difference in ion concentration. Apart from these forces of repulsion, there will be edge-to-face attraction and van der Waals attractive forces acting within particle clusters and as well as between them. It has been very thoroughly demonstrated by earlier investigators (Mitchell 1993) that repulsive forces are sensitive to changes in electrolyte concentration, cation valence, dielectric constant and pH, whereas the attractive forces are sensitive only to changes in the dielectric constant and temperature (Fig. 3.6). The net forces of interaction as a function of distance are schematically shown in Figure 3.6. These forces of interaction may be operative in a restricted sense within

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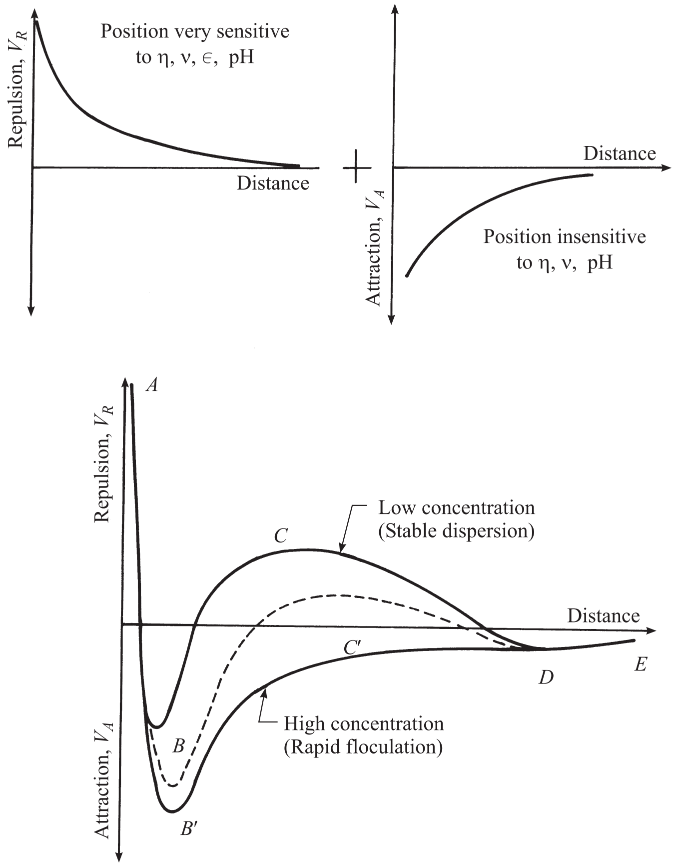

Figure 3.6. Schematic representation of energies of repulsion, attraction and net interaction energy for parallel plate model (Mitchell 1993).

stable particle units such as flocs but not between such units. A dilute suspension with flocs is one example. Concentration of particles or their aggregated units may be so low that the individual units might be separated by distances far greater than the range over which the repulsive and attractive forces operate. Interparticle forces cease to exist beyond interparticle separation distances of 300Å.

Although sands and clays have much in common as members of the same family of particulate materials, their differences are as important as their similarities. From the above discussions, it can be inferred that in the case of sands (coarse-grained soils), due to the micro-roughness of particles and low surface activity, the short range forces are dominant. On the contrary in clays (fine-grained soils), short range forces are operative only between the exposed ionic lattices (edge to face interaction) of clay particles and depressed double layers due to increase in ionic concentration. Such interactions promote the aggregation and growth of clusters towards

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stable units, both with and without external loading. Simultaneously long range forces would be operative between particle cluster units, which essentially resist the external loading. Another notable difference between sands and clays is in their susceptibility for volume changes with environment, independent of loading, such as in swelling and shrinkage. As such, it might be worthwhile to examine and analyze clay behaviour from considerations other than the conventional solid-solid contact model, which is wholly tenable only for coarse-grained soils devoid of any dominant physico-chemical interactions with the pore medium.

From the considerations of soil mineralogy and the interactions with other phases of the multiphase system, soil systems devoid of clay minerals, i.e. sands, can be regarded as non-interacting systems. On the other hand, reasoning based on the structure and mineralogy of clays and their interactions with water has established that all clay minerals have a tendency to adsorb water and/or exchangeable cations, from the fluid phase. The water held by a clay mass is essentially a balance between the urge of the clay minerals to suck in water and the tendency of applied pressure to squeeze out water. It has been observed that the degree of absorption and squeezing out of water depends upon the expanding and non-expanding clay minerals, a distinction which is only relative, without any inherent difference between them (Bolt 1956, Grim 1968). The engineering properties which clays can exhibit due to variation in mineralogy extends over a wide range. But the striking factor is that despite the wide differences that can exist between one clay and the other within a given range of water contents, they exhibit essentially the same mechanical behaviour from a qualitative point of view. Differences in mineralogy, adsorbed ions etc., manifest themselves as differences in quantitative behaviour. Although all fine-grained soils exhibit the phenomenon of consolidation, swelling and shrinkage, in soils with kaolinite clay mineral a smaller range of variations is observed than in those clays containing illite and montmorillonite clay minerals. This is mainly due to the low surface activity of kaolinite clay mineral. The physico-chemical interactions between the clay particles lead to the existence of both interparticle repulsive and attractive forces resulting in the separation of stable particle units preventing any direct contact between them, in the strict sense, to associate the measured shear strength to frictional properties derived from physical solid particle contact (including interlocking). Still, continuity in the clay-water system is maintained through the interacting adsorbed water layers and/or exchangeable cations. Hence, soil systems in which solid constituents are fully or partly clay minerals can be regarded as interacting soil systems. The differences in the specific characteristics of both non-interacting and interacting systems (Nagaraj 1981) have been indicated briefly in Table 3.2.

3.1.4 *State parameter of soils*

Since fine-grained soils with appreciable clay fractions are basically interacting particulate material, when an external load is applied to a soil-water system, there would be changes in the microfabric to mobilize interparticle and intercluster

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Table 3.2. Characteristics of particulate systems.

Non-interacting soil system (sand and silt)	Interacting soil system (clays)
Each phase has an independent continuous stress field	Independent continuous stress field in each of the multiphase is not tenable
The principle of superposition of coincident equilibrium stress fields is valid	Not tenable
In addition to stress fields in each phase an overall stress field can be assumed	The overall total stress field is the only acceptable physical stress field

forces to resist the stresses imposed. The dominant macro parameter to reflect the state of clay in its saturated condition is the water content or the corresponding void ratio. Hvorslev (1937), as early as 1937, recognized that for a saturated uncemented clay, a unique relation exists between the effective stress and the void ratio at equilibrium condition. Subsequently, Casagrande (1944), Rutledge (1947), Leonards (1953) and others showed that the void ratio at failure is of considerable significance when relating strength of clays to applied stresses while residual pore water pressures are zero. The elasto-plastic Cam-clay model developed by the Cambridge group to obtain the complete stress-strain response of clays for different loading conditions (Roscoe et al. 1958) clearly recognizes the need to consider the void ratio-pressure relation of clays, along with the consideration of the friction factor M as discussed earlier in Section 1.7.

Soils equilibrate to different void ratios as they are formed, loaded or compacted by a specified compactive effort. For example dry soils mixed with different water contents and compacted under different compactive forces result in different dry densities and hence different void ratios. While residual soils are formed due to weathering of rocks, soils with different void ratios with residual cementation result. But the simplest situation is in the formation of a dense soil system from the stage of sedimentation due to its own weight, without any cementation. A dilute suspension of soil particles (or their basic units, flocs) in water does not constitute the engineering material, soil. It would not be possible to ascribe a definite stable state to the soil. Only when a small but measurable effective stress is realized, at which stage the particles are brought into closer proximity with the interparticle forces operative so as to reach an equilibrium state to resist the applied loading. At this stage the clay particles and their aggregated units are arranged into a definite pattern, i.e. the microfabric. Consider an isotropic stress state, as applied to the soil. Obviously, all the clay particles would not arrange themselves in a parallel array as there is no preference for such an orientation in any direction. This would be a deviation from that required for direct application of the Gouy-Chapman theory for di-

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rect computation of the equilibrium condition at the micro level, and hence the extrapolation of the same considerations to the equilibrium state at the macro level. Hence an examination of the structural details at submicroscopic and microscopic level is inevitable for understanding soil behaviour and for the development of a framework for interpretation, and, wherever possible, to develop a simple method to assess the engineering properties of in-situ soils.

Although sophisticated tools like optical, transmission and electron microscopes, the X-ray diffraction analyzer and the mercury intrusion porosimeter have been at the disposal of technologists for quite sometime, due to the delay in development of refined techniques to prepare test specimens to allow these to be used, micro structural details of clays could not be easily generated. In the recent past, after many trials and extensive experimentation, it has been possible to develop reliable techniques to prepare samples. Thus, clay microstructural investigation can be undertaken more confidently than in the 1970s and 1980s (Nagaraj 1993). Of the different techniques, mercury intrusion porosimetry, despite limitations, provides quantitative information about microstructural details, facilitating relative comparisons between different systems. There is very little published information on microstructure of clays not influenced by stress history. Pore size distribution data published by Griffiths & Joshi (1989) satisfies the above requirements and hence is used for further analyses. The data consists of pore size distributions of four clays with initial void ratios corresponding to their respective liquid limit water contents and hence devoid of any stress history effects (Fig. 3.7).

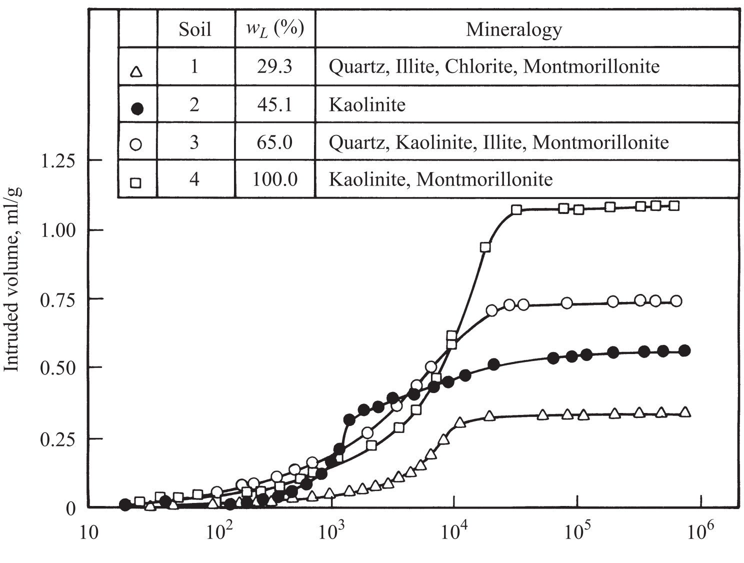

Figure 3.7. Pore size distribution of four clays at their liquid limit state (Griffiths & Joshi 1989).

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The data show that there are pores as large as 10,000Å and that the volume of pores larger than 300Å amounts to nearly 90 to 95% of the total void volume of the soil. No abrupt transition in pore size was noticed. Pores of different sizes in the range between 300 and 10000Å are present. We now examine what type of compatible microstructure can be formed under induced interparticle forces as well as to encompass such wide variations in pore sizes. The stable clusters formed in different electrolytic environments during deposition and sediment formation can be considered to account for pore volume of the order of only 3 to 5% of the total volume with the pore size in the lower range. Since net force is likely to be attractive, these clusters are stable even in the absence of external loading. If such stable units are in close proximity in the range of separation distances of 100 to 300Å, the net force of interaction would likely to be repulsive between them. The net force of repulsion, $(R - A)$, between such units would create an osmotic suction on adjacent fluid equal in magnitude to the isotropic mean effective stress p'.

When pore fluid is subjected to such suction pressure at innumerable points, due to several pairs of interacting particle units, a spherical pore can be formed as depicted in Figure 3.8. The maximum size of the pore is also dictated by the applied suction from surface tension governed by the equation $p = 2T/R$. Hence large size pores are compatible with small size pores, both developing the same order of internal resistance so as to maintain internal static equilibrium with boundary loading.

Micro-pores of the entire range, from less than 20 to greater than 10000Å, in saturated, uncemented clays by the sedimentation and densification process, together with double layer interactions restricted to less than 300Å, exist in clays. This, when examined with the earlier clay microstructural postulations (Casagrande 1932a) can possibly be explained by the following three levels of pores constituting the clay microstructure (Nagaraj et al. 1990, Mitchell 1993).

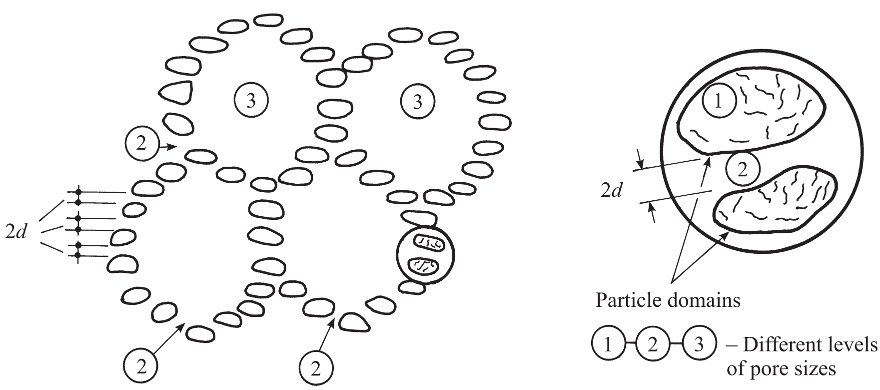

Figure 3.8. Possible micro-fabric of uncemented clays (Nagaraj et al. 1994).

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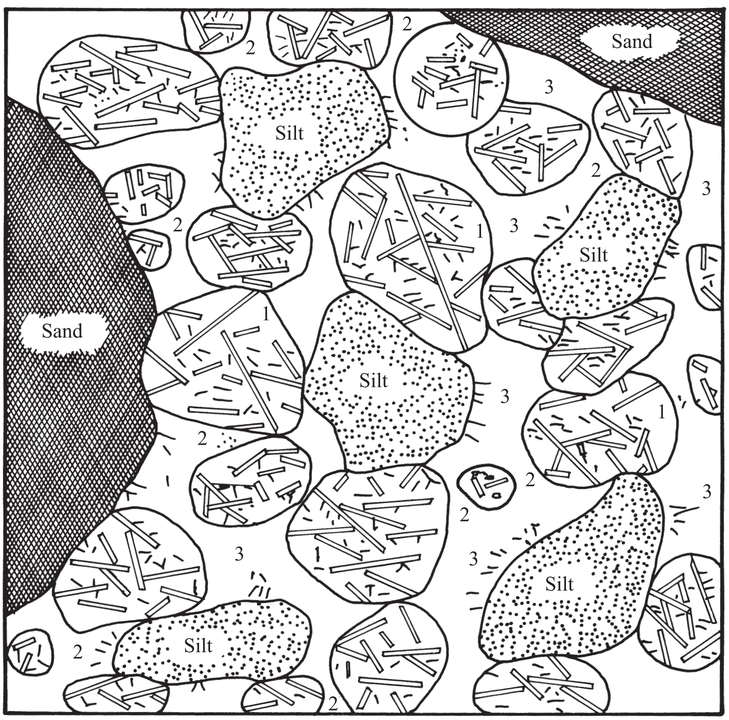
1. Intra-aggregate (or cluster) pores with sizes less than about 20Å between the individual platelets within a cluster. Particles within a cluster would have crossed the repulsive energy barrier and approach distances at which the net interaction force is one of attraction so that they are stable units.
2. Inter-aggregate pores between two interacting aggregates (where double layer interactions prevail) of sizes greater than 20Å but less than 300Å depending on the applied equilibrium pressure.
3. Inter-aggregate large enclosed pores held within a group of clusters by surface tension, with sizes far greater than 300Å.

Figure 3.9 is a schematic representation of the modified diagram of the microstructure of fine-grained soils proposed by Casagrande (1932a) with the relative disposition of different fabric units numbered 1 to 3. The clusters of clay particles are circled. It can also be noticed that direct contact between sand and silt particles is inhibited by clay clusters. A similar microstructural picture, comprising unequal pores, (Fig. 3.10) was proposed by Olsen (1962), to interpret experimental data on hydraulic conductivity.



1. Intra aggregate pores
2. Inter aggregate pores
3. Large enclosed pores within a group of aggregates

Figure 3.9. Modified diagram of Casagrande's schematic representation of clay micro-structure (Nagaraj et al. 1990).

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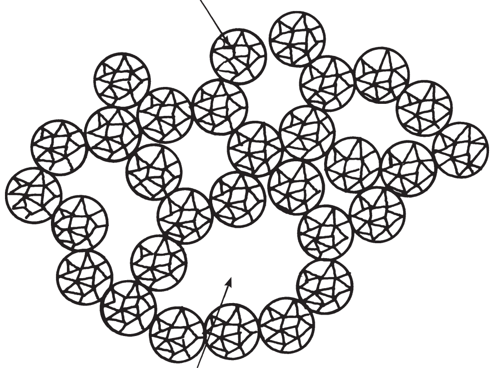

Figure 3.10. Idealized unequal clay pore structure (Olsen 1962).

3.1.5 *Reference state parameter*

The above picture of the clay micro-fabric at the liquid limit water content of clays, although it provides details for a clay, does not provide any inkling about the possibility of having a unique state for all fine-grained soils. The need for probing for such a state arises when the possibilities of unifying the soil behaviour and developing a basic framework are examined. The two possible avenues to probe are to define the relative behaviour of soils with reference to a particular stress level, or to identify a common reference state of the soils. Since the interparticle forces operative between clay particles or their aggregates are strongly influenced by separation distances it might be advantageous to probe for a unifying state for all homogeneous saturated clays. Although the void ratio is an important state parameter, it is not directly helpful, since the behaviour of two different clays at the same void ratio can be distinctly different, or at different void ratios of two clays the same response can be observed. Hence examination may still have to be at the micro-structural level than at the macrolevel. The logical step would then be to re-examine the pore size distribution data already analyzed above.

With the possibility of having three distinct levels of pores in the clay micro-fabric, it suggests that the ideal pore size distribution curve will be as shown in Figure 3.11. Even with the combinations of inherently different clay minerals present in the four clays considered, plots show the same trend (Fig. 3.12). In this plot, the pore size distribution data of four clays (Fig. 3.10) are replotted in terms of intruded pore volume per unit volume of the soil, instead of per unit weight of dry soil, so that the total pore volume of different clays are normalized for comparison. It can be seen that the relative distributions are of similar pattern for all four soils examined. This suggests that the microfabric which reflects the dis-

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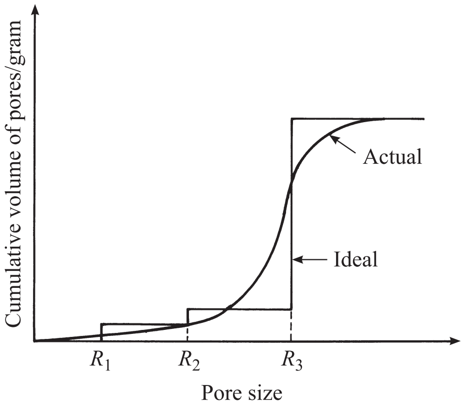

Figure 3.11. Idealized pore size distribution.

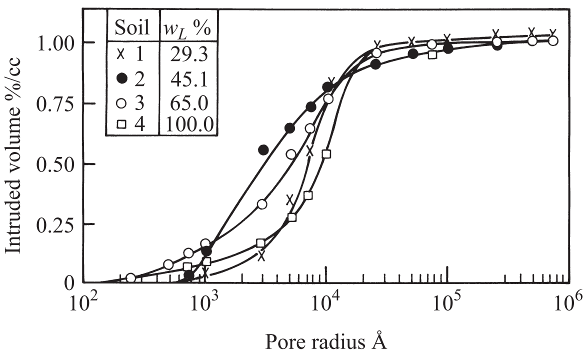

Figure 3.12. Normalized intruded volume versus pore size distribution data of Figure 3.7 (Nagaraj et al. 1990).

tribution of pores, i.e. intra-aggregate, inter-aggregate pores and large pores held within a group of clusters in a unit volume, would be of similar pattern for different clays. But the number of such groups have to be commensurate with the increase in water holding capacities, i.e. liquid limit water contents, to reflect different water contents of clays at that same state of clays. Further strong evidence to suggest that the microfabric is of the same pattern comes from the analysis of permeability data of clays at their corresponding liquid limit state. Although the water contents and void ratios at the liquid limit state for different clays tested vary over a wide range, the permeability coefficient is of the same order (Nagaraj et al. 1991, 1993, Mitchell 1993) (Table 3.3). Figure 3.13 shows the rate of flow versus hydraulic gradient for the clays tested. The slope of the linear plots are the *k* values as indicated in Table 3.3. This suggests that the effective large micro-pore space available for flow might be of the same order, despite the void ratios varying over sevenfold. It is presumed that no recognizable flow takes place through the submicroscopic void spaces within the clusters. This assumption is justifiable since most of water within the cluster is under the influence of double layer interactions.

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Table 3.3. Hydraulic conductivity at the liquid limit state of several clays (Nagaraj et al. 1991, 1993, Mitchell 1993).

Clays tested	Liquid limit, w_L (%)	Void ratio at liquid limit	Hydraulic conductivity (in 10^{-7} cm/sec)
Red soil	50	1.325	2.63
Brown soil	62	1.643	2.83
Black cotton soil	84	2.226	2.32
Marine soil	106	2.798	2.56
Air dried marine soil	84	2.234	2.42
Oven dried marine soil	60	1.644	2.63
Bentonite	330	9.240	1.28
Bentonite + sand	215	5.910	2.65

For the formation of such a self-supporting identical pattern of microfabric, the forces of interaction between the stable units of clay clusters need be of the same order for the clays examined. If this is true, the amount of water held by micro-pores per unit volume has to be of the same order. The water held within the stable clay cluster units, which is normally only of the order of 3-5% of unit volume, is neglected. For the equilibrium requirement, the same order of matrix suction should prevail in all the zones of the clay-water system at their liquid limit state. Wroth (1979), on the basis of earlier investigations and critical state concepts (Russell & Mickle 1970, Wroth & Wood 1978, Whyte 1982), has indicated that all fine-grained soils tend to equilibrate from the same high initial water content state to their respective liquid limit water contents at an applied suction of the order of 6 kPa. For this condition to prevail, for different clays at their liquid limit state, the same order of physico-chemical potential should be available from physico-chemical interactions. In order to visualize the possibility of this situation it would be worthwhile to examine the soil solids and water. The weight of solid particles in any unit volume would adjust itself such that the surface area for interaction would be of the same order to result in the same order of physico-chemical potential for all sheet clay minerals. Hence, the volume of large micro-pores enclosed by stable clay clusters will also be of the same order (Fig. 3.14). It can be seen that the clay surface area contribution per unit volume is the same in either case, with distinct variations in weight contributions due to massive (such as Kaolinite) and non-massive (such as Smectite) clay minerals. The same order of permeability implies the same order of pore space for flow and hence the overall clay micro-fabric is of the same pattern. Since this stable micro-fabric is associated with a suction of the order of 6 kPa, which can be regarded as effective stress, according to the effective stress principle shear strength should be of the same order. The shearing resistance, C_{uL}, of the soils at their liquid limit state water contents varies between 36 and 159 as measured by laboratory vane, has been reported by Federico (1983). The measured values fall within limits of 1.7 to 2.8 kPa with most of the values around 2.3 kPa.

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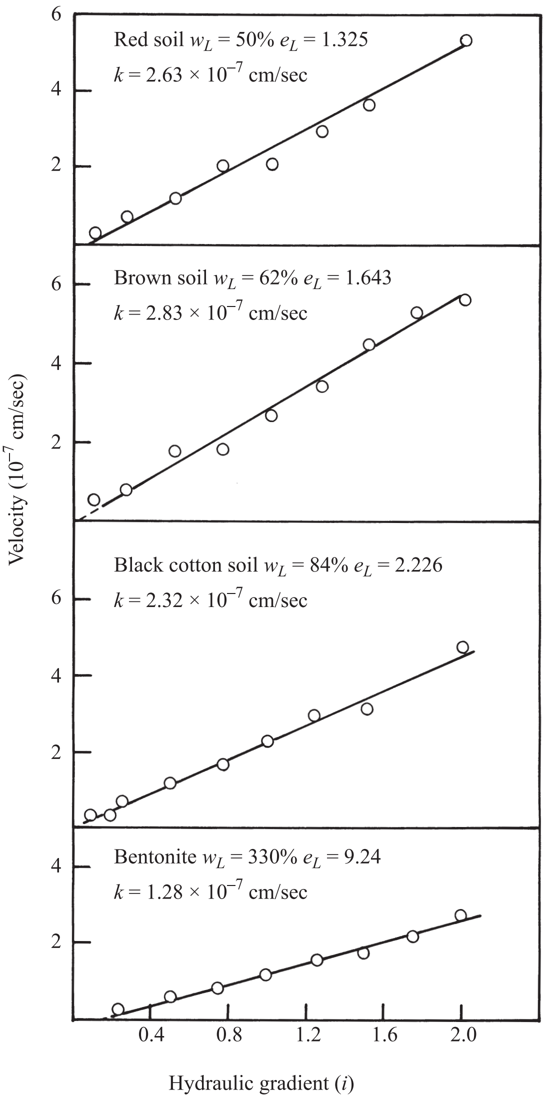

Figure 3.13. Rate of flow versus hydraulic gradient.

In retrospect, the approximate equal strengths, pore water suction, and hydraulic conductivity for clays with sheet clay minerals can be attributed to these facts.

1. The stable individual clay clusters are the basic units that interact like single particles to develop the same shear strength.
2. The average adsorbed water layer thickness is about the same order for the external surfaces of all interacting units.
3. The forces of interaction between interacting units through their adsorbed water layers correspond to a pore water suction of 6 kPa.
4. The average size of intercluster pores is of the same order for all clays. The number of units as a whole, with clay clusters along with intercluster pores,

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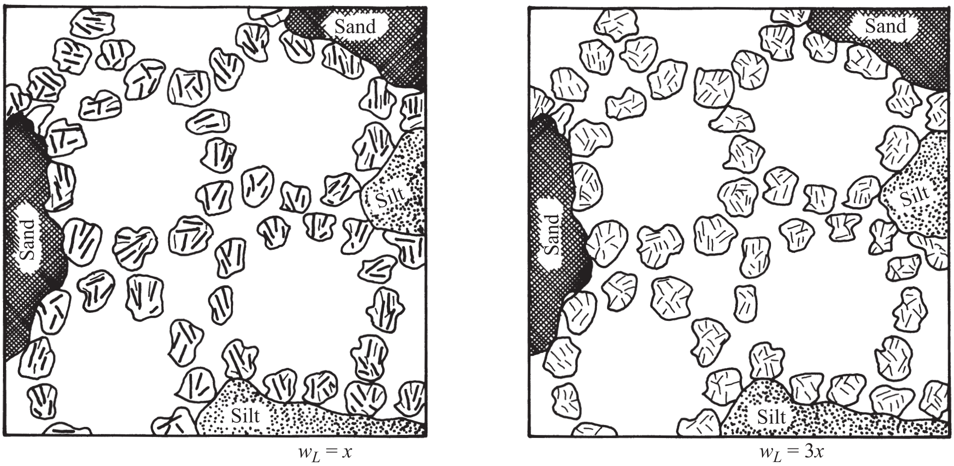

Figure 3.14. Schematic picture of the same clay micro-fabric for two clays with different clay minerals and the same coarse fraction (Nagaraj 1995).

would increase with the increase in specific surface so that the required amount of water to satisfy the conditions at the liquid limit can be held by the system.

The above observations and inferences suggest that state of clay at its liquid limits has definite attributes to be a reference state, provided that the microfabric at this state is not prone to any mechanical disturbance. Since the liquid limit state is attributed to an internal stress field and is only a function of the water content, it remains the same and hence is not prone to any mechanical disturbance. Changes in the micro-fabric would invaribaly be there as changes in the water content are brought about by drying or loading. The discrete element method for simulation of the fabric of clay, considering interparticle forces, developed by Anandarajah (1995) cannot be directly used to critically examine that postulated from poresize distribution data.

Cohesive membrane concept: So far, discussions regarding double layer interactions and their effects on water-holding capacities are only with respect to clay minerals. It is needless to stress that natural soils, apart from clay fractions which contain coarser fractions, such as sand and silt, do not contribute to any recognizable physico-chemical interactions with pore fluid. It has long been recognized that the clay fraction controls the behaviour of a soil even if its percentage is appreciably lower than that of coarse constituents not in excess of the volume of pores formed by packing. As such it would be necessary to examine how to assess the magnitude of the water holding capacities of a soil, i.e. liquid limit water contents, containing different fractions of coarser material than 425 μm fractions.

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The specific surface of clay particles is far higher than that of the coarse constituents of the soil. For example in 100 gms of soil with only 30 gms of clay minerals the surface area available can be in the range of 600 to 800 m^2/30 gms whereas 70 gms of coarse fraction would not even account for one to two percent of the above value (6-12 m^2/70 gms). Hence clay with water can generate adequate clay matrix to provide a cohesive membrane around coarse particles so as to inhibit direct contact and interference between them. No doubt this condition would not prevail as the size of the coarse particles are such that the clay matrix available is not adequate to fill the pore spaces, due to the packing of coarse particles. At that stage it would not be possible to determine the liquid limit of soil by conventional tests. In fact, even for the determination of the liquid limit water content of soil the fraction of soil coarser than 425 μm is removed. The influence of that fraction of soil between 425 μm to 2 mm is in-built in the sense that clay matrix available would provide the needed cohesive membrane to inhibit direct interference between coarse particles. What is required is a method to take into account all the soil solid constituents, including that fraction coarser than 425 μm, in assessing the gross water-holding capacity of natural soils in terms of a modified liquid limit, which would be an appropriate parameter in the assessment of soil behaviour.

Liquid limit tests do not directly provide water contents corresponding to the liquid limit state. Flow lines are obtained from the cone penetration test between the water content and depth of penetration (Fig. 3.15). The water content at 20 mm penetration is reckoned as the liquid limit water content, at a specific value of shearing resistance of the soil. Along the flow line, as the water content changes, the shearing resistance also changes. It has been shown that the generalization of flow lines is possible by the normalization of water contents by the respective liquid limit water contents along each of the flow lines. From the basic considerations of physico-chemical interactions it has been shown (Nagaraj & Jayadeva 1981) that the generalization with respect to water content at liquid limit as reference is possible. The effects of the specific surface (m^2/gm) and associated physico-chemical factors are taken care of by normalization. Along the normalized flow line, different combinations of water content and liquid limit of soils with different combinations of non-clay and clay fractions can yield the same normalized value at a specific value of penetration, reflecting a particular value of shearing resistance. We now examine what happens when a coarse fraction greater than 425 μm size is present. Examination of flow lines generated by experiments using coarse particles of sand and glass which have a twofold variation in their solid frictional characteristics (Fig. 3.15) reveal that the presence of a coarse fraction does not alter the basic nature of the interaction, resulting in the same mode of generalization. This implies that the soil fraction other than that of clay minerals to some extent dilutes the behaviour without affecting the trend.

Hence, it is possible that the above postulations can be extended to natural soils containing non-clay fractions by accounting for a reduction in physico-chemical

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potential appropriately. From the experimental observation of the linear variation of the liquid limit as the coarse fraction content varies (Fig. 3.16), the modified liquid limit can be computed from the relation:

$$w_{L(\mathrm{mod})} = w_L\,(1 - C/100) \tag{3.1}$$

where w_L = liquid limit determined for soil passing 425 μm

C = fraction coarser than 425 μm in percent.

However there would be a limit on the percentage and size of coarse particles, for the cohesive matrix formed is adequate to satisfy the requirements of a cohesive membrane, to subdue the interference effects of the coarse fraction. It is tentatively suggested that clay-water interactions can be regarded as governing soil behaviour if the computed modified liquid limit is at least 30%.

It has been observed by many earlier investigators (Skempton 1953, Seed et al. 1964, Nagaraj & Jayadeva 1981, and several others) that the composite effects of surface activity of fine-grained soils and the physico-chemical properties of the interstitial pore fluids are reflected to a large extent in their liquid limit water contents. As such, the liquid limit state can form a reference base in relation to

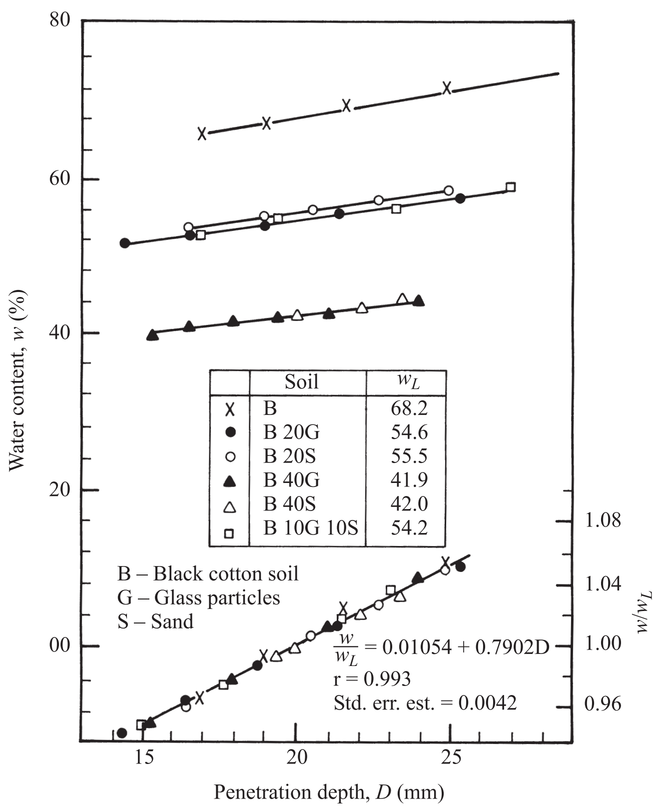

Figure 3.15. Flow lines of soil in the presence of a coarse fraction of sand for glass particles (Srinivasa Murthy et al. 1987).

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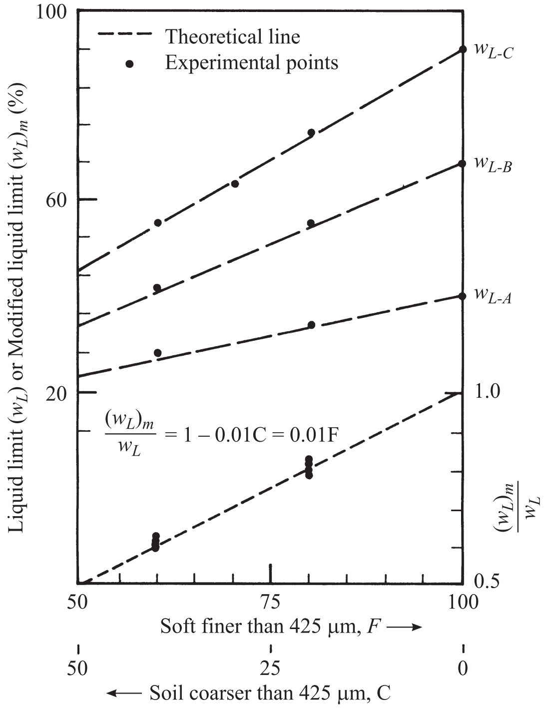

Figure 3.16. Modified liquid limit with different percentages of coarse fractions (Srinivasa Murthy et al. 1987).

which fine-grained soil behaviour can be interpreted, analyzed and possibly be assessed. In fact, Terzaghi (1926), intuitively stated: 'Two soils of similar geological origin will exhibit identical engineering properties if their liquid limits (index properties) are identical'.

3.1.6 *Critical appraisal of index parameters*

So far, from the above discussions it has been amply substantiated that the liquid limit of fine-grained soils has many attributes to be a reference state parameter for analysis of soil behaviour. Apart from the liquid limit, a few other index parameters, such as the plastic limit w_p, and colloidal activity A, are also employed to correlate engineering properties of soils. Even derived parameters such as the plasticity index I_p, and the liquidity index I_L, have also found their place in engineering property assessments. We will now examine what the above parameters mean micromechanistically, and consequently their potential for use as the reference state.

Despite the inherent deficiencies of the test to determine the plastic limit, its role can be recognized for the purpose of identifying indirectly the presence of clay minerals distinctly different from sheet minerals. For the usual inorganic clays with sheet clay minerals, the plasticity index would plot along a path nearly parallel and close to Casagrande's A-line. But while non-sheet minerals like halloysite and allophanes are in clays, the plastic limits would be very high (usual

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values are like w_L = 140%, w_P = 80%) and hence points would plot very much below the A-line. Recognition of such distinctions would enable us to identify the applicability or otherwise of relations developed for assessment of inorganic soil behaviour. It has been observed Nagaraj et al. (1994) that even for the set of data falling along a defined path below the A-line the functional form of relations between different parameters is the same, but with different constants.

The methods employed to determine the plastic limit by shaping wet clay to balls and threads with further manipulations are primarily prone to personal impressions. This aspect has been discussed in detail in Section 1.3.1. Further, it has been found experimentally that the plastic limit water contents for different soils cannot be related to a unique state of stress or shear strength to the same narrow range of acceptability as the stress state at liquid limit water contents. If the plastic limit is redefined as the water content corresponding to a hundredfold increase in shearing resistance (Wroth & Wood 1978) and the appropriate cone penetration methods suggested by Belviso et al. (1985), Watsi & Bezirci (1986) and Stone & Phan (1995) became acceptable as routine tests for determination of the plastic limit, then it would be a parameter for use as a reference state. Despite this situation, there exists a functional relationship between I_p and w_L of the form

$$I_p = A\,(w_L - B) \tag{3.2}$$

for a large set of data of 520 soils (Nagaraj & Jayadeva, 1983) and subsequently confirmed by Paul (1984) for a still larger set of data of 1100 soils, although the constants A and B varied marginally. This difference might be due to the inherent deficiencies in the determination of the plastic limit by the conventional test. This might be the reason that Casagrande (1948) did not advance the A-line as a functional relationship between the plasticity index and liquid limit. It is likely that using the plastic limit water contents arrived from the stress state definition, would result in a better defined relation of the form as indicated above (Equation 3.2).

The inherent disadvantages in the determination of the plastic limit as done at present in routine investigations also apply to all the derived parameters involving plastic limit, plasticity index, liquidity index and colloidal activity.

The plasticity index represents the range of water contents over which a defined level of change in shear strength takes place. The liquidity index, which involves the water content in addition to the liquid and plastic limit water contents, has a relation of the form

$$I_L = \frac{w - w_P}{w_L - w_P} \tag{3.3}$$

This can be expressed in terms of only two independent parameters w and w_L, since w_p is a function of w_L. For the data of natural water contents and consistency limits of Komarnik & David (1969) the relation between I_L and w/w_L is shown (Fig. 3.17) to be of the form (Srinivasa Murthy et al. 1986):

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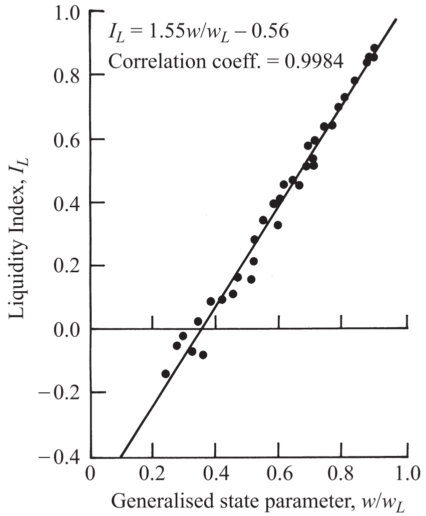

Figure 3.17. Relation between liquidity index and w/w_L parameter (Srinivasa Murthy et al. 1986).

$$I_L = 1.55\frac{w}{w_L} - 0.56 \tag{3.4}$$

Hence, the liquid limit itself would possibly be adequate as a reference state.

Skempton (1953) observed a linear relationship between the plasticity index and percent clay fraction for different clays (Fig. 3.18). The slope of variation of I_P with clay fraction is the colloidal activity of that clay. When the the effect of a clay fraction is incorporated in the modified liquid limit, according to Equation 3.1, as dilution by a soil fraction coarser than the clay fraction, the plasticity index versus liquid limit bears a unique relationship (Fig. 3.19). Clays of different activity would fall in different zones along the line. This implies that variation of I_P and w_L would not only take care of the type of clay but also the clay fraction in it. If it is accepted that for sheet clay minerals there exists a functional relation between I_P and w_L (Nagaraj & Jayadeva 1983), then the liquid limit itself reflects indirectly the colloidal activity of the soil (Pandian & Nagaraj 1990).

So far the discussions clearly recognizes the possibility of considering the liquid limit of fine-grained soils as a reference state in the analysis and assessment of soil behaviour. At present the determination of consistency limits serves the primary need for classification of soils and their use in empirical relations for engineering property assessments. If this parameter has to play a more significant role than at present, it merits examination as to how far additional reliability and quickness can be built into the already simple method of determination.

3.1.7 *Liquid limit determination-further simplified*

In recent years the cone penetration method to determine liquid limit water content has gained in importance and acceptance. Wroth (1979) as early as 1979 pro-

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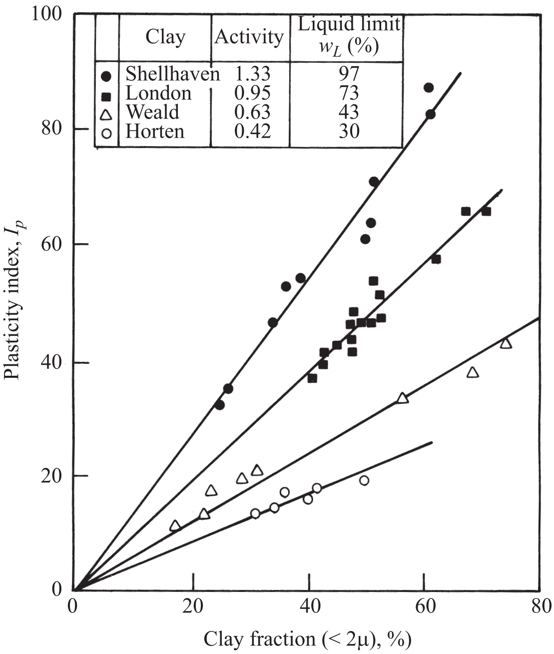

Figure 3.18. Plasticity index versus clay fraction (Skempton 1953).

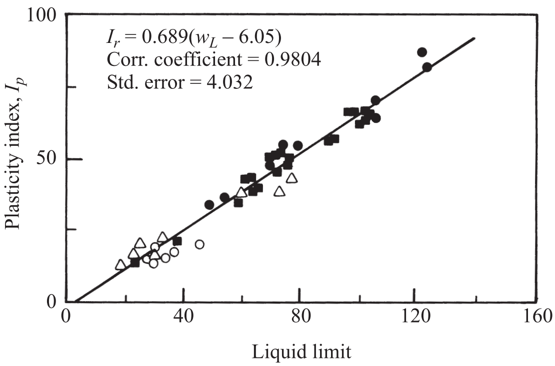

Figure 3.19. Plasticity index versus liquid limit – replotting of Skempton's data (Pandian & Nagaraj 1990).

vided adequate supporting evidence in favour of this method, as a simple strength test, instead of the more elaborate percussion test to assess the same parameter. Recent investigation (Nagaraj et al. 1987) has revealed two more specific advantages of this method.

1. The liquid limit of soils can be determined almost instantaneously, and
2. with two trials, the liquid limit water content value can be reconfirmed.

In the cone penetration method a metallic cup is used to transfer the soil paste for the penetration test. Since this cup is of standard volume, it is handy in determining the bulk density of the soil in various trials of the penetration test. In the range of water contents close to the liquid limit state, the amount of soil paste transferred to the cup is a function of both water content and the liquid limit of the soil

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and is operator independent. In the range of water contents around the liquid limit it is further shown (Nagaraj et al. 1987) that the degree of saturation of the soil in the cup is around 98%.

From the above observations the water contents at different penetrations can be estimated. The errors associated with the assumption of a value for specific gravity of the soil in the range of 2.65 to 2.8, for calculation of the water content from the bulk density of saturated soil paste, are well within the acceptable levels of accuracy. This method of assessment of water content also permits one to independently check the values obtained by oven drying or by using a microwave oven.

To have a cross check on the value of the liquid limit water content, without waiting for the results of oven drying, two penetration tests on soil paste with different water contents in the close range of penetration of 20 mm can be conducted, with water contents for each of the tests being computed from the bulk density measurements. The one point method discussed below would allow calculation of liquid limit water contents for both trials, thereby permitting examination of the accuracy of the value of the liquid limit of the soil obtained.

A rational basis to determine liquid limit by a one point method has been advanced by Nagaraj & Jayadeva (1981). The following two relationships have been developed to determine water content at the liquid limit from a cone penetrometer

$$\frac{w_L}{w} = \frac{1}{0.77 \log D} \tag{3.5}$$

$$\frac{w_L}{w} = \frac{1}{0.6 + 0.017D} \tag{3.6}$$

where D is the depth of cone penetration in mm.

The work of Federico (1983) has shown that for many soils of Southerm Italy covering a wide range of plasticity, the regression equation of the straight line of w/w_L against log δ is (see Fig. 3.20):

$$\left\{\frac{w}{w_L}\right\} = 0.102 + 0.688 \log\delta \tag{3.7}$$

with $r^2 = 0.959$, where δ (delta) the penetration is in mm. Equation (3.7) is very similar to the monomial relation of Equation (3.5).

$$\left\{\frac{w}{w_L}\right\} = 0.77 \log D \tag{3.8}$$

where D is the penetration written as in Equation (3.5). Both the above relations satisfy the condition $w/w_L = 1$ corresponding to a penetration of 20 mm.

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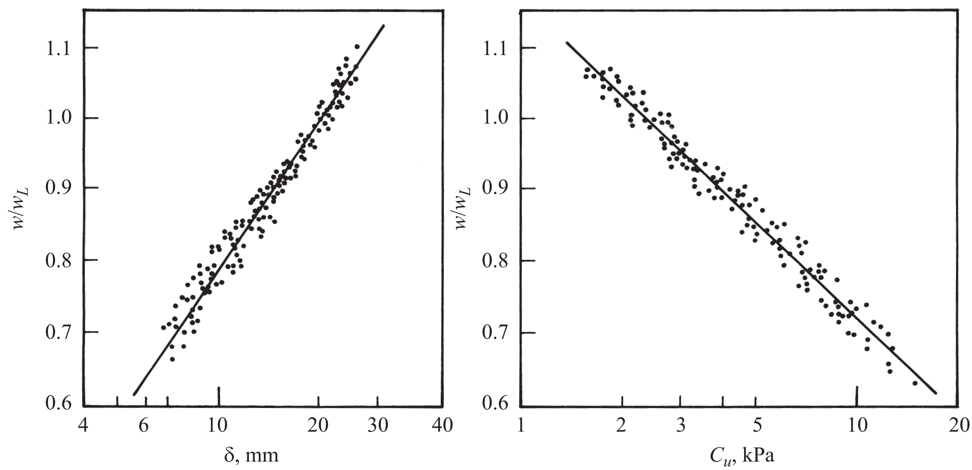

Figure 3.20. w/w_L versus cone penetration or undrained strength (Federico 1983).

3.2 INTRINSIC STATE LINE

So far, from the fundamentals of soil behaviour and with the possibility of advancing the micro-mechanistic axioms to the macro level, the liquid limit water content has been identified as an engineering reference state parameter for analysis of fine grained soil behaviour. Although the liquid limit state reflects an equilibrium state at a specific stress state (suction), it is needless to state that in-situ soils encountered, by sheer overburden would have a higher stress level than that at the liquid limit. Which is why it is necessary to examine the state of soils as they are monotonically stressed to higher levels.

The simplest clay-water system which has no stress history and time effects and is devoid of any cementation is that at the liquid limit water content. The response of such systems were recognized by Hvorslev (1937) as early as 1937 in the development of soil mechanics. In retrospect, it has been categorically stated that there exists, for a saturated clay, a unique relationship between effective stress and void ratio at the equilibrium condition when compressed from its liquid limit water content. This has been recognized as the normally consolidated state of the soil. It is equally well known that such stress-state lines are different for different fine-grained soils, thereby implying that at the same effective stress level saturated soils equilibrate to different void ratios. It merits examination of what is common between different soils, if any, such that a unifying attempt can be made similar to the identification of the liquid limit state as a reference state.

Reverting back to Figure 2.5, regarding the compression paths of clays compressed from their respective liquid limit water contents, it can be seen that the soil with a higher liquid limit has a steeper slope than the one with a lower liquid

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limit water content. Although at a specific consolidation stress the void ratios reached are distinctly different, it is interesting to observe that the change in void ratio over a pressure increment (i.e., 50-200 kPa) is proportional to the void ratio corresponding to the liquid limit water content of that soil. In other words:

$$\left\{\frac{\Delta e_1}{e_{L1}}\right\} = \left\{\frac{\Delta e_2}{e_{L2}}\right\} = \left\{\frac{\Delta e_3}{e_{L3}}\right\} = \text{Constant} \tag{3.9}$$

implying that proportional changes are induced by applied stresses. Since at the liquid limit state the microstructure is of the same pattern, although at the macro level changes in the void ratio are different, the micro-structural changes are likely to be constant for different soils over an effective stress increment. To presume that e/e_L term reflects the microstructure at any stress level and its changes as effective stress changes, it would be necessary to examine the effective stress from inter-particle force considerations. It would also be necessary to check the quantitative assessment of micro-structural data for possible substantiation of the above hypotheses.

3.2.1 *Effective stress in fine grained soils*

The simplest model of clay particle configuration is to consider interparticle forces between two clay particles in parallel array in equilibrium under an external applied stress of σ. This parallel plate model has to be such that it should accommodate possible short and long range forces for different distances between them, for mobilization of forces and their changes upon incremental loading consistent with the clay micro-fabric model already postulated.

On the premise that

1. the soil behaviour is governed primarily by inter-granular friction at the points of contact, and
2. for combining all external and internal forces, the principle of superposition is valid,

Lambe (1960) formulated the following equilibrium based on the laws of statics

$$\sigma = \sigma_m a_m + u_w a_w + u_a a_a + (R - A) \tag{3.10}$$

σ is the total external force per unit area, σ_m is the mineral to mineral contact stress, u_a is the pore air pressure, u_w is the pore water pressure, $(R - A)$ is the net interacting force on unit area of total cross-section, a_m, a_a, a_w are the fractions of total area in contact with each phase of solid, air, and water respectively.

In the later discussions Lambe (1960) visualizes two circumstances for a mechanistic interpretation of conventional effective stress (σ-u).

1. In a highly plastic saturated dispersed clay $a_w = 1$, $a_m = 0$, the effective stress would be

$$\sigma - u = R - A \tag{3.11}$$

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2. For conditions of mineral to mineral contact and with the contribution of interparticle forces being negligible, the intergranular stress would be

$$\sigma - u = \sigma_m a_m \tag{3.12}$$

Still, the above distinction did not persist for long, mainly due to the fact that most of the natural soils consist of far higher percentages of soil constituents which are coarser than clay sized fractions. The modifications to the effective stress relations for fine-grained soils, within the framework of the well established intergranular friction model, attempted by several investigators, are critically reviewed elsewhere (Nagaraj 1981, 1995).

Based on the premise that even in fine-grained soils the contact stress between solid particles is the effective stress and the principle of superposition is valid for all stress components, Sridharan (1968) proposed the equilibrium equation for saturated clays in the form (Equation 3.13).

$$C = \sigma_m\, a_m = \sigma - u\, a_W - (R - A) \tag{3.13}$$

where: C is the effective contact stress, σ is the externally applied pressure per unit area, u is the pore water pressure and $(R - A)$ is the net interacting force on a unit area of the total cross section.

The contact stress C has been tacitly assumed to govern the compressibility and shear strength of saturated fine-grained soils. Sridharan & Rao (1971, 1973, 1979) have discussed the same hypotheses by the analysis of diagnostic experimental data generated for this purpose. The substantiations have been based on the observed opposite trends in the volume change response of the soil in changed physico-chemical environments, i.e. different pore fluids such as acetone, benzene and methanol. It has been shown elsewhere (Nagaraj 1992, Nagaraj et al. 1994) that these are not the diagnostic experiments to prove or otherwise the existence of contact between the particles in clays. In an interacting system, change of pore fluid implies that it constitutes an altogether different system. Hence comparison of the similarity in responses of different clays with different pore fluids is not tenable. In the equilibrium Equation (3.13), u, C, R and A are all reactions to the applied stress. While u can be an independent variable, and the other components are dependent only on the mutual interactions between clay particles and pore fluid, they can be grouped together along with C on one side of equilibrium equation as:

$$C + (R - A) = \sigma - u_w$$

In a detailed discussion on the transmission of force through soil, Lambe & Whitman (1969) have shown that a stress of the order of 5500 kg/cm^2 is required to squeeze out the adsorbed water completely between the two interacting clay particles so that C is operative. Hence, it may be appropriate to consider Equation (3.11) itself to characterize the behaviour of fine-grained soils.

More specifically, the effective stress consideration is to recognize the soil system with appropriate predominance of contact junctions or surface forces due

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to clay water interactions as distinctly different in the analysis of soil behaviour (Nagaraj 1993, Nagaraj et al. 1994) viz.

1. Non-interacting particulate soils (sands) with negligible clay minerals

$$\sigma' = \sigma a_m = \sigma - u \tag{3.14}$$

2. Interacting particulate soils (fine-grained soils with a soil fraction containing clay minerals)

$$\sigma' = (R - A) = \sigma - u \tag{3.15}$$

where σ′ is the effective stress in the conventional sense.

The mode of effective stress consideration in the case of non-interacting particulate materials needs no substantiation. The relation cited for interacting particulate materials (Equation 3.15) does not violate the conditions stipulated for interacting particulate materials cited in Table 3.1. Further, it has been shown by Bloch (1978) that in an interacting system such as a clay-water system, in a partly saturated state arrived at by monotonic loading, consideration of independent stress fields for different phases and their superposition to get the total response is not physically tenable. Extending the same logic to saturated systems, although pore water pressure is an independent stress field, the mobilized interparticle forces are only the consequences of the conventional effective stress variations. Accordingly, in the case of clay (interacting systems) the long range forces (internally mobilized induced forces) balance the externally applied stresses as the pore water pressure dissipates progressively, changing the void ratio (state) of the system. Manifestations of interparticle forces due to changes in the stress, as the pore water pressure dissipates, is essentially the reaction of the soil system dictated by its physico-chemical factors. These postulations are on the premise that the stresses to be transferred are not of such magnitude as to squeeze out the adsorbed water layer, so that particle to particle contact stress occurs. Hence the particle interactions are assumed to be only through adsorbed water layers arising due to clay-water interactions. In the same context, density, water content and structure of the clay-water system itself to reflect the response of the system to the effective stress change as the pore water pressure dissipates has been critically examined by Mikasa (1977) as early as 1977. The implications of such considerations are that the effective stress mobilized as the pore water pressure dissipates, and the change it induces in the state of the clay-water system (void ratio) due to changes in interparticle forces, can be interrelated (Equation 3.15). This circumvents the actual formidable task of computation of interparticle forces of a clay-water system.

3.2.2 *Development of intrinsic state-effective stress relation*

In a clay-water system, wherein the stress transfer is assumed to be through the interacting fluid phase as the pore water pressure dissipates, the soil state can be assumed to be dictated by the requirement of equilibrium between long range

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forces $(R-A)$, mobilized between the interacting units and the externally applied stress. Accordingly, from the Gouy-Chapman diffuse double-layer theory, based on the significant finding by Sridharan & Jayadeva (1982), for the three basic sheet clay minerals kaolinite, illite and montmorillonite, for the assumed parallel plate model, d versus $\log(R-A)$ is practically of the same order for the same physico-chemical environment. Consequently, the average adsorbed water layer thickness is about the same for all clay particle surfaces. Hence, the equilibrium condition can be expressed by a relation of the form:

$$d = a - b\log(R-A) \tag{3.16}$$

and for equilibrium

$$(R-A) = \sigma - u = \sigma' \tag{3.17}$$

where: d is the average half space distance between paricles, $(R - A)$ is the net internal repulsive pressure, and a and b are constants.

With the use of the Langmuir equation and modified relationship of van Olphen (1963) between the mid-plane potential and the half space distance in the assumed parallel plate model, the half space distance corresponding to various magnitudes of repulsive pressure have been computed for three clay minerals for the properties as detailed in Table 3.4. (Nagaraj & Srinivasa Murthy 1983).

Figure 3.21 shows the plot between the half space distance and osmotic repulsive pressure. Irrespective of the type of clay mineral, for the osmotic repulsive pressure in the range of 40-800 kPa, the $d - \log(R-A)$ plot can be linearized to the form

$$d = 63.40 - 18.86\log_{10}(R-A) \tag{3.19}$$

with a correlation coefficient of 0.992.

In Equation (3.19) the effect of a clay mineral on the unique relation is subdued by the specific surfaces of each type of clay. To ensure generality of the $(R-A)$ relationships, further investigations were carried out by considering the possible field variation in the values of the electrolyte concentration, n, from 0.007 to 0.013 and that of valency, v, as 1 and 2 (Nagaraj & Srinivasa Murthy 1986).

Examination of the above analytical approaches reveals that the fundamental relations between half space distance, d versus $(R-A)$ would be general with re-

Table 3.4. Properties of soils used in the analytical study.

Soil type	Specific surface area m^2/gm	Base exchange capacity μ eq/gm
Kaolinite	15	30
Illite	100	400
Montmorillonite	800	1000

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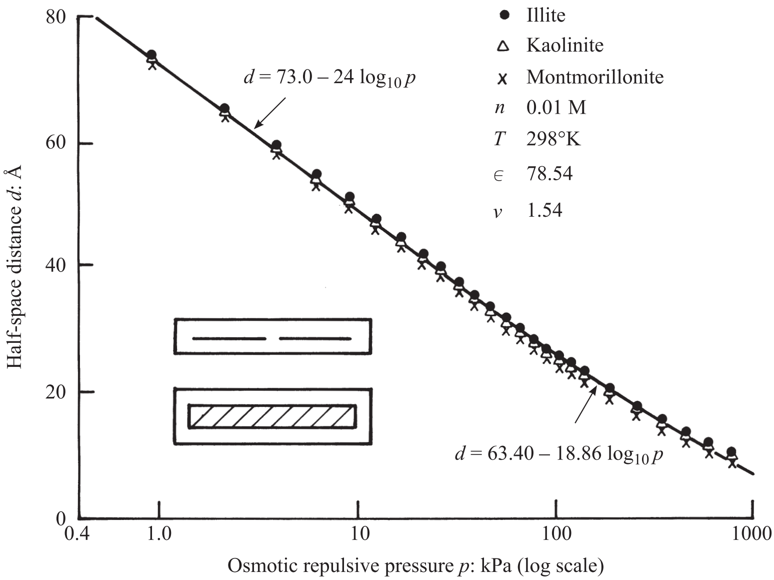

Figure 3.21. Linearized half space distance, d versus osmotic repulsive pressure ($R - A$) plot (Nagaraj & Srinivasa Murthy 1986).

spect to the principal sheet clay minerals having very wide variations in specific surface which get subdued in these relations.

How the above relations can be transformed for practical use to encompass the parameters commonly determined in geotechnical engineering practice merits examination.

For the parallel plate model, which is the basis in the above analytical formulation, Bolt (1956) and Nagaraj & Jayadeva (1981) have interrelated the void ratio, a macro-parameter with geometrical characteristics of clay units, the external surface area and separation distance by the relation

$$e = G\,\gamma_w\,S\,d \tag{3.20}$$

when γ_w is in g/cm^3, S is in m^2/g, half space distance d is in Angstrom units, and G is the specific gravity

$$e = G\,\gamma_w\,S\,d\,10^{-4} \tag{3.21}$$

For saturated clays $e = w\,G$; when water content w is expressed as percentage, the above equation reduces to

$$w = 0.01\,S\,d \text{ and } d = w/0.01S \tag{3.22}$$

The analytical relationship can be expressed as

$$w/0.01S = 63.40 - 18.86 \log\,(R - A) \tag{3.23}$$

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$$w/S = 0.6340 - 0.1886 \log (R - A) \tag{3.24}$$

In Equation (3.24), the specific surface, S, and the net interparticle force, $(R - A)$, are not engineering parameters amenable for measurement in routine investigations. As it is obvious from the discussions on the microstructure of clays (see Sections 3.24 and 3.2.5), the parallel plate model is not representative of particle dispositions in actual clay water systems. If the above considerations have to serve any useful purpose, it should be possible to transform the relation in Equation (3.24) with appropriate engineering parameters. Referring again to the microstructural considerations of clay-water systems at their liquid limit state, it is possible to ascribe a value of 6 kPa to osmotic suction $(R - A)$, the corresponding equilibrium water content being w_L.

$$w_L/S = 0.6340 - 0.1886 \log_{10} 6 \tag{3.25}$$

$$w_L = 0.4872\, S \text{ or } S = 2.052\, w_L \tag{3.26}$$

on substitution of this value in Equation (3.24) and replacing $(R - A)$ by $(\sigma - u)$ the resulting equation is the following generalized state-effective stress relation:

$$\{w/w_L\} = \{e/e_L\} = 1.3 - 0.387 \log (\sigma - u) \tag{3.27}$$

The above relation is based on the premise that the applied stress after dissipation of pore water pressures is resisted by long range interparticle forces mobilized after appropriate changes in the micro-fabric. The microfabric picture as depicted in Figure 3.14 precludes only parallel plate configurations and interactions. Another commonly observed fact is that a clay-water system monotonically compressed over a pressure range does not rebound to its initial state upon unloading. Hence it is necessary to examine what happens to the clay microstructure under imposed loading. Parry (1959) as early as 1959 examined this point. As there are no preferred orientations of clay particles in the clay-water system, due to random orientations the inter-particle separation distances vary over a very wide range from point to point, such that both short and long range forces are operative adjacently. This possibility has been illustrated by considering a compression curve (Parry 1959) (Fig. 3.22). Each point on the virgin compression curve and on any rebound curve is an equilibrium point with zero pore water pressure. At any void ratio, e, the external stress may have a value within a wide range (e.g., σ_a, σ_b, σ_c) the maximum being the virgin consolidation value σ_{NC}. With this possibility, the general equation has been enunciated as

$$\sigma_E - \sigma_I + \sigma_{AL} = 0 \tag{3.28}$$

where σ_E is the external stress, σ_I is the intercluster repulsion, σ_{AL} is the internal latent attractive stress.

The internal attractive stress, σ_{AL}, has been termed as the latent stress, since it is not available for rebound of the system upon unloading. This component is attributed to London-van der Waals attractive forces and hence tends to be irreversible,

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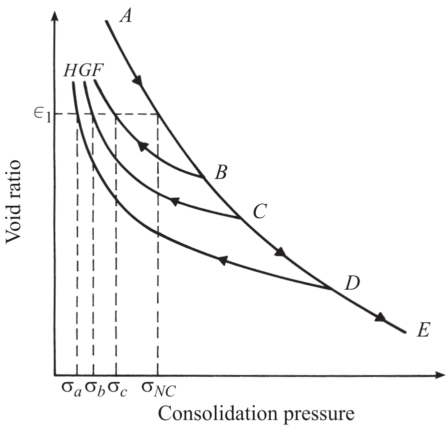

Figure 3.22. Compression and rebound curves from different consolidation stress levels (Parry 1959).

contrary to what happens while the net repulsive forces ($R - A$) alone are operative. As such, a clay-water system is a dynamic system with progressive internal changes to the microfabric occurring at all levels of application of external loading and/or environment, due to the occurrence of short and long range forces simultaneously. Hence, a direct extension of micro-analytical relations to the macro-level may have to be through the analysis of compression of actual experimental data. Hence what appears to be tenable is the phenomenological approach in order to bridge the gap between the micro- and macro- approaches.

The compression path of eleven soils with a wide range of liquid limit water contents, in the range of 36 to 160%, collated from published literature as well as the compression data generated, where consolidation has been started from water contents corresponding to the liquid limit state, have been analysed (Nagaraj & Srinivasa Murthy 1986, Nagaraj et al. 1993) (Fig. 3.23). It can be observed that the compression paths are spread out, with the position of each of the plots of the soil being in the decreasing order of its liquid limit water content. The higher the liquid limit, the steeper is its path. When void ratios at different consolidation pressures are normalized by void ratios corresponding to the liquid limit water content of that soil, it can be seen that all the normalized points fall within a narrow band $\{e/e_L\}$ versus consolidation pressure. Within the stress range of 25 to 800 kPa the disposition of points can be represented by a linear relation of the form

$$\{e/e_L\} = a - b \log_{10} (\sigma - u) \qquad (3.29)$$

For the data analysed $a = 1.122$, $b = 0.234$ with a high degree of correlation of over 0.96. For the data generated, the corresponding values of $a = 1.23$, and $b = 0.276$ show a high degree of correlation in the same range. It is likely that the constants may slightly vary with the analysis of still more data, but the form of the relation would be the same as obtained by micro-mechanistic considerations (Equation 3.27). We should examine in what way this phenomenological relation obtained for the generalized state line represented by Equation (3.29) with the

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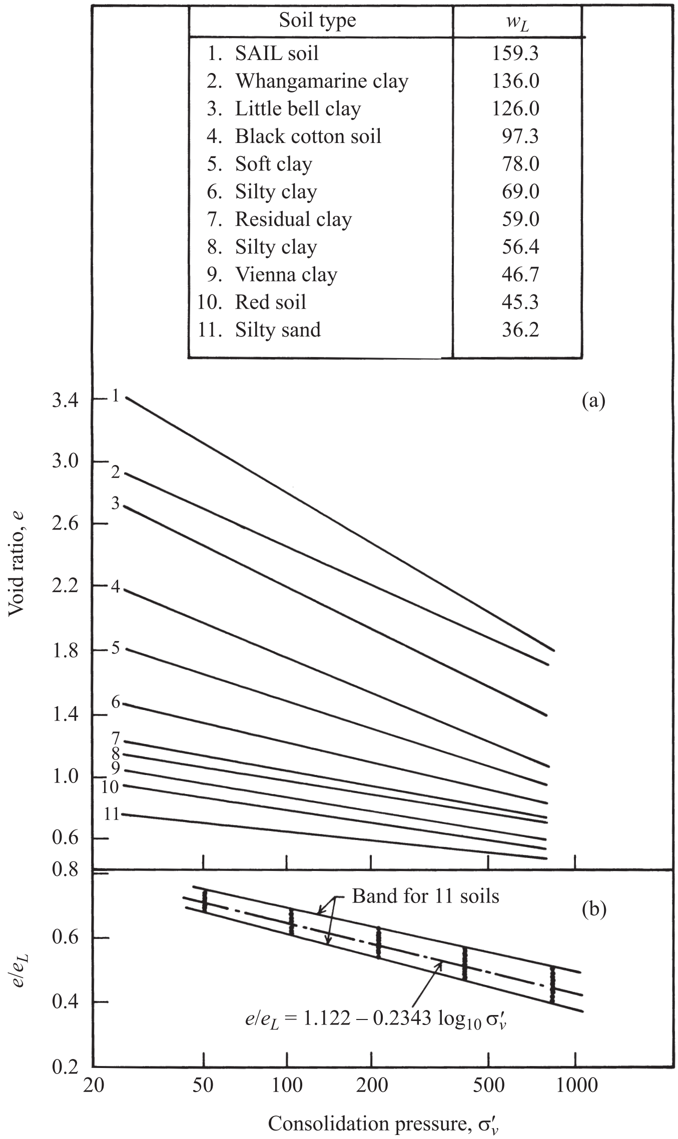

Figure 3.23. Experimental compression paths and their generalization (Nagaraj & Srinivasa Murthy 1986).

reference state parameter e_L, is different from the unique relationship between the consolidation stress and the void ratio at equilibrium for a saturated clay, recognized by Hvorslev (1936) as early as 1936.

It is worthwhile to examine what the generalized state parameter reflects. Since at the liquid limit state of clays the microstructure is of the same pattern, perhaps

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any attempt to probe the pattern of change in this microstructure with progress of consolidation i.e. the change in effective stress, might provide some inkling. To examine this aspect, published data are analysed on the microstructural changes due to consolidation of four soils (w_L = 29 to 100), each stressed to four stress levels in one dimensional compression, freeze dried, pore size distribution determined by mercury intrusion porosimetry (Griffiths & Joshi 1989). Figure 3.24 shows typical data of four soils at their liquid limit states and consolidation stress of 1500 kPa. It can be seen that the major changes are in the micropore volume rather than at submicroscopic level. The ratio of ΔV_t, which is the change in the intruded pore volume between the liquid limit state and at a consolidation stress of 1500 kPa, to the total pore volume at the liquid limit state, ΔV_{tL}, is also constant. Consequently, the relationship between normalized pore volume with respect to that at the liquid limit state and consolidation stress suggested by Griffiths & Joshi (1989) is of the form

$$\{V_t/V_{tL}\} = 1.193 - 0.206 \log(\sigma - u) \tag{3.30}$$

and is similar to the e/e_L relation indicated in Equation (3.29). The above relationship implies that change in pore volume due to a monotonic stress increase is proportional to initial pore volume.

Further independent supporting evidence regarding the possibility of proportional changes in the void ratio leading to result in the same micro-fabric pattern is available from permeability data. It has been observed (Nagaraj et al. 1993) that in the case of normally consolidated clays at the same level of consolidation stress, although the void ratios at equilibrium are different the permeability coefficient is of the same order, reflecting the same pattern of the microstructure at the micropore level.

3.3 INTRINSIC REBOUND-RECOMPRESSION LINES

Reiterating the discussions of Parry (1959, 1960) on the latent interparticle forces mobilized along the virgin compression path, (see Section 3.3.2), the rebound and recompression characteristics are now examined. If the interaction between clay particles was conforming purely to the diffuse double layer theory, with assumed parallel particle arrangement throughout the clay water system, then upon reduction of the stress level it should revert back to the original position, i.e. the compression would be purely elastic. Obviously this does not happen. In a random arrangement of clay particles at many intra- and inter-cluster sites the particles are constrained to closer configurations even crossing the energy barrier, thereby resulting in an altered stable unit. In Figure 3.25 this possibility is illustrated for an ideal particle system although such a situation arises between stable cluster units. Such a grouping of particles results in a reduction in the operating specific surface, tending towards a non-conservative thermodynamic system. Based on the Gouy-Chapman diffuse double layer theory, Klausner & Shainberg (1967) generated $e - \log \sigma_v$

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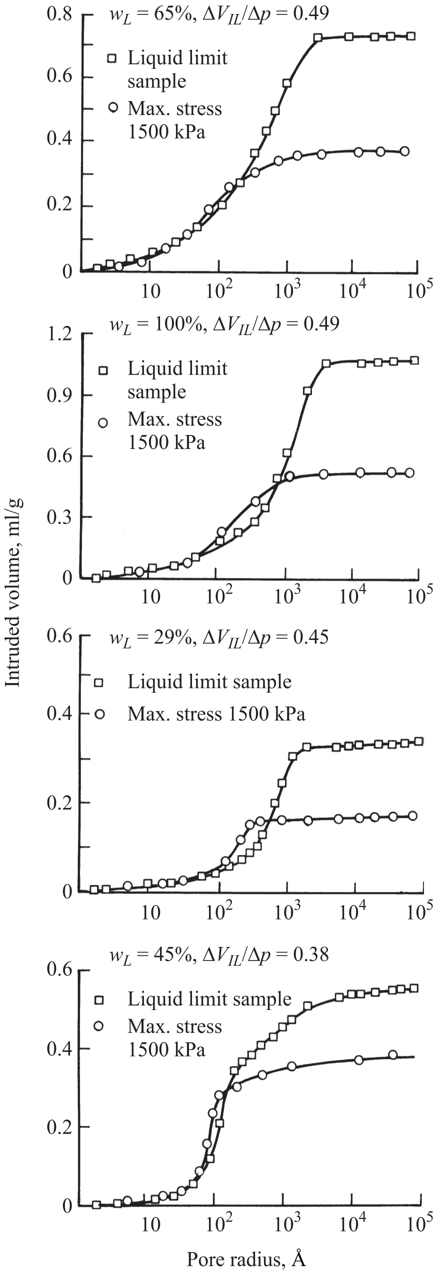

Figure 3.24. Intruded pore volume versus pore radius of four clays at their liquid limit state and compressed to 1500 kPa (data from Griffiths & Joshi 1989).

curves for possible different number of particles in a cluster. Comparing the experimental $e - \log \sigma_v$ curves of a Na-Montmorillonite soil with these curves, Nagaraj & Srinivasa Murthy (1986a) logically showed that there is a gradual grouping of particles into clusters along a normally consolidated path due to the monotonically increasing consolidation stress level.

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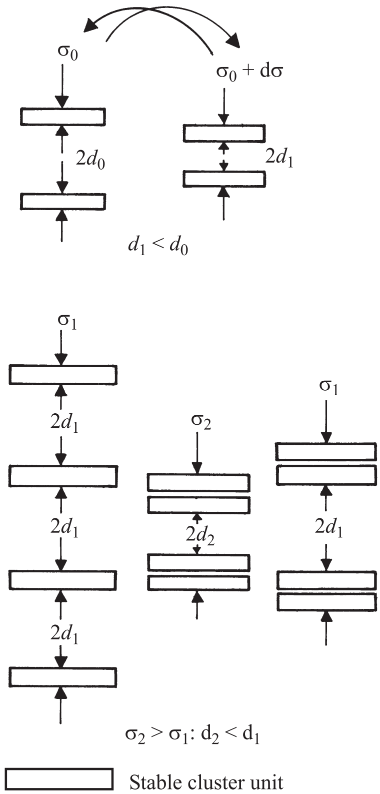

Figure 3.25. Illustration of unloading and reloading processes at the micro-level for the assumed parallel plate model.

Consequently the latent interparticle force component, as elucidated by Parry (1959, 1960), increases along the monotonically increasing stess level. This process, to a great extent being irreversible, upon unloading further interactions would take place between altered cluster units with reduced potential resulting in a flatter rebound path. Since the void ratio realized at a specific consolidation pressure is a function of the operating specific surface, *S*, the rebound and recompression paths are also a function of the reduced specific surface. Reverting to Figure 1.12, the experimental data reveal that while the slope of the average rebound-recompression paths exhibit the same order of slope as the stress level at which it is unloaded along the normally consolidated path, the magnitude of slope increases as the liquid limit of the clay increases. It is interesting to observe that the ratios of slopes (0.115/0.055 = 2.09) are in the same order as the ratio of the liquid limit water contents (93.5/47 = 1.98). This suggests that in uncemented saturated clays, in the absence of time effects, the stress history effects are proportional to the potential of the clay, i.e. the initial specific surface area, or at the engineering level the liquid limit water content of the clay.

It is now necessary to integrate the above observation with the actual state of the microfabric of clay already discussed (see Figs 3.8 and 3.25) before attempt-

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ing to generalize about the behaviour of stress-induced overconsolidated clay. At all stages, obviously, the compatibility between the stresses acting and the state of the clay has to prevail. In the microfabric the enclosed large micropores are held in equilibrium, probably by pore tension, and their size cannot be arbitrary but is very much controlled by the equilibrium force field. Under an increase of pressure from σ_1 to σ_2, the size of the pores would decrease, and simultaneously the distance between the interacting particle units would decrease from d_1 to d_2. But this reduction from d_1 to d_2 may not be sufficient to accommodate the reduction in the perimeter of the enclosed pores, while this would be possible by grouping of particles into larger clusters. Upon unloading back to σ_1, if the large enclosed pores have to regain their original size for equilibrium requirements, the clusters have to return to their original size and number. But, since it is difficult to separate the particles once they are in the attractive range of inter-particle separation, the clay microfabric may possibly reform to have fewer pores of specified size D_1. This results in a lower void ratio than that of the normally consolidated state at the same pressure. This suggests that the only difference in the microfabric at the two void ratios, given the same consolidation stress level along the normally and overconsolidated compression paths, is in the degree of aggregation of particles, i.e. cluster growth with the same pattern of microfabric.

Given this premise, that the behaviour of mechanically overconsolidated uncemented clays is influenced by the same initial potential of the clay, reflected by their respective liquid limit water contents, the average rebound-recompression experimental data of the same clays probed for obtaining the intrinsic compression line (see Fig. 3.23) are examined for possible generalization to obtain the intrinsic average rebound-recompression path. Figure 3.26 shows the spatial disposition of these lines in the working range of 25-800 kPa in the $e - \log \sigma'_v$ plot. For the purpose of clarity, these lines are plotted from their computed virgin void ratio corresponding to the maximum stress at rebound. Further, a narrow band is indicated, to which all the lines collapse upon plotting in the $e/e_L - \log \sigma'_v$ form. The best fit line of all the points in the band has a slope of 0.046 with a correlation coefficient of 0.98. In this plotting all lines are referred to an origin with an ordinate of $e/e_L = 1$ at 1 kPa. Even though the $e/e_L - \log \sigma'_v$ relations for rebound recompression paths vary with the preconsolidation pressure, their slopes are more or less parallel, implying only a parallel shift in space with an average slope of 0.046. This shifting of origin does not alter the magnitude of the slope. This validates the proposition that the average rebound-recompression lines of clays are also normalizable using their corresponding void ratios at the liquid limit of clays.

3.3.1 *Intrinsic state-effective stress relation*

The possibility of having a combined intrinsic compressibility relation, both for the normal and overconsoldiated behaviour of the clay in its reconstituted state, merits examination. The schematic $e/e_L - \log \sigma'_v$ compression and average re-

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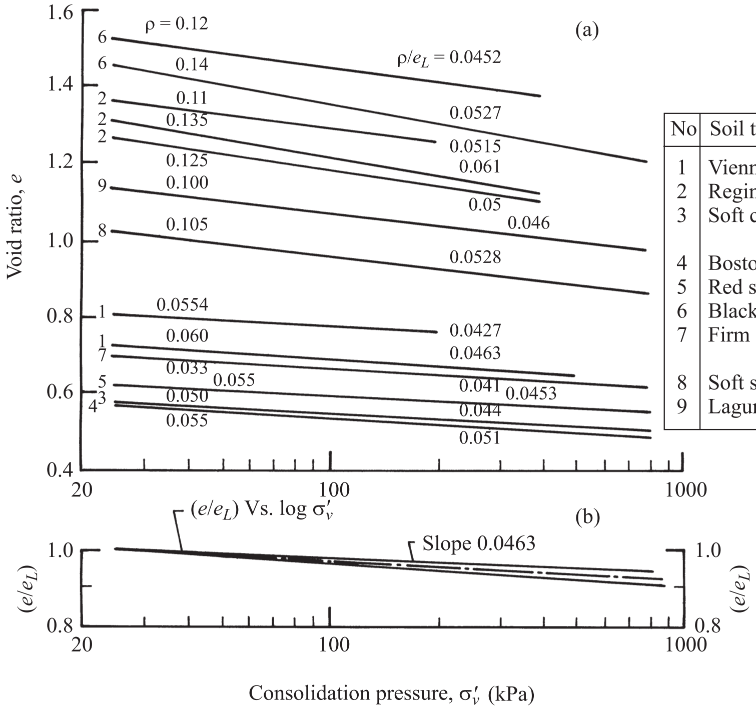

No	Soil type	w_L
1	Vienna clay	47.0
2	Regina clay	75.5
3	Soft clay	41.0
4	Boston blue clay	38.8
5	Red soil	45.3
6	Black cotton soil	97.3
7	Firm clay	50.0
8	Soft silty clay	71.0
9	Lagunillas clay	79.0

Figure 3.26. Experimental average rebound recompression paths of clays and their generalization (Nagaraj & Srinivasa Murthy 1986).

bound-recompression paths of clays are shown in Figure 3.27. The normally consolidated path is ABC, and BD is the average rebound-recompression path, with σ'_c as the preconsolidation pressure corresponding to point B. The compression path ABC is defined by Equation (3.29) and the equation for the line for the path DB can be written in the combined form as

$$\left\{\frac{e}{e_L}\right\} = a - b\log_{10}\sigma_c' + c\log_{10}\left\{\frac{\sigma_c'}{\sigma_v'}\right\} \tag{3.31}$$

with appropriate values for the constants. For the published data which have been analyzed and the experimental data generated, the values of the constants are a = 1.122 and 1.23, b = 0.234 and 0.276 and c = 0.046 and 0.042. For $\sigma'_v = \sigma'_c$ the relation reduces to that of the intrinsic compression path. Thus the stress state relations can be extended also to mechanically overconsolidated states.

At this juncture it is not possible to suggest the most probable constants for general application as the data examined are limited. However, as more experimental compression data of reconstituted clays devoid of any natural cementation are examined, it should be possible to arrive at universal constants for inorganic clays

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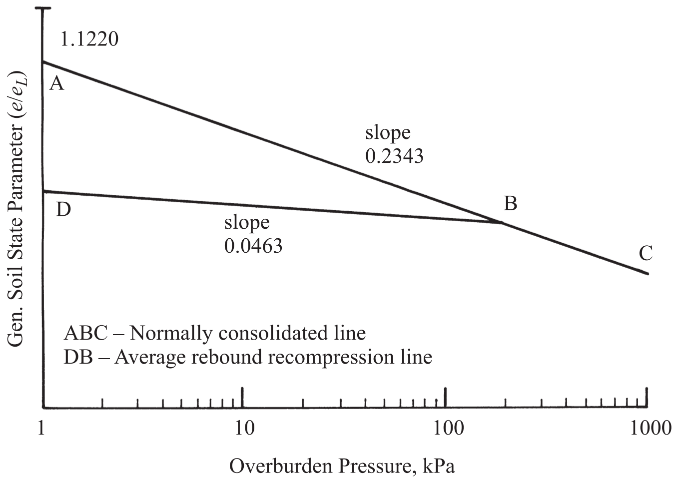

Figure 3.27. Intrinsic compression and rebound path at point B.

with sheet minerals. Despite this predicament, the predicted intrinsic compression paths as per Equation (3.31) are illustrated in Figure 3.28. What is depicted by these paths is the reference template. What has been considered is only the physico-chemical environment of the clay-water-electrolyte system and mainly for clays with sheet clay minerals group such as kaolinite, smectite and illite. With reference to this the disposition of the in-situ state of a soft clay along with the overburden pressure can be worked out. As to what extent this helps to identify the dominant factor of the three factors, stress, time and cementation, which infleunces soft clay behaviour, due to subsequent behaviour merits examination.

Subsequent to the above basic analysis and generalizations (Nagaraj & Srinivasa Murthy 1983, 1985, 1986, 1986a), Burland (1990) in his Rankine lecture, by analysis of the compressibility and shear strength of natural clays, has suggested a new soil parameter, the void index, I_v, defined by the relation

$$I_V = \left\{ \frac{e - e^*_{100}}{e^*_{100} - e^*_{1000}} \right\} = \left\{ \frac{e - e^*_{100}}{C^*_c} \right\} \tag{3.32}$$

where e^*_{100} and e^*_{1000} are void ratios at consolidation pressures of 100 and 1000 kPa respectively, obtained preferably by one dimensional compressibility tests on reconstituted clays, and C^*_c is the intrinsic compression index. A reconstituted clay, as defined by Burland (1990), is one that has been thoroughly mixed at a moisture content of between 1 to 1.5 times the liquid limit w_L. The basis for this proposition has been that the mechanical properties of a reconstituted clay are intrinsic since they are inherent to the material and are independent of the natural state which is normally influenced by time and environment. It has also been explicitly stated

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(Burland et al. 1996) that the intrinsic properties of a clay form a valuable frame of reference for assessing the corresponding properties of the intact natural material, and in particular the influences of the structured states acquired over a geological timescale in their formation to the present state. In fact this has been the very objective of the investigations cited during the period 1983-1986 and subsequently. The development of the basic framework for analysis discussed in this chapter is intended to reflect the rational basis for development of the intrinsic behaviour of clay-water systems when they are mechanically stressed.

From analysis of an extensive set of data it has been shown by Burland (1990), that the required parameters e_{100} and C^*_c can be related to the liquid limit void ratio e_L of the form

$$e^*_{100} = 0.109 + 0.679\,e_L - 0.089\,e^2_L + 0.01e^3_L \tag{3.33}$$

$$C^*_c = 0.257\,e_L - 0.04 \tag{3.34}$$

The resulting C^*_c and e^*_{100} are shown (Fig. 3.29) to agree well with those predicted, shown in the dotted line, from the generalized Equation (3.29) for the constants a = 1.122 and b = 0.2343 over a wide range of liquid limit values. It is very interesting

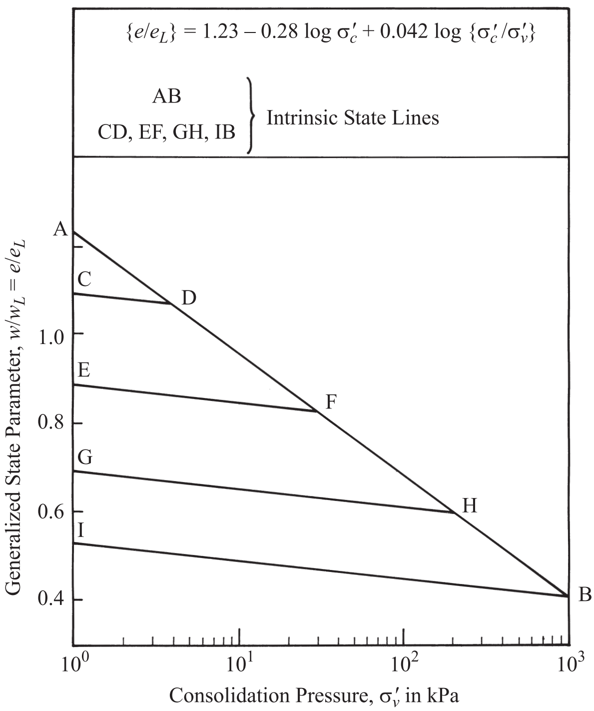

Figure 3.28. Intrinsic State Lines for mechanically stressed states of reconstituted clays as a reference template.

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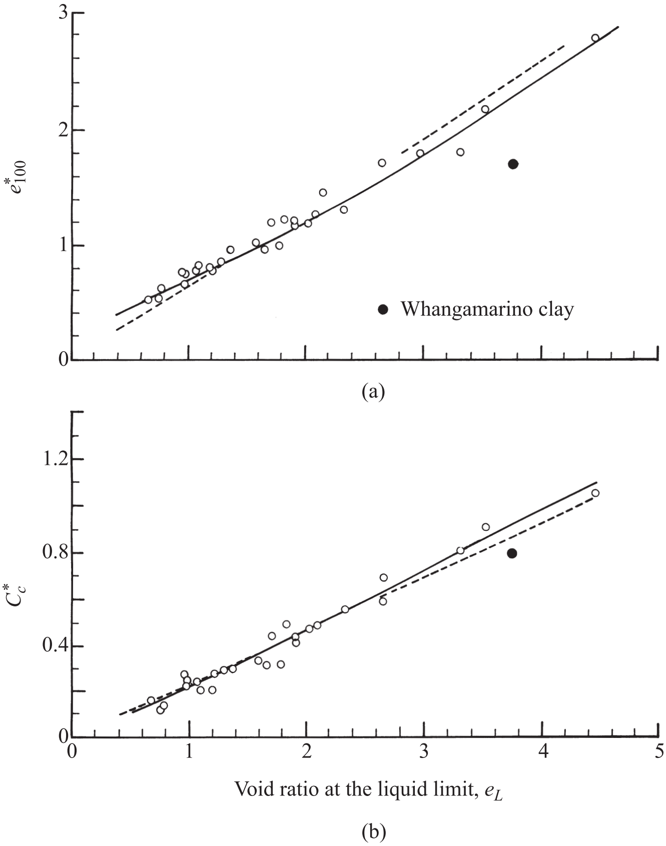

Figure 3.29. Relationships of a) e^*_{100} versus w_L and, b) C^*_c versus w_L (Burland 1990).

to note that the result $e_{100}/e_L = 0.653$, obtained from a value of e at 100 kPa from Equation (3.29), is very close to the average value of 0.662 for all soils considered by Burland (1990). Further examination of the void index and the intrinsic state parameter reveals that both are identical (Nagaraj et al. 1999a) and are intended to take into account the type of clay minerals and non clay constituents of the soil:

$$I_V = \frac{e - e^*_{100}}{C^*_c} = \frac{\Delta e}{be_L} = f\left\{\frac{e}{e_L}\right\} \tag{3.35}$$

The factor b has been evolved from double layer considerations and examined on the basis of the normalization of published compression data of reconstituted clays (Nagaraj & Srinivasa Murthy 1983). Further intrinsic behaviour of the rebound and recompression states can be directly incorporated into the intrinsic state parameter approach. The same is not possible using the void index-consolidation stress relationship. It is inferred that both approaches are complementary to each other and reinforce the direction in which the analysis can be pursued.

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As such, Equation (3.31) itself allows us to obtain intrinsic compression and recompression paths, as well as obviating either the need to conduct consolidation test on reconstituted clay or providing a means to cross-check the experimental compression path in case one is generated.

3.4 SOFT CLAY DEPOSITS – CLASSIFICATION

Within the above basic framework for analysis of soil behaviour, the data for natural clays are re-analysed. The relative disposition of the natural state of soft clays encountered at different depths in various countries (see Table 3.1) are shown in Figure 3.30. The numbers indicate different soft clays as in Table 3.1. It is inter-

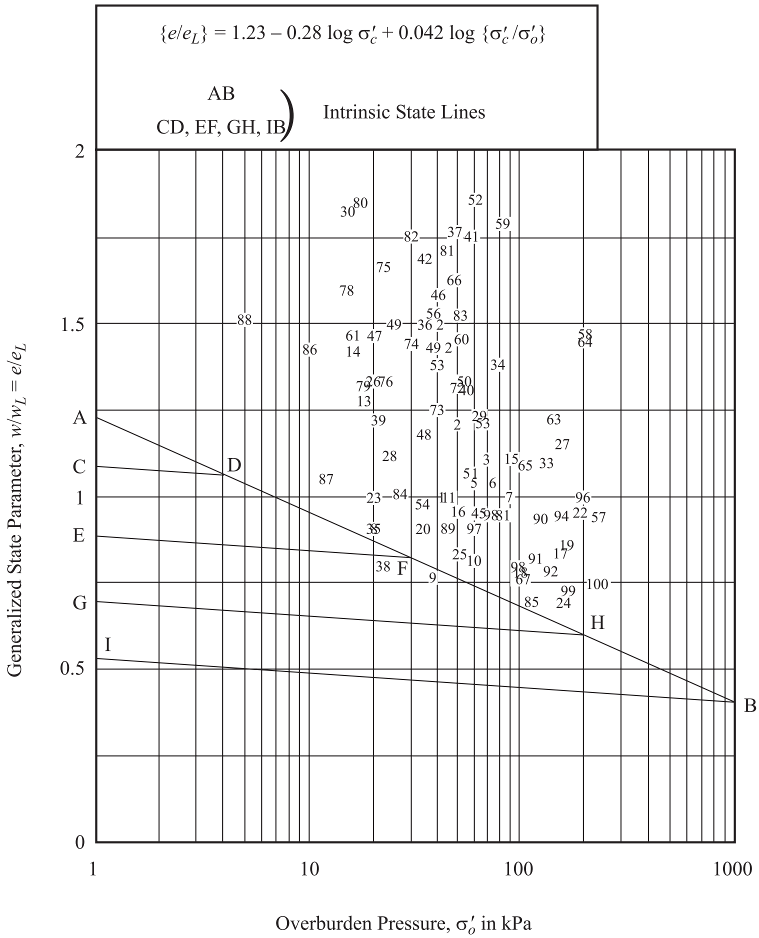

Figure 3.30. Relative disposition of natural state and overburden stress with reference to intrinsic compression paths. (see Table 3.1 for data and reference numbers).

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esting to observe that most of the clays fall on the right of the intrinsic compression line with quite a few at higher values of w/w_L or e/e_L greater than unity with the overburden pressure supported being in the range of 25 to 200 kPa. According to the reconstituted paths, clays ought to have water contents lower than their liquid limit water contents to support this order of overburden. As discussed earlier (see Section 3.3.2), for a suction of 6 kPa at the liquid limit arising due to interparticle forces of interaction, the value of w/w_L or e/e_L would have been unity, according to Equation (3.27). As clays are mechanically stressed and released and further recompressed, the values of these parameters would have been less than unity. There are a few points closer to the intrinsic line whose behaviour may not be akin to that of normally consolidated. It is very likely that all the clays would have reached the present state due to the combined effects of stress, time and cementation. Several stages of deposition and erosion with or without any cementation during the geological timescale would have been responsible for the realization of the present stage. The present in-situ state and overburden pressure may not have any direct inter-relationship, as in the case of compression and recompression of reconstituted clays. To encounter natural aged sedimentary clays exhibiting dominant stress history effects only may be a chimera. As such the need to use the prefix 'normally' and 'overconsolidated' in the classification of soft clays might have relevance to soft clays dredged and used in reclamation of land. In these cases the ageing effect, due to time alone or time and cementation, is unlikely to be dominant since the span of time in compressing and rebound and recompression due to construction loading would be relatively negligible compared to the geological timescale elapsed, with an unknown mechanical stress history, in the case of natural clays. What is obvious with natural soft clay deposits is aging due to the timescale involved in the formation of the deposit now found. As it might be also relevant to identify the natural deposits as aged clays without any prefix such as 'normally' or 'overconsolidated' about which no definite information is available.

Further, the effects of time and/or cementation might subdue effects due only to overburden stress. In the subsequent chapters the possibility of and the associated specific advantages of classifying soft clay deposits as:

– normally consoldiated,
– overconsolidated, and
– aged deposits (time or time and cementation effects),

are discussed along the methods to analyze and assess their engineering properties.

3.5 IN-SITU TEST DATA – A POSSIBLE GENERALIZATION FOR ANALYSIS AND ASSESSMENT OF SOIL BEHAVIOUR

Laboratory testing enables us to carry out tests, apart from simulated in-situ conditions, under a variety of conditions relevant to further changes in environmental and other stress states. Despite this fact, there is growing interest and activity

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worldwide in assessing mechanical properties right from the stage of subsurface investigations itself. This is intended to speed up the generation and analysis of in-situ test data and to overcome the scale and size effects arising due to macro-fabric features of soil deposits. For detailed discussions regarding the relative efficacy of laboratory and in-situ tests, the reader is referred to detailed treatment elsewhere (Nagaraj 1993).

In most of the in-situ testing, the response of the in-situ soil to stimuli such as penetration by a cone, lateral deformation by dilatometers and pressure meters due to induced fluid pressures, is measured. The measured response is translated to the strength and deformation characteristics of the material by appropriate analysis within the framework of a theory, for example, the expansion of cylindrical cavity in an isotropic material and shear failure of the material below the cone tip. In most cases the relationships are between one stress field and another or with resulting deformation. Lunne et al. (1997) have reported the possibility of identifying the deposit as underconsolidated or overconsolidated by considering the deviations of the cone penetration resistance path from the linear band with depth, as shown in Figure 3.31. The cone resistance, q_t increases by 2.5 to 5.0 times effective overburden pressure which in turn varies with depth. If the q_z q_t profile is left to this band the

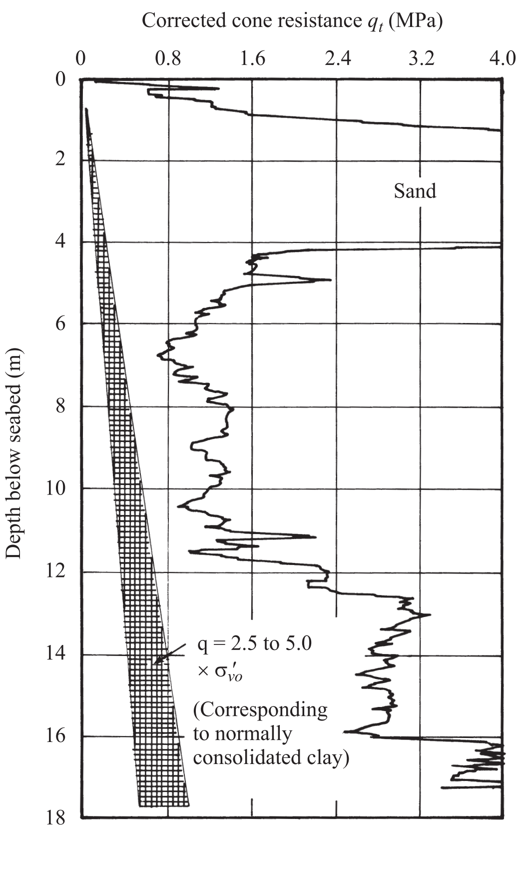

Figure 3.31. Rough indication of the OCR from q_t versus depth (Lunne et al. 1997).

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soil is regarded as underconsolidated. In the example depicted in Figure 3.31 the North Sea clay is overconsolidated, since the cone penetration resistance is greater than that assessed by the normally consolidated path.

Further efforts to classify the state of in-situ material have been by the normalization of appropriate stress components. Wroth (1984) and Houlsby (1988) suggested that cone penetration test data can be normalized using the following parameters, using corrected cone resistance, q_t, and sleeve friction, f_s, and σ_o is vertical stress both total and effective.

1. Normalized cone resistance:

$$Q_t = \frac{q_t - \sigma_{vo}}{\sigma'_{vo}} \tag{3.36}$$

2. Normalized friction ratio:

$$F_R = \frac{f_S}{q_t - \sigma_{vo}} \times 100\,\% \tag{3.37}$$

3. Pore pressure ratio:

$$B_q = \frac{u - u_o}{q_t - \sigma_{vo}} = \frac{\Delta u}{q_t - \sigma_{vo}} \tag{3.38}$$

In Figure 3.32 the soil classification chart based on normalized CPT and CPTU data, proposed by Robertson (1990), has been presented. Since all the classification systems are based on the use of normalized stress components, it would be in order if attempts are made to account for stress history effects. As the actual state of the material often encountered is a result of the combined effects of stress, time, and cementation it may not always be easily possible to account for all states by the above mode of identification. There might be difficulties in identifying the various types of soils which vary in composition without having any inkling about the same at least in the form of representative samples with depth.

Towards getting information about the in-situ deposits along with cone penetration data, Shibata et al. (1994) developed Radio isotope-cone penetrometers and have used this facility successfully at many sites. Two different kinds are Neutron Moisture (NM) and Nuclear Density (ND) cone penetrometers (Fig. 3.33).

The basic details of the units are indicated in the same figure. One of the major advantages of RI cones over conventional ones is that void the ratio, *e*, profile, through density and water content can be monitored continuously without sampling and laboratory tests. Even with this the lacunae of having representative soil at various locations is not circumvented. As of now there is no facility to provide a representative sample, density and water content varying with depth, along with the corresponding cone penetration data with depth.

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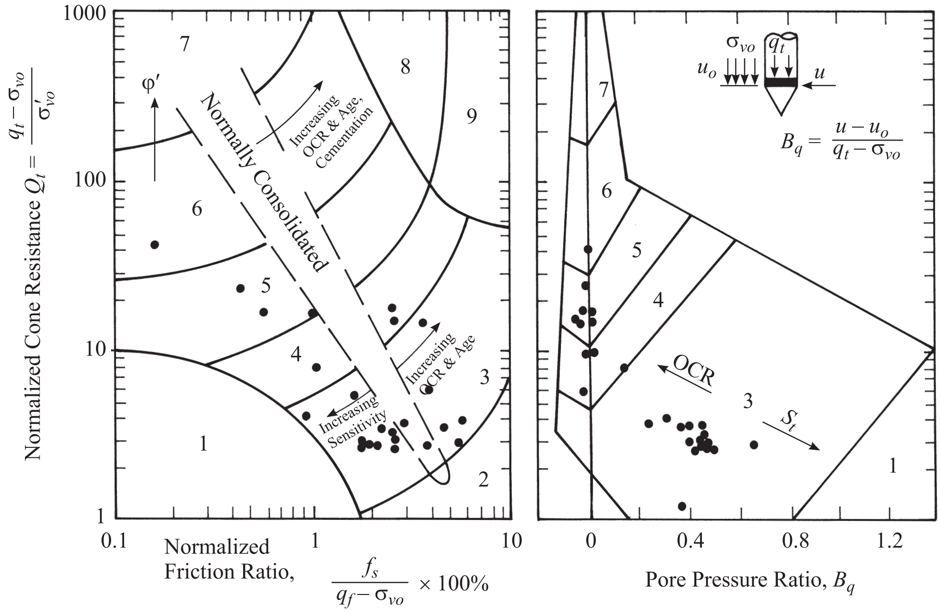

1. Sensitive, Fine, Grained
2. Organic Soils-Peats
3. Clays-Clay to Silty Clay
4. Silt Mixtures-Clayey Silt to Silty Clay
5. Sand Mixtures-Silty Sand to Sandy Silt
6. Sand-Clean Sand to Silty Sand
7. Gravelly Sand to Sand
8. Very Stiff Sand to Clayey Sand
9. Very Stiff, Fine Grained

(•) *Heavily Overconsolidated or Cemented*

Figure 3.32. CPT and CPTU data from a deep borehole on the proposed classification chart (Robertson 1990).

In order to have an integrated approach between laboratory and in-situ testing the possibility of having a cone penetrometer with all the facilities to determine water content and density profiles, along with representative sample at chosen depths, in addition to continuous profiles of cone penetration data, is explored. This is a combination of the Swedish Cup Sampler discussed by Hvorslev (1949) (Fig. 3.34a) along with RI cones and the cone penetrometer (Fig. 3.34b). The basic Swedish Cup Sampler consists of a drive pipe with a drive point and a short slit. The drive pipe is closed temporarily by a spring actuated piston valve. When a sample is desired, a cup attached to a rod is lowered into the drive pipe, and the piston valve is depressed until the top of the cup is slightly below the drive pipe. The later is rotated until sufficient soil has passed through the slit to fill the cup, whereupon the cup is slightly withdrawn and the slit is again closed to allow continued penetration by the cone. In this setup, schematically shown in Figure 3.34b all the features of the Swedish cup are incorporated except the drive point. In this case the drive point is that of the regular cone penetrometer or piezo-cone. The RI cones are also shown. Such an

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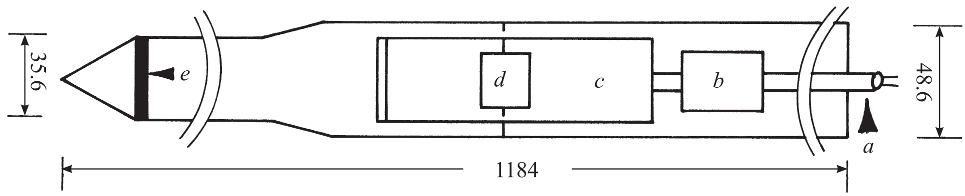
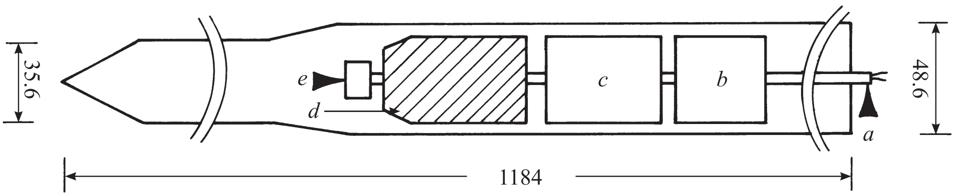


a: cable leading to data collection system; *b*: pre-amplifier; *c*: He^3-filled proportional tube;
d: Cf^{252} fast neutron source; *e*: porous ceramic filter (all dimensions in mm)

(a)



a: cable leading to data collection system; *b*: pre-amplifier; *c*: photomultiplier tube;
d: lead (pb); *e*: C_S^{137} gamma ray source

(b)

Figure 3.33. Schematic representation of a) the NM and, b) the ND cone penetrometer (Shibata et al. 1994).

integrated facility is not at present commercially available. If it were to be a possibility, then it would be possible to have, in addition to cone resistance, the in-situ density and water content profile. At specific depths the representative sample also would become available. The representative sample could be tested to get the grain size and index properties, which provide a means to define the intrinsic state line (ISL). The density and water content and overburden pressure would enable location of the point on the generalized soil state parameter and consolidation pressure plot. The classification of the soil deposit can be done as discussed in this chapter and the cone penetration resistance would be response data to penetration, which can be translated to engineering parameters as discussed by Lunne et al. (1997). By this mode, it is perhaps possible to integrate both the state parameter approach and in-situ testing, thereby enjoying the specific advantages of both approaches. A word of caution is that the presence of macro-fabric features needs to be identified and considered in exercising engineering judgment.

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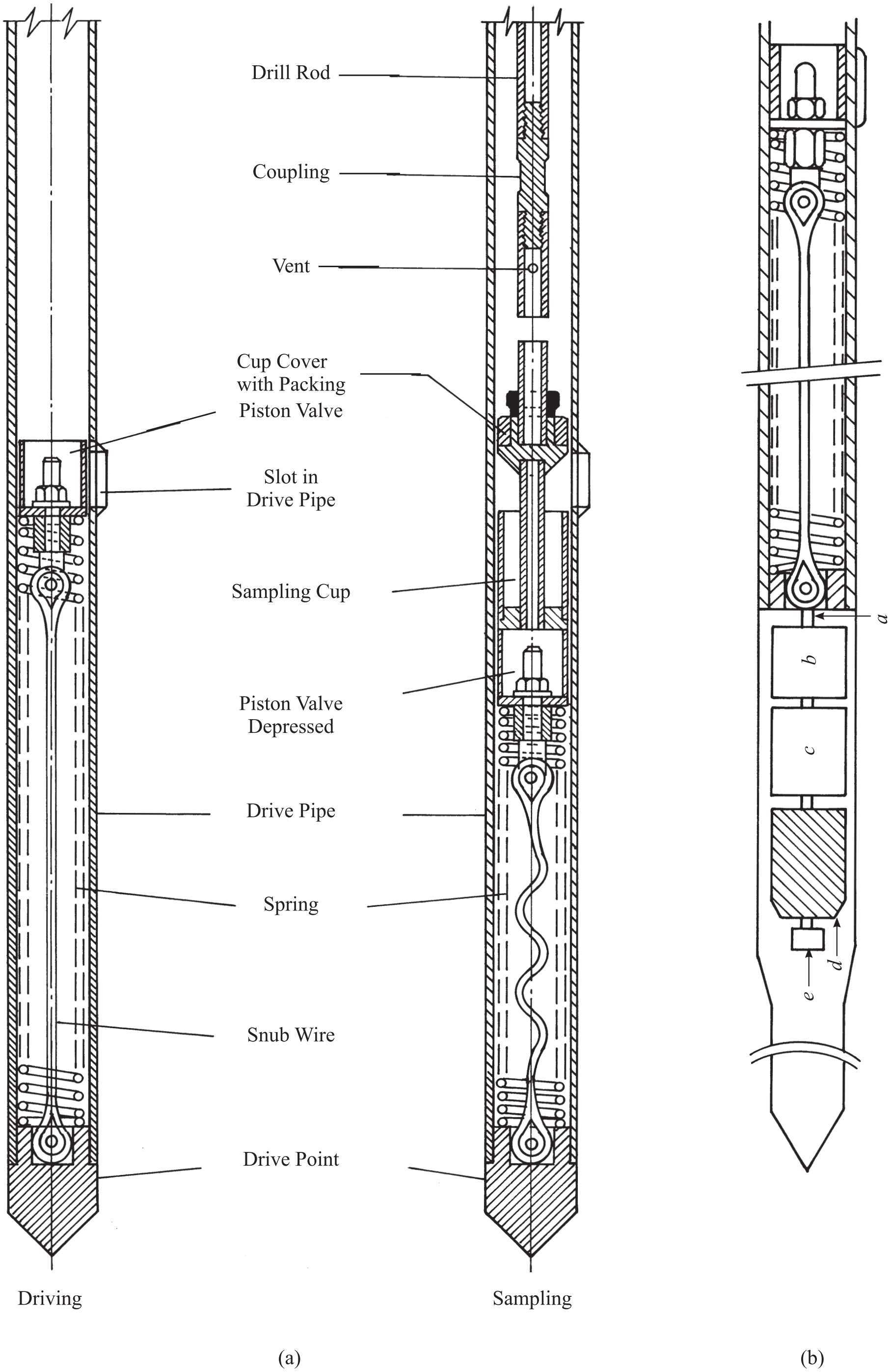

Figure 3.34. a) Swedish cup sampler (Hvorslev 1949), and b) schematic diagram of possible integrated cone penetrometer.

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3.6 CONCLUDING REMARKS

In retrospect, the above discussions suggest that $\{e/e_L\}$ indirectly reflects the micro-fabric of the clay-water system in equilibrium due to interparticle forces induced by external loading. To consider the role of interparticle forces, in addition to the stress conditions the state of the clay water system at every stage needs to be considered. That is, the suggested and discussed effective stress relation for fine-grained soils in Equation (3.15) can, for all practical purposes, be regarded as the one stated in Equation (3.31).

$$\sigma' = (R - A) = \sigma - u \tag{3.38}$$

$$\{e/e_L\} = a - b \log_{10} \sigma'_c + c \log_{10}\{\sigma'_c/ \sigma'_v\} \tag{3.39}$$

In a way, this need has already been recognized and met with in the critical state concepts as advanced by the Cam clay model.

The intrinsic state line depicted by Equation (3.31) is a variation of the clay micro-fabric with monotonically increasing consolidation stress, both in virgin compression and recompression paths. While the $e - \log_{10} \sigma_v$ relation for a saturated uncemented clay is unique, the intrinsic state-effective stress relation (Equation 3.31) is unique for a wide spectrum of clays. For the known potential of clay in terms of its liquid limit water content from the generalized relation, the intrinsic compression and recompression paths can be assessed.

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CHAPTER 4

Uncemented saturated soft clays – stress and time effects

4.1 INTRODUCTION

Most of the laboratory and field investigations dealing with soft clays, in soft soil engineering, are involved in understanding, analysing, generalizing and, if possible, assessing the responses of soft clays to the stresses induced due to loading. Stability calculations and settlement computations are performed with appropriate input parameters to arrive at acceptable design details for the construction of the contemplated structure. Further, assessment of the engineering properties of large tracts of soft clay would go a long way to allow engineering decisions regarding the need and mode of ground improvement to be used, as well as to monitor the same at various stages of their implementation. Conventionally, practical problems are being analysed for their stability, concerning bearing capacity, slope stability and lateral pressures, as a limit equilibrium problem. Settlement computations are made and their magnitudes examined in relation to the tolerable limits prescribed. Partly for simplicity in practice, and partly because of the historical development, the stability and deformation problems are, all along, treated as distinctly separate problems instead of as being interdependent. But the present trends are clear: to carry out a more realistic analysis and design with complete knowledge of stresses and strains at compatible levels right up to failure, which are inelastic deformations over a wide range of working stresses before they reach critical state. This behaviour is described by the recent work hardening elasto-plastic constitutive relations, which define unique relationships between general stresses and strains encompassing both settlement and stability problems (Chen & Mizuno 1990).

Whenever saturated clays are loaded, they deform. If their behaviour is governed by particulate considerations, the rate of deformation is time dependent as well as depending on the permeability of the soil. Under a given increment of loading, due to the incompressibility of the bulk water, excess pore water pressure mobilizes, which sets in a hydraulic gradient for water to flow so as to progressively dissipate induced pore water pressure. This state of instantaneous equilibrium of the clay in terms of total stress, effective stress and excess pore water pressure, is an undrained situation of the clay. As the dissipation is completed, with excess pore water pressure becoming practically zero, the total stress is the

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effective stress. The process of time dependent dissipation of excess pore water pressure and the consequent change in pore volume is recognized as consolidation. Between two equilibrium undrained and drained situations, the process of change in volume brought about by the change in the effective stress is compression. The equilibrium state after complete pore water pressure dissipation is the drained state.

The engineering property assessment of soft clays for the analysis of practical problems and implementation of several ground improvement methods involves determination of:

1. compressibility and consolidation behaviour,
2. shear strength,
3. stress-strain behaviour in undrained and drained conditions, and
4. permeability.

To reiterate, the experimental compression path on an undisturbed clay forms the basis for classification of the soft clay deposit encountered. Hence sampling with least disturbance is required. Before dealing with the classification of soft deposits and using appropriate methods to assess the engineering properties, a brief discussion is provided on sampling requirements and techniques.

4.2 SAMPLING REQUIREMENTS AND TECHNIQUES

Sampling in geotechnical engineering is an act of removal of small portions of material primarily to characterise their variation in-situ and for laboratory testing. Scientific studies on sampling were pioneered by Hvorslev (1949) as early as 1949. This has been all along, and is still, a valuable guide for practising engineers. An updated compendium on sampling prepared by the Subcommittee Report on Soil Sampling (1981) would provide details of recent developments for use. Although very sophisticated samplers and techniques are at the disposal of geotechnical engineers, they are not fully utilised either due to lack of adequate transfer of technology or due to the general feeling that the use of these methods would slow down the progress of exploration, which impedes meeting the time targets stipulated for design of the substructure.

4.2.1 *Sampling requirements*

Irrespective of the practical situation, sampling involves different tools and demands considerable time, effort and expenditure. This is particularly so, if the undisturbed sampling component in any investigation increases. Hence, it is always desirable to limit undisturbed sampling to the minimum as warranted by the geological history of the area and practical needs for the design of sub-structures. Rowe (1970) has re-examined the concepts of undisturbed representative sampling of Hvorslev (1949). The sample obtained might in itself be undisturbed but not repre-

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sentative of the local minor geological details of the strata. Figure 4.1 shows the sample size, in relation to the stratum, to be truly representative of the in-situ geological features. If the deposit is uniform, the test results on small samples can completely represent the mass behaviour. If the soft clay stratum is layered with silt or sand of varying thickness or clay with a system of silt filled fissures, for the sample to be representative, the size of the sample should be such that the gross fabric is present in the sample and mass behaviour is reflected in the laboratory testing.

As a first step, without any detailed examination of the sample disturbances, the following three practical requirements suggested by Hvorslev (1949) are useful for an overall assessment of the soil sample quality devoid of any macro-structural features:

1. The specific recovery ratio, SR, should be between 1 and $(1 - 2C_i)$ where C_i is the inside clearance ratio, i.e. the ratio of $(D_s - D_e)$ to D_e (D_e is the diameter of the cutting edge and D_s is the inside diameter of the sampler). The ratio of the increment of the sample entering the tube Δl and the increment in the advance of the sampler, ΔH is the specific recovery ratio.

$$SR = \frac{\Delta l}{\Delta H} \tag{4.1}$$

2. The net length, weight and the results of control tests must not change during shipment, storage and handling of the samples.
3. On the surface or the sliced sections, there must not be any visible distortion, plane of failure, pitting, or any other sign of disturbance.

A more detailed examination will be made, in the next chapter, of the assessment of the quality of the samples, the quantification of the sample disturbance and

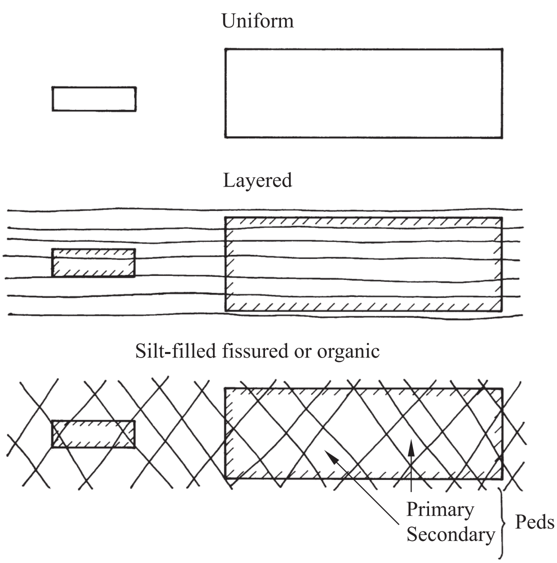

Figure 4.1. Sample size on the fabric representation (Rowe 1970).

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methods to modify the engineering properties determined using partially disturbed samples.

4.2.2 *Sampling techniques*

Developments in sampling techniques to cope with the different field situations are so vast that it warrants a detailed discussion. Since this is outside the scope of this treatise, the reader is referred to a more detailed discussion provided elsewhere (Nagaraj 1993). Of the various soil groups, sampling of soils in a marine environment requires considerable skill and care, due to the possibility of these deposits possessing a highly sensitive metastable micro-structure susceptible to marked mechanical disturbance at all stages of sampling, and subsequent handling prior to laboratory testing. With the thickness of clay extending either to shallow depths or to greater depths both on land as well as below appreciable depths of water, the main requirement is to obtain the maximum length of an undisturbed sample in a single operation.

Detailed investigations by La Rochelle & Lefebvre (1971) clearly demonstrate the superiority of block samples over tube samples with regard to the disturbance introduced by either method of sampling. It is relatively easy to obtain block samples by advance trimming or carving of samples with lateral supports in the case of open trenches at shallow depths. To cover block sampling up to 10 to 15 metres the Sherbrooke sampler developed by Lefebvre & Poulin (1979) can be advantageously used to obtain 25 cm. diameter cylindrical block samples. Even though adequate refinements to this technique have been built into the methodology, due to the sheer self-weight of the unsupported block during removal and handling slight mechanical disturbance is inevitable unless the soft clay has an undrained strength of the order of 10-15 kPa. On the other hand, if strength is high, of the order of 100 kPa or more, disturbances might creep in due to the brittle nature of the sample.

While drive sampling is resorted to, in general high quality undisturbed clay samples can be obtained using thin wall tube samplers with a stationary piston. It has been shown that the test results from the samples of soft and sensitive soils obtained by the hydraulic piston sampler (Osterberg 1973) compare very well with those obtained with block samples.

Subsoil investigations may have to be carried out for engineering purposes quite frequently on the beds of lakes, bays and near ocean shores. A depth of water up to 30-35 metres can be taken care of by casing and the use of drive samplers. Next, wire-line operated sampling devices have been used to cover depths of as much as 150 m. Drill pipes are first advanced to the desired depths, followed by the sampler attached to the wire along with a sliding weight until it rests on the bottom. The advancing of the sampler is done by raising the weight by 1.5 m and dropping it on the sampler a number of times till a penetration of 0.6 m is realized.

Reverting to the discussions in the Section 3.5, the premise for classification is that all soft natural deposits are formed due to the combined effects of stress, time

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and environment. Perhaps what can be attempted is to base the classification on the predominance of any one of the above factors. In-situ undrained strength data, normally available in routine investigations for assessment of sensitivity, can also aid classification. From the in-situ data and index properties the following broad groupings have been advocated (Nagaraj et al. 1997a).

Normally consolidated uncemented clays: If the point of (e/e_L, σ'_o) lie along the intrinsic state line and the clay is identified as very soft clay (vane strength in the range of 5-10 kPa), for all practical purposes it can be regarded as normally consolidated uncemented clay. The effects of cementation due to the environment factors on soil behaviour can be ignored. This would be in order, as beyond the yield stress level, which is low compared to the stress levels envisaged due to construction activity, the responses of soil are governed predominantly by particulate considerations.

Overconsolidated clays with or without cementation: If the points locate below the intrinsic state line, the state of the clay can be generally regarded as overconsolidated. Time effects also result in the same disposition. To identify whether it is stress dependent overconsolidation, time effects or that the deposits have inherited cementation effects as well, the laboratory recompression path provides additional information. The possibility of having an in-situ overconsolidated state with cementation can be identified from undrained strength data. This aspect will be examined in detail in the next chapter.

Naturally cemented clays: If the points locate to the right of the intrinsic state line, the dominant factor influencing engineering behaviour is cementation. The cementation effects, would subdue the effects of stress history and time.

4.3 COMPRESSIBILITY

4.3.1 *Normally consolidated*

This is the state of clay devoid of any stress history, time or cementation effects. The generalized state and effective stress relation as typified by Equation (3.29) with appropriate values of the constants, a, and b, would enable one to assess the compressibility of such deposits. The compressibility characteristics of a clay are usually reckoned in terms of the negative slope of the e – log σ'_v plot which is defined as:

$$C_c = \left\{ \frac{de}{d(\log \sigma'_v)} \right\} \tag{4.2}$$

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For example when the natural water contents are in the range of the liquid limit water contents, considering the relation with the value of constants $a = 1.122$ and $b = 0.234$,

$$C_c = \left[\frac{de}{d(\log\sigma'_v)}\right] = 0.234\,e_L = 0.234\,G_s w_L \tag{4.3}$$

Equation (4.3) predicts a constant slope for a given clay being a function of its liquid limit water content. Skempton (1944) has empirically shown that for remoulded clays, the compression index of the soil can be related to its liquid limit water content in the form:

$$C_c = 0.007(w_L - 10) \tag{4.4}$$

In a recent analysis, based on the fundamentals of soil behaviour, Nagaraj & Srinivasa Murthy (1983) have shown that Equation (4.4), generally regarded as an observation-based statistical relation, has a strong scientific basis from the considerations of diffuse double layer of clay water interactions. The compression index assessment by the above relation is realistic only for normally consolidated deposits. The more important consideration is the identification of the type of deposit for which the assessment of the compression index made by the above relation is tenable.

Terzaghi & Peck (1967) as early as 1967 modified this equation to include the compressibility characteristics of low to medium sensitive clays, of the form

$$C_c = 0.009(w_L - 10) \tag{4.5}$$

There has been an increase of 30% in the assessment of the compression index values, still grouping them under normally consolidated soft clays. Since the yield stress is in the lower ranges, the compression paths would be such that in the working stress ranges the compression index would be in accordance with that assessed from the modified Skempton's relation Equation (4.5).

The compression paths of Figure 3.23, after plotting in the double logarithm scale, (Fig. 4.2) results in yet another linear relation of the form

$$\log\left(\frac{e}{e_L}\right) = 0.1433 - 0.168\log\sigma'_v \tag{4.6}$$

with the same degree of correlation and standard error of estimate as obtained for Equation (3.29). Upon differentiation and rearranging the terms, it results in another equation for the compression index of the form

$$C_c = -\left\{\frac{de}{d(\log_{10}\sigma'_v)}\right\} = 0.39e \tag{4.7}$$

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Soil type	w_L
1. SAIL soil	159.30
2. Whangamarino clay	136.00
3. Littlebelt clay	126.00
4. B. C. Soil	97.30
5. Soft clay	78.00
6. Silty clay	69.00
7. Residual clay	59.00
8. Silty clay	56.40
9. Vienna clay	46.70
10. Red soil	45.30
11. Silty sand	36.20

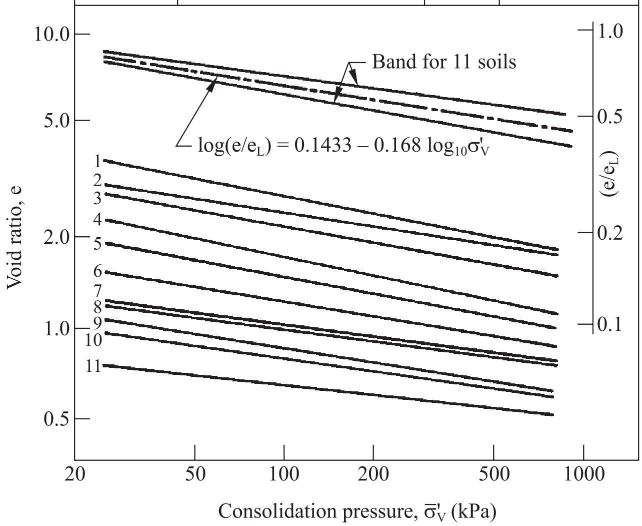

Figure 4.2. Compression paths of soils in Figure 3.23 in log *e* versus log σ'_v plot.

Equation (4.7) is different from that relating the compression index to the liquid limit water content. Empirical correlation linking the compression index and the natural water content or the void ratio of normally consolidated uncemented saturated soils, cited in the literature (Koppula, 1981, Bowels, 1979, Nishida 1956) have the same rational basis as discussed earlier. For a general value of specific gravity of the soil of 2.65, Equation (4.7) reduces to:

$$C_c = 0.0103 w_n \tag{4.8}$$

where w_n is the natural water content in percent. The practical significance of Equation (4.8) is that the natural water content expressed as a decimal, itself represents the compression index, if the soil is normally consolidated in an uncemented condition, i.e. for w_n = 50%, C_c = 0.5 and for w_n = 100% its value is unity.

The intrinsic state effective stress relation, of the form depicted in Equation (3.29), has been obtained for one-dimensional compression. The basic con-

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siderations of clay-water interactions and the consequent formation of the clay fabric can be expected to be the same whether it is one dimensional, isotropic or any other loading. Since the premise has been that double layer interaction takes place between the clusters placed on the periphery of large pores, except for the shape of the large pore, the possibility of having a generalized relation for applied mean principal stress $p' = (\sigma_1' + \sigma_2' + \sigma_3')/3$ merits examination. Figure 4.3 shows the $e - \log p'$ plots of four isotropically consolidated soils. The compression paths collapse into a narrow band when their generalized states are considered along with each of the compression paths. The data points of the generalized plots can be linearized within the stress range of 100 to 800 kPa in the form

$$\left\{\frac{e}{e_L}\right\} = a' - b'\log p' \tag{4.9}$$

with a high degree of correlation coefficient. The constants a' and b' are 1 and 0.21. These values appear to be slightly lower than that for the K_0 compression path rather than being slightly higher. This is possibly due to stress history effects

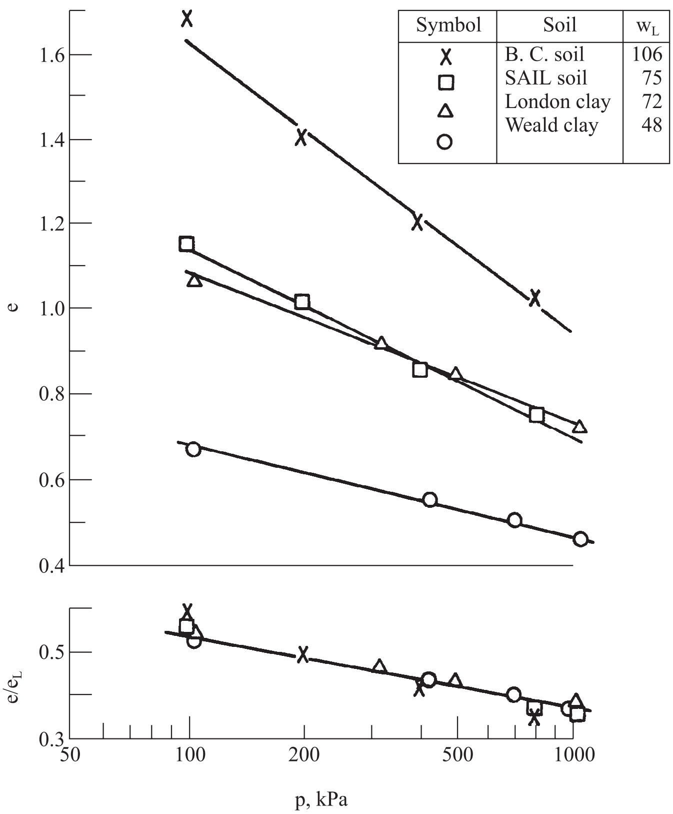

Figure 4.3. Generalization of isotropic compression paths.

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due to sample preparation technique before subjecting the samples to isotropic compression.

Since the consolidation paths with different *q*/*p'* values, according to critical state concepts, are all known to be parallel, the gereralization of compressibility behaviour both for one dimensional and isotropic stress compression behaviour is in order. With examination of more data the most probable constants can be arrived.

4.3.2 *Overconsolidated*

In retrospect (see Section 3.4.2), due to the gradual grouping of particles along the normally consolidated path, the operating specific surface area available for rebound and recompression would be lower than that of the initial value at the liquid limit water content. As discussed earlier, from microstructural considerations, this reduction is proportional to the intrinsic specific surface and during unloading from a particular stress level it is assumed that the clusters will not break down. At the engineering level, the intrinsic specific surface is indirectly reflected in the water content determined in the liquid limit test. As can be seen in the relation developed as in Equation (3.31) a generalized relation is possible encompassing both normally consolidated and overconsolidated mechanically compressed states of the same clay. In a way the average rebound-recompression path of a clay can be regarded as the normally consolidated path of a clay of low colloidal activity, i.e. low liquid limit clay. Examination of the intrinsic compression equation reveals that the ratio of compression and recompression indices, due to mechanical stress histories, are of the same order for different reconstituted clays (see Fig. 3.27). The specific advantage of this generalization is that the slopes of the compression paths prior to and after transitional stress level would aid in the analysis of soft clay behaviour as influenced by any other dominant factors, such as time and cementation subduing the effects of mechanical stress history. It has also been possible to have a critical reappraisal of different empirical equations proposed for different regions to assess the compressibility of natural clays (Nagaraj & Srinivasa Murthy 1986).

The examination of micro-fabric changes during the monotonic increase in effective stresses as discussed in Section 3.4 suggest that the mechanically recompressed paths of a clay can be analyzed as the compression paths of a clay of low colloidal activity. If this is a possibility, it should be possible to assess the recompression paths from the normally consolidated path. The examination of rebound paths of an isotropic consolidation test on a reconstituted Weald clay unloaded at different maximum pressures by Shantarajanna & Vatsala (1995) reveals that at any given *OCR*, there is a definite ratio of the stress p'_{oc} on the overconsolidated state to that of an equivalent normally consolidated state, p'_{nc}, irrespective of the value of the maximum past pressure. For example for the data of Weald clay (Fig. 4.4) for all rebound paths with p'_c of 120, 60, and 30 psi, at an *OCR* of say 4, the stress ratio $R = p'_{nc}/p'_{oc}$ is about 2.66. At any other stress level similar obser-

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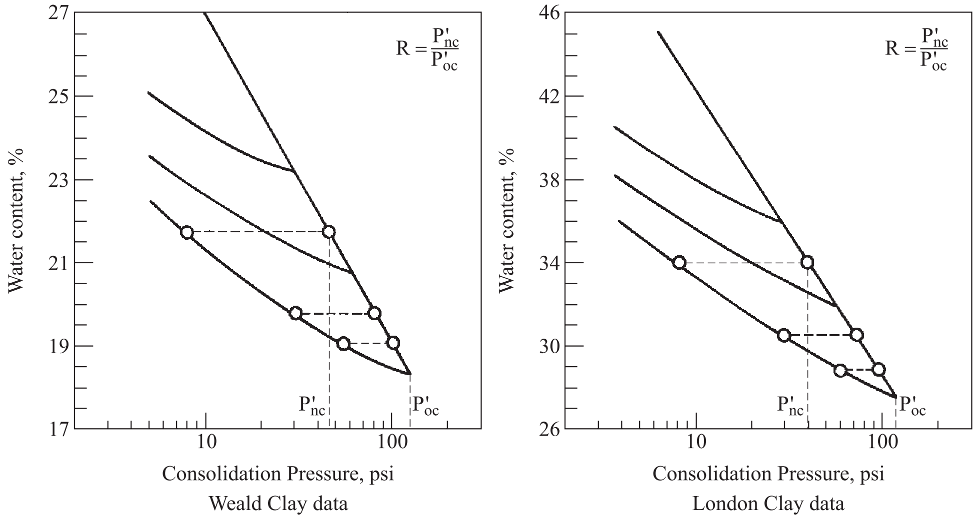

Figure 4.4. Analysis of compression and rebound paths of isotropic consolidation test on Weald and London clays.

vation can be made. The same features can also be observed for London clay. This would enable us to obtain the relation between *R* and *OCR* from isotropic consolidation test data.

Based on a purely statistical analysis of the experimental results of clays of different regions, several empirical compression and recompression indices relations, with index properties and in-situ water content and void ratios, have been proposed at different stages in the development of soil mechanics. Extensive regression analysis of the data from more than 700 consolidation test results was made by Azzouz et al. (1976) as early as 1976. It was concluded then that both compression and recompression indices can reasonably be estimated by use of a linear regression analysis involving only the void ratio. More recently Nagaraj & Srinivasa Murthy (1986), from the basic considerations of clay-water interactions, have made a critical reappraisal of compression index equations. In Table 4.1 some of these relations are presented.

Apart from the estimation of compression and recompression indices from the laboratory consolidation test, the coefficient of consolidation, C_v used to predict the rate of settlement, can also be assessed. Narasimha Raju et al. (1995, 1997) have examined the possibility of estimating C_v using the stress-state permeability relationships (Nagaraj et al. 1993, 1994a). The following is the general expression to assess the value of C_v.

$$C_v = \left\{ \frac{1 + e_L\left(1.229 - 0.102 \ln \sigma'_v\right)}{e_L} \right\}$$

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Table 4.1. Compression index equations (Nagaraj & Srinivasa Murthy 1986).

Equation	Reference	Applicability as stated in references in column 2	Applicability from above reference
$C_c = 0.007\ (w_L - 10)$	Skempton (1944)	Remoulded Clays	Normally consolidated $S_t < 1.5$
$C_c = 0.009\ (w_L - 10)$	Trezaghi & Peck (1967)	Normally consolidated, moderately Sensitive	Moderately sensitive $S_t < 5$
$C_c = 0.01\ w_n$	Koppula (1981)	Chicago and Alberta clays	Normally consolidated $S_t < 1.5$
$C_c = 0.0115\ w_n$	Bowles (1979)	Organic silt and Clays	Normally consolidated $S_t < 1.5$
$C_c = 0.015\ (e - e_0)$	Nishida (1956)	All clays	Normally consolidated $S_t < 1.5$
$C_c = 0.015\ (e - 0.35)$	Nishida (1956)	All clays	Normally consolidated $S_t < 1.5$
$C_c = 0.54\ (e - 0.35)$	Nishida (1956)	Natural soils	Normally *consolidated* $S_t < 1.5$
$C_c = 0.75\ (e_0 - 0.50)$	Bowles (1979)	Soils with low plasticity	Moderately sensitive $S_t < 5$
$C_c = 0.0046\ (w_L - 9)$	Bowles (1979)	Brazilian clays	Moderately overconsolidated
$C_c = 1.21 + 1.055\ (e_0 - 1.87)$	Bowles (1979)	Motley clays from Sao Paulo city	Highly sensitive $S_t > 5$
$C_c = 0.30\ (e_0 - 0.27)$	Hough (1957)	Inorganic silty sand or clay	Overconsolidated
$C = 0.208(e_0 - 0.0083)$	Bowles (1979)	Chicago clays	Moderately overconsolidated
$C_c = 0.156(e_0 - 0.0107)$	Bowles (1979)	All clays	Moderately overconsolidated
$C_c = 0.5\ (\gamma_w/\gamma_d^2)^{1..2}$	Oswald (1980)	Soil systems of all complexities and types	Not applicable

C_c = compression index; e_0 = initial or in-situ void ratio; w_L = liquid limit water content; w_n = natural water content; γ_d = dry density of the soil at which C_c is required; γ_w = unit weight of water; S_t = sensitivity of the clay.

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$$\times\left\{\frac{\left(3.964\times10^{-3}\right)\left(\sigma'_{v}\right)^{0.827}-0.017\ln\sigma'_{v}}{\left(\sigma'_{vo}\right)^{1.04}}\right\} \tag{4.10}$$

where σ'_v and σ'_{vo} are the consolidation pressure and preconsolidation pressure respectively in kPa and C_v in cm^2/sec. It is unnecessary to stress that the above relation is not tenable for aged cemented clays.

To obtain C_v for normally consolidated soils, substitute $\sigma'_v = \sigma'_{vo}$ in the above relation and divide the resulting C_v by a factor of 7, since the ratio between λ and κ has been found to be 7 for the data analyzed. It is to be remembered that the above estimation holds good only if the clays are in their uncemented state. Further, the above assessment is not to be taken as a substitute for existing methods of testing and computation. It is to be regarded as additional method for rechecking, with the added advantage of relatively quick assessment.

4.3.3 *Secondary compression*

As rightly stated by Schmertmann in his 25th Trezaghi lecture (Schmertmann, 1991), *'Everything on this earth has at least one thing in common – everything changes with time.'* All soils age and change. Apart from the mere passage of time, which has been identified as pure ageing, during the passage of time many other events such as fluctuations in ground water levels, desiccation, leaching and precipitation of cementing agents might mask those effects due only to mere ageing. Such combined events would normally obscure any insight into processes that could be attributed to the effects of time alone. The discussion in this section concerns only the secondary time effects on compressibility.

Terzaghi's theory of consolidation, along with subsequent refinements, has provided the conventional means for assessment of the settlements founded on clays. Laboratory experiments and field observations have shown (Lo 1961, Mesri 1973, and others) that after the excess pore pressures (hydrodynamic effects) have dissipated, compression does not stop and is generally followed by an additional compression even under constant effective stress. Such processes were earlier designated as secondary time effects and secular effects (Lo 1961). At present, in a general sense, volume changes that take place in response to the hydrodynamic effect are regarded as primary compression (consolidation) while the remaining volume change is termed secondary compression.

For situations where the total vertical stress remains constant after loading, the end of primary consolidation is best identified by the measurement of pore water pressure. However, there is now substantial experimental evidence that other time or rate dependent processes exist that are not associated with changes in effective stress. Two extreme approaches have been examined for evaluation of the compressibility of a clay layer due to load increase. One is based on the assumption that at the end of the primary consolidation, the compression curve is the same for laboratory specimens and for thicker field layers (Mesri et al. 1995). The second

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approach (Leroueil 1995) considers that clays are viscous, which implies that the compression curve depends on the strain rate and temperature. However, both approaches are corroborated by experimental evidence.

4.3.4 C_α/C_c *relationships*

The concept developed by Mesri & Castro (1987) was for analysis and assessment of secondary compression parameters. This concept is based on the observation that the magnitude of C_α with time is directly related to the magnitude of C_c with consolidation pressure. At any instant (e, σ'_v, t) during secondary compression represent the slopes of the e – log t and e – log σ'_v passing through that point. The postulate for the unique relationship between the secondary compression index and the compression index is based on the following definitions for C_α and C_c and their corresponding expressions for graphical procedure.

$$C_\alpha = \frac{\delta e}{\delta \log t} = \frac{\Delta e}{\Delta \log t} \tag{4.11}$$

$$C_c = \frac{\delta e}{\delta \log \sigma'_v} = \frac{\Delta e}{\Delta \log \sigma'_v} \tag{4.12}$$

where: e = void ratio, t = time, and σ'_v = effective vertical stress.

The graphical procedure is illustrated in Figure 4.5, using compression curves for three consolidation pressures. The value of C_α at each consolidation pressure

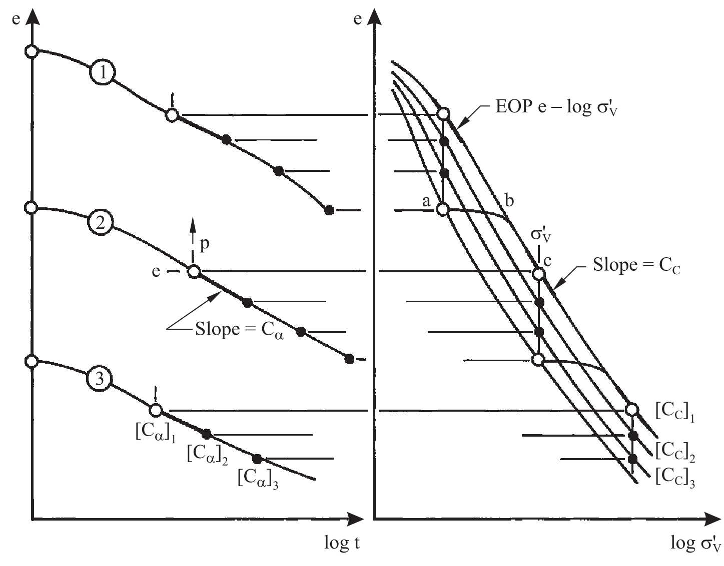

Figure 4.5. Details of graphical procedure for determination of C_α and C_c values (Mesri & Castro 1986).

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σ'_v is obtained from the linear segment of the e – log t curve immediately beyond the transition from primary to secondary compression. The corresponding value of C_c at the consolidation pressure σ'_v is obtained from the slope of end of primary e – log σ'_v curve. One log time cycle of secondary compression is required for defining the C_α that corresponds to the C_c value. According to the data provided by Mesri & Godlewski (1977) for natural clays the value of C_α/C_c ranges from 0.030 to 0.085. Whether such a variation can be attributed to time effects only, or general aging effects imply inclusion of cementation in addition to time effects, merits examination.

Normally consolidated clays that have remained under constant effective stress for long periods of time, upon subsequent loading, develop an apparent preconsolidation pressure which is significantly higher than the sustained effective stresses that acted on them. This phenomenon has been defined recently by Schmertmann (1991) as ageing preconsolidation. At the laboratory scale, the one dimensional consolidation test with long duration of secondary phase provides the basic information. Athanospoulos (1993) demonstrates a quantitative equivalence between effective stress and age induced overconsolidation. If this is a possibility, then the clay undergoes only mechanical ageing, with C_α/C_c value being constant irrespective of the type of fine-grained clay. From microstuctural considerations of normally consolidated and overconsolidated clays due only to stress history, as discussed in Section 3.4, and the conceptualised considerations of the equivalent fabric of aged clays by Hanzawa & Adachi (1983), this is a possibility (Fig. 4.6). For this postulation cluster growth due to time at constant effective stress and due to monotonic stress increase along the normally consolidated path, have to be

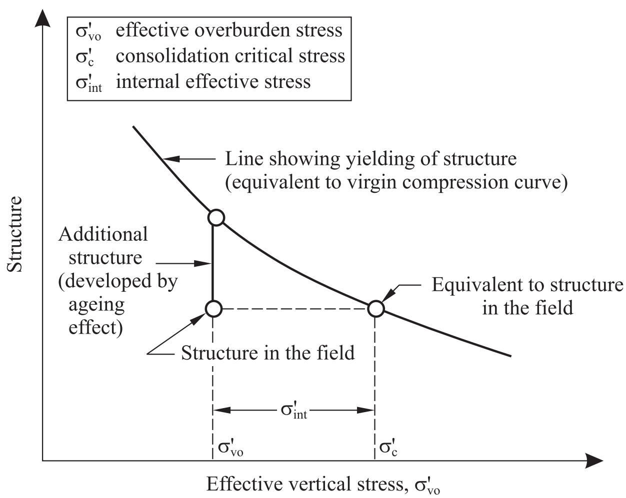

Figure 4.6. Conceptualised plot showing structure versus effective stress relation due to ageing (Hanzawa & Adachi 1983).

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identical. Sills (1995) discusses this possibility of time dependent changes due to densification of flocs, which might be responsible for different effective stress-void ratio relationships. This inference is made from the analysis of scanning electron micrographs of Stepkowska et al. (1994). If this is a possibility then the Stress-State-Time relationships reflected in C_α/C_c values can be premised to be a constant value, provided the initial state of the clay is not influenced by any natural cementation. Since most of the natural clays tested by Mesri & Godlewski (1977) are already naturally cemented, C_α/C_c values are likely to be affected by cementation. Only very limited secondary compression data on reconstituted clays with an initial water content close to the liquid limit are available for examination. The reported test data on Ariake, Hitachi and Kanda clays and Moriya silt (Yasuhara et al. 1983) indicate that C_α/C_c values are in the narrow range of variation (0.28-0.33) suggesting that an average value of 0.3 appears to be a possibility. Only the generation and analysis of extensive data on reconstituted clays might provide clarification on this point.

The clarification of the above postulations would help in the classification of soft clay deposits as aged only or aged with cementation. The intrinsic state line would still be the reference for classification. The recompression path of mechanically aged clay would practically be horizontal, and the stress level at which the compression path deviates is on the normally consolidated path of the reconstituted clay, and the compression index would practically be the same as that of virgin compression path. Any deviation in the recompression path intersecting the normally consolidated path before reaching transitional stress level with the compression index beyond yield stress level being higher than that of normally consolidated path of the reconstituted clay would be due to cementation. Then such a soft clay deposit can be regarded as aged and with combined effects of time and cementation, with the latter playing a dominant role in the engineering behaviour of such deposits. However, the postulations remain hypothetical unless reinforced substantially by extensive experimental data on time effects, using appropriate tests on undisturbed clays and on reconstituted condition of the same clays.

4.3.5 *K_0 due to stress history effects*

Predicting the in-situ stress state of soil is of significance in a wide variety of geotechnical engineering problems. So far, numerous investigations have addressed this aspect and have achieved varying degrees of success in K_0 predictions. The attempts made by Mayne & Kulhway (1982) in the analysis of extensive data and advancing the relationships between K_0 and *OCR* for normally and overconsolidated clays are noteworthy. The geostatic vertical stress can be estimated from a profile of effective overburden stress with depth. But the in-situ horizontal stress in a natural deposit is highly dependent on the geological history of the soil such as that due to ageing and cementation. It is common to represent the ratio of horizontal to vertical effective stress by the K_0 coefficient:

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$$K_0 = \frac{\sigma'_h}{\sigma'_v} \tag{4.13}$$

The simplest empirical relationship for K_{0nc} for the normal consolidated state as well as the most widely used expression for the primary loading is due to Jaky (1948):

$$K_{0nc} = 1 - \sin\phi' \tag{4.14}$$

in which ϕ' is the effective friction angle. Edil (1983) expresses that the correlation analysis of extensive data by Mayne & Kulhawy (1982) only defines the trend but does not establish the reliability of that trend for predicting either average or individual values needed for design. This would be so since time and environmental factors influence the data of soils used for statistical analysis. In fact, on the basis of the framework discussed in Chapter 3, clays subjected to mechanical stress history only from an initial state devoid of any stress history, time and cementation effects, K_{0nc} would be of constant value. This is so since, for a wide spectrum of clays of different mineralogy, the value of ϕ' is practically constant. In practice this does not happen since encountering a natural clay deposit formed only due to monotonically increased loading is a figment. Overconsolidation, because of rebound, results in higher values of K_0 than K_{0nc}. The simplest relationship between K_{0u} and OCR for the overconsolidated state has been expressed as:

$$\frac{K_{0u}}{K_{0nc}} = OCR^{\alpha} \tag{4.15}$$

in which α is an exponent defined at-rest rebound parameter for the soil. This parameter is also the slope of the line between log K_{0u} and log OCR. From detailed analysis of the collected data, and with the hypothesis that $\alpha = \sin\phi'$, Mayne & Kulhawy (1982) proposed the K_0 relation during loading and unloading as:

$$K_{0u} = (1 - \sin\phi')\, OCR^{\sin\phi'} \tag{4.16}$$

Schimdt (1983) suggested the following generalized expression for K_0 for both unloading and reloading paths.

$$K_0 = \left\{\frac{1 - \sin\phi'}{OCR_{\max} - 1}\right\}\left\{OCR_{\max} - OCR + [OCR - 1]\, OCR_{\max}^{\alpha}\right\} \tag{4.17}$$

The above relation describes a straight line between the points $OCR = OCR_{\max}$ and $OCR = 1$, i.e. between the minimum and maximum stresses.

Another aspect which merits elucidation is K_0 values as affected by time. Will $K_0 = \sigma'_3/\sigma'_1$ of a normally consolidated cohesive soil increase or decrease or remain constant during secondary compression in 1-D loading? The answer to this question has attracted the attention of the geotechnical profession for long

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(Schmertmann 1983), but it has not been, so far, categorical, although some clarity exists regarding microstructural changes with time. According to the basic microstructural model discussed in Chapter 3, cluster growth similar to stress changes along the normal consolidation path also takes place during the secondary compression phase. McRoberts (1984) basing his arguments from the cavity channel network or macropore-micropore model of De Jong (1968), also indicated this possibility. The implications of such considerations are that an equivalence of microstructure realized either due to lapse of time or due to stress changes can be envisaged. This supports the conceptualised equivalent structure considerations suggested by Hanzawa & Adachi (1983) (see Fig. 4.6). This suggests that the K_0, remains constant during secondary compression. Such considerations further reinforce Schmertmann's explanation of attributing strength gain due to secondary compression to the clay's ability to mobilize its frictional shear resistance, particularly at small shear strains after ageing (Schmertmann 1984).

The equivalent microstrucural considerations also enable assessment the *OCR* due to ageing. As seen in the figure, the additional structure developed due to ageing is a function of the effective stress, thus:

$$\sigma'_{\text{int}} = \sigma'_c - \sigma'_{vo} \tag{4.18}$$

where σ'_{int} is the internal effective stress equivalent to the additional structure developed by the ageing effect. Since this increases linearly as effective overburden stress increases, the *OCR* would be:

$$\sigma'_c = \sigma'_{vo} + \sigma'_{\text{int}} = \alpha\sigma'_{vo}$$
$$\sigma'_{\text{int}} = \sigma'_{vo}(\alpha - 1)$$

where α is *OCR*. σ'_{int} is akin to the development of latent interparticle forces discussed in Section 3.4. The above considerations would again be introspected while discussing shear strength mobilization due to stress history and mechanical ageing effects.

4.4 SHEAR STRENGTH

The ability of particulate material such as soils to support an imposed loading or to support itself is governed by the shear strength of the material. As such, assessment of the shearing resistance of soils is of primary importance in the analysis and design of stability problems. As the shear stress imposed due to loading increases with the mobilization of the shear strength of particulate media, initial effective mean principal stress also changes. Consequently, the shear strength is dependent on the level of confining stress and its change during shearing. When a metal, a highly bonded system, is sheared it deforms in a manner similar to soils but the volume remains constant. This forms an important distinction between the response of the

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particulate system and that of bonded materials, like metals. In the case of fine-grained soils, due to faster rates of loading, pore water pressure mobilizes which control, the level of mobilization of shear strength. The soil fabric acquires the ability to withstand imposed shearing stresses by compatible volume changes.

4.4.1 *Normally consolidated*

It has been inferred from critical state concepts that for saturated clays, devoid of any stress history effects, the compression lines in e versus log p' plot with different q/p' values will lie parallel to each other and also they are parallel to the void ratio versus shear strength line. Since the (e/e_L) versus log p' relation is a generalized one, the possibility of a similar generalization to obtain (e/e_L) versus log q plot merits examination.

The two practical tests to determine the liquid limit are analogous to shear tests, with the number of blows N or the depth of penetration D being a measure of the undrained shear strength at that water content. Flow lines obtained with the number of blows for liquid limit determination, are normalized with a water content corresponding to the liquid limit of each of the clays. The resulting plot has data points virtually falling on a unique line having the relation of the form:

$$\frac{w}{w_L} = a - b\log N \tag{4.19}$$

A similar attempt to generalize the cone penetration test results (Bindumadhava 1985, Nagaraj et al. 1987) has indicated that water content versus cone penetration plots can also be normalized (Fig. 4.7) using their respective water contents at 20 mm penetration, with a resulting relation of the form

$$\frac{w}{w_L} = 0.77 + 0.012D \tag{4.20}$$

where D refers to the cone penetration in mm.

The possibility of generalization of the ultimate shear strength of soils at lower water contents/higher consolidation pressures has been examined (Srinivasa Murthy et al. 1988) by the analysis of the strength data of Henkel (1960) on Weald and London clays, together with the data on two other soils. The isotropic consolidation data for these soils together with the generalized plot is shown in Figure 4.8. All the data points fall within a narrow band and the best linear fit equation is of the form

$$\frac{e}{e_L} = 0.932 - 0.21\log q_f \tag{4.21}$$

A similar exercise on the mean principal stress has yielded an equation of the form

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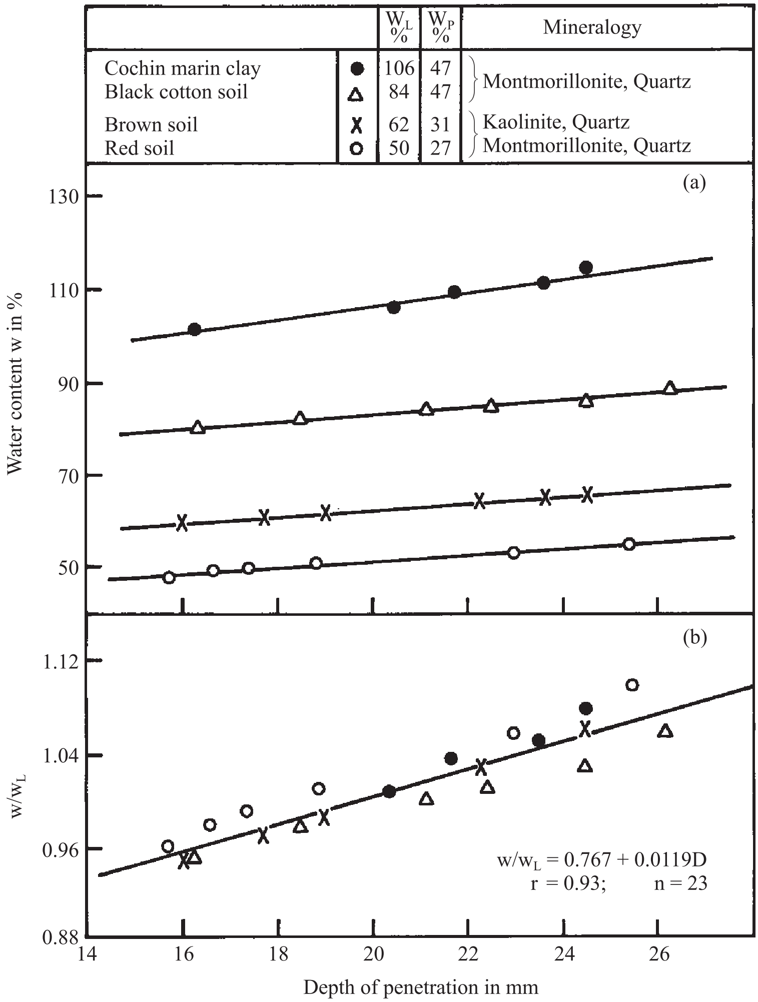

Figure 4.7. Flow curves of soils and their generalization from the fall cone test. (Nagaraj et al. 1994).

$$\frac{e}{e_L} = 0.952 - 0.21\log p'_f \tag{4.22}$$

Both Equations (4.21) and (4.22) have high correlation coefficients in the range of 0.98 with low error of estimate (0.015). This clearly demonstrates that the ultimate shear strength behaviour of normally consolidated fine grained soils can be generalised. This implies that the mobilized shear strength is a function of the microstructure reflected in e/e_L values. At the same void ratio, while the shear strength is different for different clays, at the same e/e_L value the shear strength

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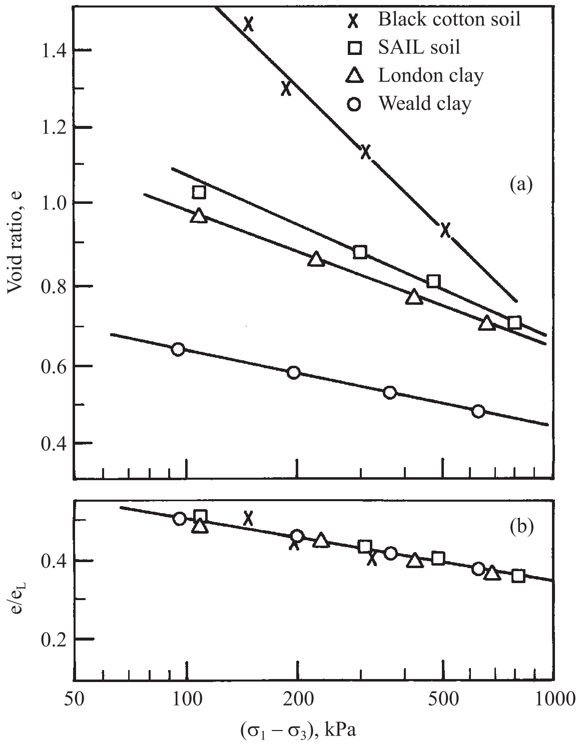

Figure 4.8. Void ratio versus shear strength plots and their generalisation (Data Srinivasa Murthy et al. 1988).

has a unique value while the shear behaviour is controlled by the interacting particulate characteristics of the soil. It has been shown (Srinivasa Murthy et al. 1991), that during shearing the spherical pores are distorted to an ellipsoid to maintain equilibrium under deviator stress conditions. The size and shape of the ellipsoid is dictated by the mean principal stress conditions. The intercluster spacing of clusters placed on the periphery of the ellipsoidal pore depends on the effective mean principal stress. This in turn induces the same effective stress even in the large pore. Since all these manifestations are due to changes in the applied stress which is reflected in the stress conditions in the continuous fluid phase, the generalisations have been possible.

The shear strength is conventionally characterised by cohesion, *c*, and friction, ϕ, components. The cohesion component is stress independent while the frictional component is normal stress dependent. At present, it is accepted that there is truly nothing like cohesion unless the clays acquire the same due to natural or induced cementation. The apparent cohesion component normally observed in overconsolidated states is only due to the mode of analysis of the strength data and, in the case of the partially saturated condition, is due to capillary stresses. These strength components are influenced by several factors including drainage conditions and the method of testing.

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The consequence of the two plots, e/e_L versus log q and e/e_L versus log p'_f being parallel is far reaching. In the development of the critical state concepts, the ratio of (q/p') at failure, which is found to be constant for a soil, is referred to as the friction factor. This factor $(q/p')_f = M$, is related to the angle of friction in the form

$$M = \frac{6\sin\phi}{(3-\sin\phi)} \tag{4.23}$$

Now from Equations (4.21) and (4.22), e/e_L can be eliminated to obtain $(q/p)_f = M = 0.803$. Since this has resulted from generalised equations, it implies that it must be practically a unique value for all fine-grained soils, i.e., the effective angle of internal friction is about 21°. Figure 4.9 depicts the Modified Mohr diagram for the data of soils shown in Figure 4.8. It is clear that a value can be assigned to the intrinsic friction angle for fine-grained soils. Table 4.2 indicates the data of liquid limit and factor M, of five proven normally consolidated clays presented by Schofield & Wroth (1968). The variation of the value of M from 0.845-1.02 (ϕ' from 22°-26°) is quite a small range for as wide a variation in the liquid limit

Table 4.2. Liquid limit, M and ϕ^0 values of normally consolidated clays.

Soil	w_L %	M	ϕ^0
Klein Belton	127	0.845	21.74
Weiner Tegel	47	1.010	25.61
London clay	78	0.888	22.75
Weald clay	43	0.950	24.21
Kaolin	74	1.020	25.84

Data from Schofield & Wroth (1968).

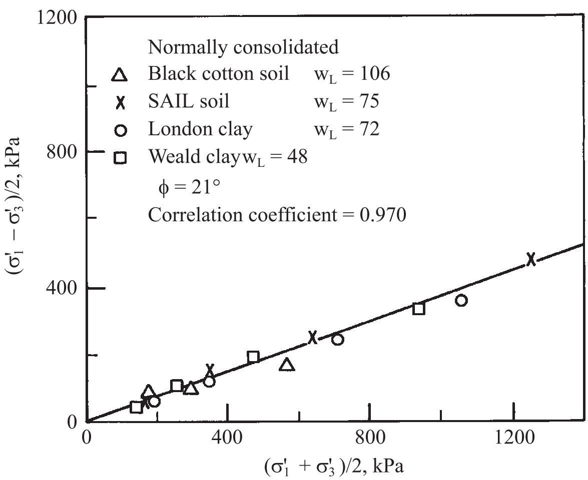

Figure 4.9. Modified Mohr diagram for the data of Figure 4.8.

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values from 43 to 127%. It should be remembered that such a nearly constant value is a possibility provided the behaviour is governed predominantly by interacting particulate considerations, and effects of time and environment are not prevalent.

However in literature values of ϕ varying from 12^o to 35^o have been reported. The deviations from the intrinsic value could arise because:

1. the state of stress may not truly represent effective stress only on account of partial saturation,
2. the critical state might not have reached as in the overconsolidated states, and
3. the strain rate may have profound influence on the friction parameter.

The possibility of unique friction factor, which is intrinsic in nature for fine grained soils, where only the interacting particulate characteristics prevails, is reflected from the assumptions made in deriving the gereralised equations. For the case of a continuous water phase with inter-particle interactions through diffuse double layer, the shearing phenomena could be independent of the mineralogy of sheet minerals. Trollope (1960) has conceptually explained the mobilization of shearing resistance in such interacting particulate media as early as 1960. This has been designated as colloidal friction. This is analogous to the case of two magnets of like poles placed at a distance apart with the aid of an external force requiring a certain amount of horizontal force for lateral displacement. Reverting to the possible micro-fabric pictured in Figure 3.8, to sustain a deviator stress q and still keep the stress in the pore fluid isotropic the configuration of the micropore has to tend towards elliptical. The magnitude of shearing resistance is a reflection of resistance to displacement between interacting particle units and the number of such units constitute the micro-fabric configuration already discussed. As such the micro-fabric considerations provide better insight into the shearing resistance mobilized than considerations of the equilibrium void ratio of the soil system. Since the changes in the micro-fabric with effective consolidation stress is of the same pattern for different soils the frictional resistance would also be of the same order for different fine-grained soils. This being so, it is also possible to comprehend the following two modes of observed behaviour (Figs 4.10 and 4.11):

1. Two soils can mobilize different strengths at the same void ratio. It is possible that two different clays equilibrate to the same void ratio at different consolidation pressures. The corresponding e/e_L values are different. As such the shear strength is different as the variation of shearing resistance is a function of the microfabric, i.e. e/e_L values rather than a mere function of the void ratio changes in different soils (Fig. 4.10).
2. Two soils can have the same strength at widely different void ratios at the same consolidation pressure. The corresponding e/e_L versus are the same. Hence shear strength is the same since e/e_L versus shear strength bears unique relation (Fig 4.11).

Within the framework of the above general observations some of the empirical strength relations used in engineering practice are re-examined.

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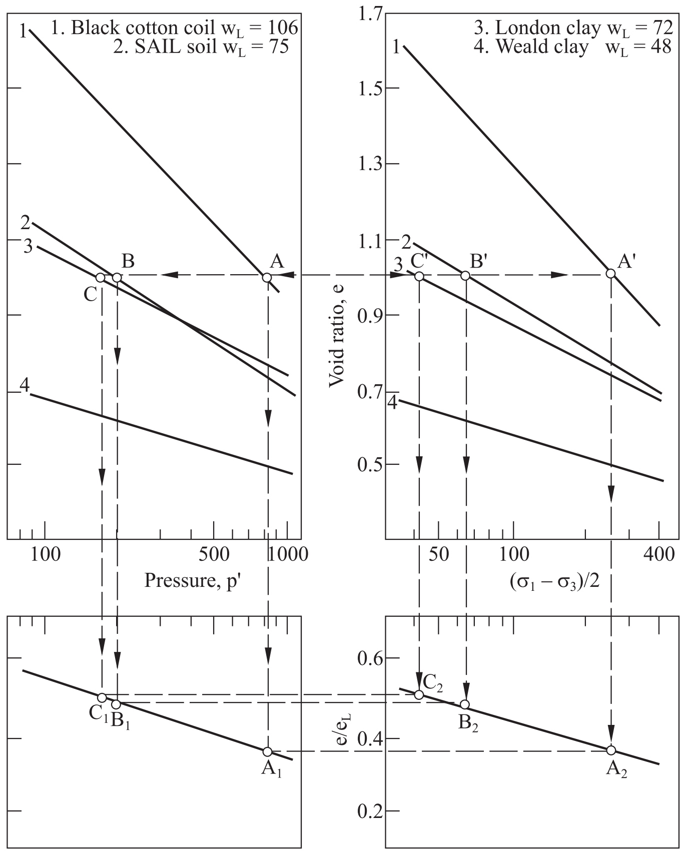

Figure 4.10. Shear strength of different soils at the same void ratio (Nagaraj et al. 1994).

Skempton & Henkel (1953) and Skempton (1954) based on their observation of field data, propose an empirical relation between c_u/p' being the ratio of undrained strength c_u to effective overburden pressure p' and plasticity index I_p of the form

$$\frac{c_u}{p'} = 0.11 + 0.0037 I_p \tag{4.24}$$

There have been several studies to examine the possibility of such a correlation. Bjerrum (1954), Wu (1958), Osterman (1960), Leonards (1962), Bishop & Henkel (1962), based on their observations, in general support the above correlation. The analysis of data by Metcalf & Townsend (1960) and Cox (1970) contradicts the possibility of such a relation. In fact the analysis of data by Sridharan & Narasimha Rao (1973) indicates that c_u/p' decreases with increase in I_p. Kenney (1959, 1960) doubts the very possibility of having a relation between the c_u/p' ratio for undisturbed soils and two inferential parameters ($I_p = w_L - w_P$) determined on re-

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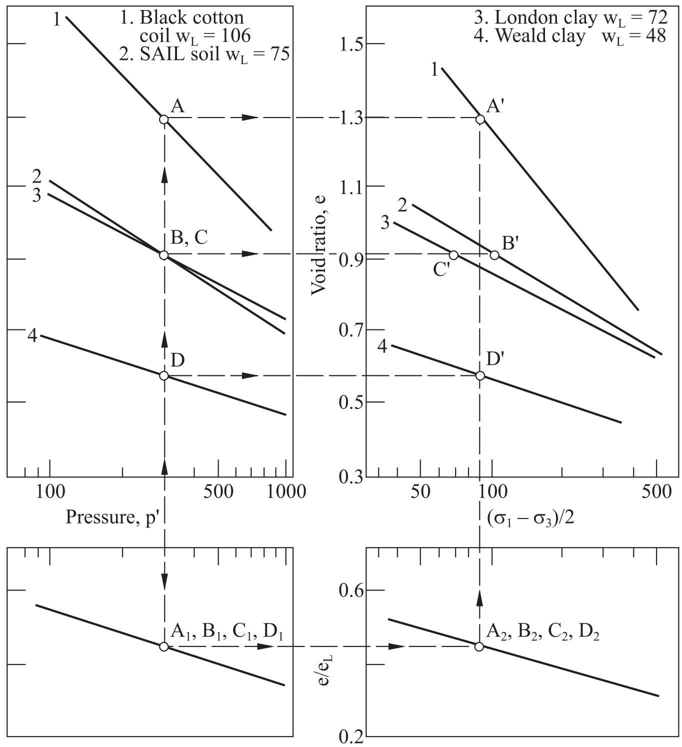

Figure 4.11. Shear strength of different soils at the same consolidation pressure (Nagaraj et al. 1994).

constituted soil. Examination of the above relation with the framework discussed above indicates that, since the liquid limit and plastic limit water contents reflect the same range of variation in the equilibrium effective stress and also the same order of variation in shear strength, this ratio is likely to be constant with an increase in I_p. This is so, provided time and environmental effects are absent. In the case of soils whose behaviour is dictated by time and environment the trends in the variation of c_u/p' and I_p cannot be uniquely defined.

Alternatively, the relations between the generalized state parameter e/e_L and consolidation pressure σ_v', and undrained strength c_u,

$$e/e_L = a - b \log \sigma_v'$$

and

$$e/e_L = c - b \log c_u$$

yield parallel lines of the same slope, b, for normally consolidated soil devoid of time and cementation effects. Subtracting one from the other the interrelation would be of the form:

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$$\frac{c_u}{\sigma'_v}=10^{-\left(\frac{a-c}{b}\right)} \tag{4.25}$$

In Equation (4.25), the constants *a*, *b*, and *c* have been arrived at so as to take care of variations in the type of clay. Hence c_u/σ'_v is constant for different soils. For the ideal case, i.e. devoid of time and environment, c_u/σ'_v does not vary with the plasticity index. Prediction of c_u/σ'_v from index properties is further examined from critical state concepts. The expression developed by Schofield & Wroth (1968) is of the form

$$\left\{\frac{c_u}{p'}\right\}=\frac{1}{2}M\exp\left[-\frac{(\lambda-\kappa)}{\kappa}\right] \tag{4.26}$$

Values computed from Equation (4.26) and corresponding values using Skempton's equation for five soils reported by them are indicated in Table 4.3. It can be seen that the actual values of c_u/σ'_v are fairly constant, with an average of 0.24 as against 0.2025 to 0.4467 given by Skempton's equation, for the reported values of λ, κ, M, w_L and w_P. In Equation (4.26), p' is isotropic consolidation pressure. For K_0 consolidation pressures, values of c_u/σ'_v would be slightly higher but will be in the same range for different clayey soils. Further, Skempton's relation predicts higher strength for soils of higher plasticity index at the same confining pressure which is contrary to the generally observed behaviour. Further, Srinivasa Murthy et al. (1986) have shown that for sensitive soils Skempton's relation is very unlikely to be tenable since c_u is of the same value for confining pressures up to yield stress, σ'_y and hence c_u/σ'_v varies with the level of σ'_v, even for the same soil.

Based on the observations of a definite trend in the relation between liquidity index, I_L and remoulded strength of several clays, (Skempton & Northey 1953, Schofield & Wroth 1968, Houston & Mitchell 1969), Wroth & Wood (1978) have linearly idealized this variation. The following micro-mechanistic analysis has been made to examine the possibility of this linearization.

Table 4.3. Comparison of c_u/c'_v values.

Soil	Klein Belton	Weiner Tegel	London clay	Weald clay	Kaolin
w_L	27	47	78	43	74
w_P	36	22	26	18	42
λ	0.356	0.122	0.161	0.093	0.260
κ	0.184	0.026	0.062	0.035	0.050
M	0.850	1.010	0.890	0.950	1.020
c_u/p' (Eqn 4.23)	0.261	0.230	0.240	0.245	0.227
c_u/σ'_v (Eqn 4.21)	0.447	0.203	0.271	0.203	0.228

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The liquidity index, which is the ratio of the difference in natural water content and plastic limit to the plasticity index, can be expressed in terms of inter-cluster spacing, *d*, as:

$$I_L = \frac{w_n - w_P}{w_L - w_P} = \frac{kd_n - kd_P}{kd_L - kd_P} = \frac{d_n - d_p}{d_L - d_P} \tag{4.27}$$

Since the functional form '*k*' which reflects the soil type vanishes, and d_L and d_P are constants for all soils, I_L reduces to a function of inter-cluster spacing alone.

As seen earlier, the generalised state parameter e/e_L is also a function of '*d*' and hence there must be a relation between I_L and e/e_L. From the data of consistency limits as reported by various sources and at different water content values, Srinivasa Murthy et al. (1986) have shown that the relation between the two can be expressed by the equation

$$I_L = 1.55\frac{w}{w_L} - 0.56$$

with a correlation coefficient of 0.992 (see Table 4.4). Similarly the I_L versus *q* is also tenable and is of the form

$$I_L = 0.89 - 0.33\log q \tag{4.28}$$

It is obvious that e/e_L is a simpler parameter to correlate the mechanical properties than I_L, because this does not involve the additional parameter plastic limit.

4.4.2 *Overconsolidated*

It has been shown in earlier sections that shearing involves primarily the interactions in the interacting fluid phase, and hence it is unlikely to have any basic difference in the shearing resistance of normally and overconsolidated states of the same clay. But still there is a difference in the level of interaction because of the reduced operating specific surface due to aggregation of clay particles in monotonic loading, and stress levels being lower than the stress levels reached during monotonic loading. In addition, it is likely that the clusters formed during consolidation may undergo gradual breakdown with shearing. At large strains, i.e. at critical states, when the dismembering of clusters is complete, the intrinsic angle of shearing will be the same both for normally and overconsolidated states provided time and environmental effects are absent. To reinforce this possibility, shear test results on Weald and London clays (Henkel 1960) overconsolidated to a maximum pressure of 120 psi (846 kPa) and those of SAIL soil and Black cotton soil (Srinivasa Murthy et al. 1988), overconsolidated to a maximum pressure of 800 kPa, have been examined. Figure 4.12 shows the modified Mohr diagram at large strains. It is clear that the angle of shearing resistance is of the same order as that observed in the case of the normally consolidated states of these clays, i.e. 21°. The shear strength at critical

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Table 4.4. Data extracted from reference, Komarnik & David (1969) for the relation in Equation (4.25).

Liquid limit w_L %	Plastic limit w_P %	Natural water content w_n %	w_n/w_L	Liquidity, I_L
45	18	35	0.778	0.630
		30	0.667	0.444
50	20	35	0.700	0.500
		23	0.460	0.100
94	32	56	0.596	0.387
		33	0.351	0.016
85	29	72	0.847	0.768
		64	0.753	0.625
78	25	55	0.705	0.566
		22	0.282	–0.057
81	29	43	0.531	0.269
		65	0.802	0.692
42	17	38	0.905	0.840
		30	0.714	0.520
69	25	45	0.652	0.455
		29	0.420	0.091
86	29	52	0.605	0.403
		77	0.895	0.842
57	23	50	0.877	0.794
		30	0.526	0.206
111	35	33	0.297	–0.026
		69	0.622	0.447
73	25	65	0.890	0.833
		33	0.726	0.583
81	28	66	0.815	0.717
		74	0.914	0.868
92	30	80	0.870	0.806
		35	0.380	0.081

state can be assessed from the same relation given by Equation (4.21) provided the value of e/e_L is known at that state. The initial value of this parameter by itself is the value at the critical state for undrained shearing. For drained tests, however, shear strength can be computed directly using the known stress paths and the common Mohr diagram (Fig. 4.12). There are many engineering situations, such as problems associated with excavations and earth retaining structures, where peak stress assessments are needed. This value for a soil depends upon the overconsolidation ratio and the maximum past pressure. Just as the consolidation rebound lines of different soils could be generalised in terms of a common slope, the peak shear strength data may also be amenable for generalisation. Analysis of e – log q data of

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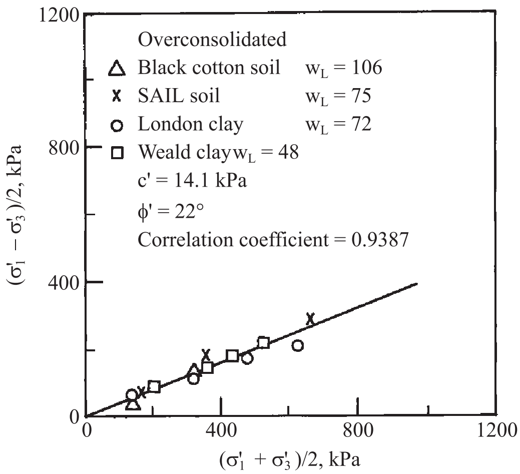

Figure 4.12. Modified Mohr diagram for overconsolidated states at large strains (Srinivasa Murthy et al. 1988).

SAIL soil and Black cotton soil consolidated to a maximum pressure of 800 kPa results in a constant slope of 0.076 e_L. To build up the necessary authenticity for such relations, analysis is needed, with more data specifically generated for this purpose.

4.4.3 *Shear strength due to ageing.*

Soft clay deposits have generally been aged for hundreds to thousands of years since deposition and consequently they possess an additional strength developed by the ageng effect. Since natural cementation is also possible over the same time, it is inevitable that the strength and compressibility of such deposits reflect combined effects. It is necessary to identify which of the effects is dominant and control the behaviour of soft clay deposits. The published compressibility and strength data of undisturbed clay available is not adequate to characterise the effects of time only. To overcome this difficulty the conceptualized approach of Hanzawa (1983) (see Fig. 4.6) is re-examined. The recompression paths of aged undisturbed clay in relation to the compression path of the reconstituted condition of the same clay provide the clue. If the clay has experienced only stress history effects then the recompression paths would exhibit slopes within and beyond preconsolidation pressure, according to the paths shown in Figure 3.28. Despite the fact that most of the data available to examine the compression paths of clays relate to undisturbed clays, it is not completely valid for substantiation. If the additional structure developed due to mechanical ageing the equivalent microstructure on the normally consolidated path as visualised by Hanzawa (1983) implies that the recompression path has to be practically horizontal and, after reaching the critical stress σ'_c, the compression path has to follow that of the normally consolidated path. In most of the data generated by Mesri & Godleweski (1977) the recompression path is far flatter than that due to the overconsolidation stress history. The compression path often crosses the intrinsic state line and the compression index

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beyond the critical stress is much steeper than that of the intrinsic compression value. This is very much a possibility since the behaviour of most of the aged clays are masked by natural cementation effects.

The undrained strength of an aged clay devoid of cementation is composed of two strength components as visualized by Hanzawa & Kishida (1981), Hanzawa & Adachi (1983) (Fig. 4.13).

1. S_{un} is the strength developed by the primary consolidation under the effective stress acting on the clay as an external stress such as σ'_{vo}.
2. S_{ua} is the strength developed by ageing.

$$S_{uf} = S_{un} + S_{ua} = \frac{S_{un}}{\sigma'_v} \times \sigma'_c \tag{4.29}$$

where S_{un}/σ'_c is the undrained strength ratio in the normally consolidated state.

On the premise that the effect of ageing is akin to overconsolidation due to single cycle reduction in effective stress, Hanzawa (1983) attributes the enhancement of shear strength to an increase in cohesive bonding between the particles.

The shear strength, τ_f in terms of effective stress is given by

$$\tau_f = c' + \sigma' \tan\phi'$$

c' is apparent cohesion in terms of effective stress, σ' is effective normal stress acting on the failure plane, ϕ' is the angle of shearing resistance in terms of effective stress.

Since τ_f corresponds to S_{uf}, it is suggested that c' and $\sigma' \tan\phi'$ are equivalent to S_{un}, and S_{ua}, respectively. It is also suggested that effective normal stress can be divided into external effective normal stress σ'_{ext} and internal effective stress σ'_{int}. The value of $\tan\phi'$ will be constant because it is equivalent to the S_{un}/σ'_v value. The shear strength, τ_f can be regarded as

$$\begin{aligned} \tau_f &= (\sigma'_{int} + \sigma'_{ext}) \tan\phi' \\ c' &= \sigma'_{int} \tan\phi' \end{aligned} \tag{4.30}$$

Schmertmann (1983) reiterates, on the basis of his earlier detailed investigations (Schmertmann 1976, 1981), that very strong evidence exists to indicate that little, if

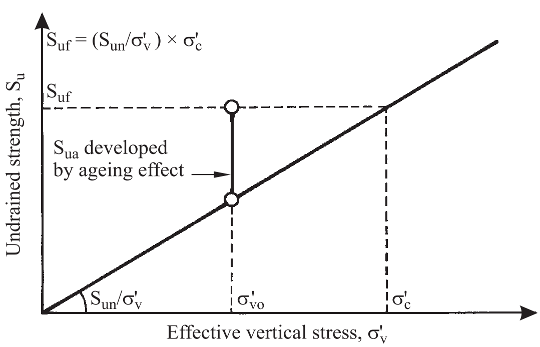

Figure 4.13. Conceptualized plot showing undrained strength vs effective vertical stress for aged clay (Hanzawa & Adachi 1983).

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any, of the strength increase due to laboratory ageing results from increasing bond cohesion. On the contrary, there is strong evidence to indicate that the strength gain results from a soil structure change that increases the ability of clay to mobilize its frictional resistance, particularly at small strains after ageing. If this is a possibility then, micro-mechanistically, particle clustering due to ageing and due to monotonically increasing stresses would be of the same phenomenon, the only difference being stress levels reckoned to account for ageing effects and due to maximum past consolidation stress levels. Since there is little experimental evidence to substantiate this, the micro-mechanistic explanations can be regarded at this juncture as conjectural.

4.4.4 *Normalized soil parameter (NSP) concept*

With the exception of very stiff, highly consolidated clay deposits, the $\phi = 0$ method of analysis is used to examine the stability of structures founded on saturated clay deposits. It is assumed that no drainage occurs during the loading period and that the undrained strength s_u of the clay remains constant irrespective of the applied stresses. Thus the usual limiting equlibrium stability analysis uses $c = s_u$ with $\phi = 0$, with the undrained strength to be more or less a unique function of water content. The value of the undrained strength could be reliably determined from field vane tests or from widely used laboratory unconsolidated-undrained triaxial compression, *UU*, or unconfined compression, *U*, tests.

The normalized soil parameter concept originates from the observation that the results of laboratory tests on clay samples with the same overconsolidation ratio, but with different consolidation stresses (and therefore different maximum past pressures, σ'_{vm}), exhibit very similar strength and stress-strain characteristics when normalized with respect to the consolidation stress (Fig. 4.14) (Ladd & Foott 1974). The most frequently used normalized soil parameter for stability analysis is s'_u/σ'_{vo}, in which σ'_{vo} is the in-situ vertical effective stress. This is equivalent to the c_u/p' ratio so often quoted in geotechnical engineering literature. It is to be remembered that this normalization is possible if the clay exhibits particulate behaviour. However, tests on naturally cemented clays which possess a high degree of cementation will not exhibit normalized behaviour due to their non-particulate nature.

4.4.5 *SHANSEP – practical implications*

The stress history and normalized soil engineering properties (SHANSEP) method developed and described by Ladd & Foott (1974) and Ladd et al. (1977) consists of evaluating the stress history of the clay deposit by evaluating the σ'_{vo} and σ'_{vm} (maximum past pressure) profiles to determine the *OCR* variation through the deposit, and then applying the appropriate NSP values to give representative soil properties for design purposes. The basic steps involved in this method are:

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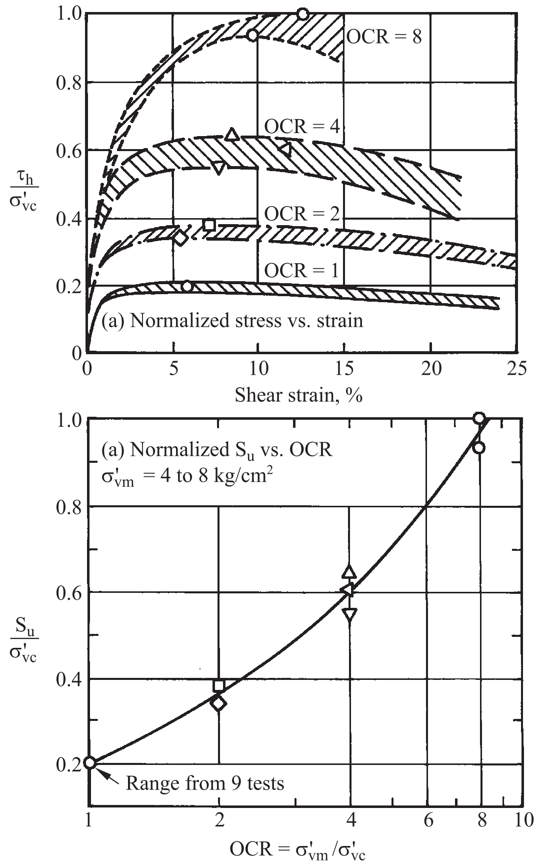

Figure 4.14. Normalized stress-strain and strength data for resedimented Boston Blue clay (Ladd & Foott 1974).

1. On the basis of boring logs, the soil profile is subdivided into different depths.
2. Procure high quality undisturbed samples at various locations and from different depths.
3. Samples are first consolidated well beyond the in-situ preconsolidation pressure and maintained at these conditions or rebounded to varying *OCR* before being submitted to undrained shear tests.

Ladd et al. (1977) has shown that the results can generally be written as follows:

$$\left\{\frac{\tau_{fu}}{\sigma'_{vc}}\right\}_{OC} = \left\{\frac{\tau_{fu}}{\sigma'_{vc}}\right\}_{NC} OCR^m \tag{4.31}$$

in which m is a parameter approximately equal to 0.8. Ladd (1991) has specified that SHANSEP technique is strictly applicable only to mechanically overconsolidated and truly normally consolidated soils exhibiting normalized behaviour. As extensive data generated on natural soft clays indicate that most of them are structured (Mesri 1975, Tavenas & Lerouiel 1985, Burland 1990), it appears that SHANSEP might have relatively limited applicability and scope in general.

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4.5 CONSTITUTIVE RELATIONS

As discussed in Section 1.7, the critical state theory was developed as a general stress-strain theory for normally consolidated and lightly overconsolidated clays which are treated as an isotropic, elasto-plastic, strain-hardening material. This is the simplest model in which the yield surface was derived theoretically on the assumption of pure frictional dissipation of plastic energy during shearing. The soil is assumed to have a flow rule which satisfies the normality condition, implying that the plastic strain increment vector is everywhere normal to the yield locus. This model underpredicts the stresses or overpredicts strains. In order to overcome this limitation, Burland (1965) suggested some modifications, based on an alternative energy dissipation function, replacing the bullet shaped surface of the original Cam-clay model with an elliptical shape. This was later extended by Roscoe & Burland (1968) to a model known as the modified Cam clay model (Fig. 4. 15). In this model, energy dissipation during plastic volume changes was included.

4.5.1 *The modified Cam clay model*

The experimental yield locus forms a boundary of stress states such that the strains for the stress paths wholly within this boundary are recoverable and small. Stress states crossing the yield locus will cause large irrecoverable plastic strains. The following are the pertinent equations for obtaining the stress-strain relationships.

For axial symmetry ($\sigma'_2 = \sigma'_3$), the yield locus is defined by the equation:

$$p'_{oy} = p'\left\{1 + \frac{(q/p')^2}{M^2}\right\} \tag{4.32}$$

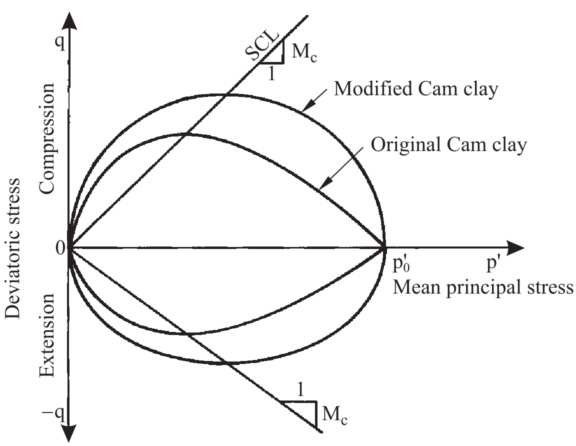

Figure 4.15. Schematic diagram showing original and modified Cam clay yield curves.

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where p'_{oy} is the intersection of the yield locus with the isotropic consolidation line ($\sigma'_1 = \sigma'_2 = \sigma'_3$) in the ($p'$, q') plane. P'_{oy} can be obtained experimentally from an isotropic consolidation test. M is the friction factor as defined earlier.

The total volumetric strain increment, dv, resulting from an increment of stress causing yield is the sum of a recoverable component and an irrecoverable component of strains. The recoverable component, caused by an increment in mean normal stress, $\delta p'$, is:

$$\delta v^r = \frac{K}{(1+e)p'}\delta p' \tag{4.33}$$

and the irrecoverable component is:

$$\delta v^p = \frac{\lambda - \kappa}{1+e}\left\{\frac{2q/p'}{M^2 + (q/p')^2}\left(\delta q - \frac{q}{p'}\delta p'\right) + \delta p'\right\}\frac{1}{p'} \tag{4.34}$$

The total volumetric strain increment is then:

$$\delta v^p = \frac{\lambda - \kappa}{1+e}\left\{\frac{2(q/p')}{M^2 + (q/p')^2}\left(\delta q - \frac{q}{p'}\delta p'\right) + \delta p'\right\}\frac{1}{p'} \tag{4.35}$$

The total volumetric strain increment is then:

$$\delta v = \frac{1}{1+e}\left\{(\lambda - \kappa)\frac{2(q/p')}{M^2 + (q/p')^2}\left[\delta q - \frac{q}{p'}\delta p'\right]\frac{1}{p'} + \lambda\frac{\delta p'}{p'}\right\} \tag{4.36}$$

The parameters λ and κ are the slopes of the isotropic consolidation line and swelling line respectively. For all practical purposes, they can be expressed as:

$\lambda = 0.434\ C_c$ and $\kappa = 0.434\ C_r$

where C_c is compression and C_r rebound indices.

For q/p' constant or $d(q/p) = 0$ being one-dimensional compression, Equation (4.33) reduces to Terzaghi's equation:

$$\left\{\frac{\lambda}{1+e}\right\}\frac{\delta p'}{p'} = \left\{\frac{0.434C_c}{1+e}\right\}\frac{\delta\sigma'_v}{\sigma'_v} = \delta\varepsilon_v \tag{4.37}$$

where $d\varepsilon_v$ is the increment of vertical strain. When we put $\delta v = 0$ in Equation (4.36) the equations for undrained stress paths or constant void ratio contours can be obtained.

For the shear strain increment, it is assumed that the elastic shear strain component is zero, i.e. $\delta\varepsilon_r = 0$ and $\delta\varepsilon = \delta\varepsilon_p$. The increment of plastic shear strain is made up of two components:

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$$\delta\varepsilon^p = \left(\delta\varepsilon^p\right)_{v^p} + \left(\delta\varepsilon^p\right)_{q/p'}$$

$$or: \ \delta\varepsilon^p = \left(\frac{d\varepsilon^p}{d\left(q/p'\right)}\right)_{v^p} \delta\left(\frac{q}{p'}\right) + \left(\frac{d\varepsilon^p}{dv^p}\right)_{q/p'} \delta v^p \tag{4.38}$$

The first component in Equation (4.38) yields a plastic shear strain which is independent of the plastic volume change and is only a function of the stress ratio, q/p'. Experimentally, it can be obtained from undrained triaxial tests by establishing a relationship between q/p' and $\varepsilon^p{}_v$. The second component arises from the shear distortion caused by a state path on the state boundary surface. During yield, the plastic strain increment ratio is uniquely related to the stress ratio, q/p', by the flow rule:

$$\left\{\frac{d\varepsilon}{dv^p}\right\}_{q/p'} = \frac{2\left(q/p'\right)}{M^2-\left(q/p'\right)^2} \tag{4.39}$$

Combining Equation (4.39) with Equation (4.35) provides an expression to compute the components of the plastic strain increment.

$$(\delta\varepsilon^p)_{q/p'} = \left\{\frac{\lambda-\kappa}{1+e}\right\}\left\{\frac{2\left(q/p'\right)}{M^2-\left(q/p'\right)^2}\right\}$$
$$\times\left\{\left(\frac{2\left(q/p'\right)}{M^2+\left(q/p'\right)^2}\right)\left(\delta q-\frac{q}{p}\delta p'\right)+\delta p'\right\}\frac{1}{p'} \tag{4.40}$$

It should be noted that the relation (Equation 4.35) to compute the volumetric strain increment and relation (Equation 4.40) for the plastic strain increment are applicable for stress changes which cause the soil to yield. Even this model underpredicts the strains, although the predicted undrained stress paths agree well with experimental results. It is to be remembered that no mathematical model can describe the complex behaviour of natural soils under all conditions and hence a compromise is necessary for simplicity, generality and compliance with the basic principles of modelling. Hence this necessitated further probing and formulation of constitutive relations to provide realistic predictions.

4.5.2 *Revised Cam clay model*

In the Cam clay models and other plasticity models the emphasis was on macrobehaviour rather than considering processes at the microlevel. No distinction was made between the mode of stress transfer in coarse- and fine- grained soils. With the fond hope of developing more appropriate constitutive modelling, a microlevel examination of the various processes the fine-grained soil particulate material undergoes during compression and shearing has been made (Srinivasa Murthy et al. 1991). It has been generally recognized that the physical character-

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istics of particulate soil material is altered as the stress changes, creating each time an altogether new soil material with its own plastic properties. Consistent with the examination of behaviour at the microlevel in fine-grained soils as discussed in Chapter 3, the plastic compressions in soils have been assumed to result from the grouping of particles into larger clusters and that the elastic compressions result from the decrease in the intra-cluster spacing between the clusters. During shearing, these clusters gradually get dismembered, releasing the locked-in energy. In fact, Dafalias (1986) remarks that for realistic predictions, a constitutive model should consider the release due to dismembering of particles within the clusters. The effect of such dismembering has been incorporated into the original Cam clay model, identified as the 'Revised Cam Clay Model' (Srinivasa Murthy et al. 1991).

For analysis, it is necessary to know the magnitude and the rate of additional stress induced by dismembering. For explanation, reference is made to the specific volume variation during normal and overconsolidation stress paths, illustrated in Figure 4.16. Point B, by the dismembering of clusters under constant volume, reaches point E on the normally consolidated line with effective pressure p_e'. Hence, the difference $(p_e' - p')$ is the additional pressure that would be induced by dismembering.

However, the breakdown of clusters does not occur in one step during shear, but happens gradually with shear strain. Instead, hypothetically, if the stability of clusters can be maintained during shearing, the soil at the overconsolidated state B would reach a critical state at B' in an undrained test, and the normally consolidated state E at the same ratio would reach $E' = B_c$.

If the clusters are dismembered after reaching the critical state at B' to the same level as at B_c and induced stress at the critical state would be, $P'_{BC} - P'_B$ is the

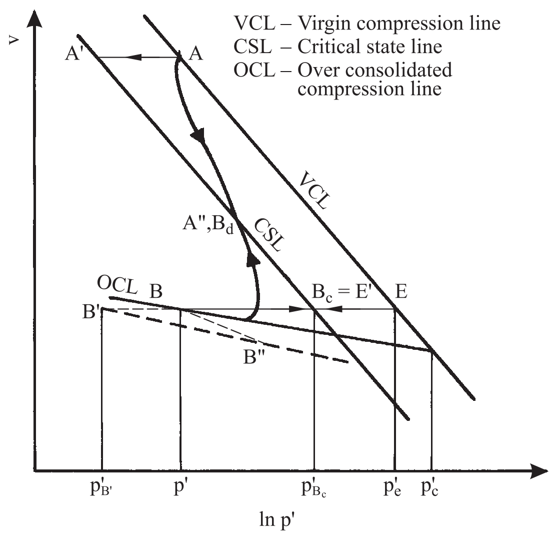

Figure 4.16. Effect of dismembering on test paths (Srinivasa Murthy et al. 1991).

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pressure caused by dismembering, $p' - p_B'$ is the reduction in pressure caused by frictional yielding along the Roscoe surface and their difference, $p'_{BC} - p'$, is the net change in pressure or net pore pressure observed in actual tests.

Similarly, in the drained test, the soil at B is expected to trace a path BB' similar to AA'. But in actual tests the soil dilates and moves towards the critical state $B_d = A''$ a state which the normally consolidated soil at A would reach. Again the dilation of the soil can be explained as an effect of the same phenomenon of dismembering of clusters formed while soil is loaded monotonically. In the revised Cam clay model, the original Cam clay model with the associated flow rule can itself be used to predict better stress strain behaviour of soils by incorporating the appropriate an additional internal stress parameter induced due to micro-strucutral changes, without causing external strains. The behaviour of overconsolidated soil can be described as yielding along successively expanding Roscoe surfaces with gradually increasing plastic properties, coupled with an increase in the value of the friction factor M. Although logical micromechanistic interpretations of soil behaviour could be advanced, further examination of the implementation of the revised Cam clay model, for systematic programming and numerical simulation to solve practical problems, is still lacking.

4.5.3 *Elasto-plastic model with variable moduli*

Re-examination of the dismembering effects of clusters during release of the locked-in energy enabled Santarajanna (1995) to propose an alternative approach, with the added advantage of implementation for systematic programming. Under isotropic loading the soil undergoes volumetric compression. In general, for saturated uncemented normally consolidated clays, within the engineering stress range, the $v - \ln p'$ relationship is found to be fairly linear, i.e. the compressibility coefficient $\lambda = dv/d(\ln p')$ is constant at all stress levels. In the case of isotropic unloading, it is generally found to be nonlinear, the path being steeper with further unloading. In the earlier models, when the unloading response was considered to be purely elastic, an average linear slope κ was being used. But subsequently it has been recognized (Vatsala et al. 1997) that an increasingly steep path is due to plastic volumetric dilation. Further, as discussed earlier, shear loading causes breakdown of clusters and releases the locked-in energy which, in turn, increases the effective stress in the case of the undrained test, or results in volumetric dilation in the case of the drained test. This has the same effect as an increased or higher recoverable volume change. Hence the soil may be thought of as having a higher elastic modulus represented by κ or possibly a reduced $\lambda - \kappa$ momentarily. As the degree of dismembering of clusters varies during shearing, the value of these modified moduli would also vary. Thus the soil may be treated as a material with variable values of deformation moduli during shearing.

To incorporate the variations for λ and κ the following method has been proposed by Vatsala et al. (1997). λ_i and κ_i refer to the values of λ_N and κ_o, being the

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soil constants of the original Cam clay model, at the current stress state. The following are the modified expressions for the elastic and plastic volumetric strains.

$$\delta\varepsilon_v^e = \frac{\kappa_i dp'}{vp'} \tag{4.41}$$

$$\delta\varepsilon_v^p = \frac{\lambda_i - \kappa_i}{Mp'v}\left[\left(M - \frac{q}{p'}\right)dp' + dq\right] \tag{4.42}$$

In the case of the undrained triaxial shear test (CIU test), the volume change is zero, i.e. from the above expressions for elastic and plastic volumetric strains

$$\frac{\kappa_i dp'}{vp'} + \frac{\lambda_i - \kappa_i}{Mp'v}\left[\left(M - \frac{q}{p'}\right)dp' + dq\right] = 0 \tag{4.43}$$

On simplification of the above equation, the incremental stress components *dp'* and *dq′* as suitable functions of the remaining terms can be expressed as:

$$\left[M - \frac{q}{p'}\right]dp' + dq = \frac{-\kappa_i dp'}{vp'} \cdot \frac{Mp'v}{\lambda_i - \kappa_i}$$

$$dq = \frac{-\kappa_i dp'M}{\lambda_i - \kappa_i} - \left[M - \frac{q}{p'}\right]dp'$$

$$\frac{dq}{dp'} = \frac{q}{p'} - M\left[\frac{\kappa_i}{\lambda_i - \kappa_i} + 1\right]$$

$$\frac{dq}{dp'} = \frac{q}{p'} - M\left[\frac{\lambda_i}{\lambda_i - \kappa_i}\right] \tag{4.44}$$

Equation (4.41) is the expression for the incremental stress ratio dq/dp' for a undrained path. The data of weald clay (Parry 1960) has been used to obtain the variation of λ_i and κ_i. Keeping λ_i constant, as that of isotropic consolidation value, λ_N, a suitable κ_i variation was found to be as per equation,

$$\kappa_i = \lambda_N\left[\frac{\kappa_o}{\lambda_N}\right]^{\left(\eta/M\right)} \tag{4.45}$$

where $\eta = q/p$. At $\eta = 0$, $\kappa_i = \lambda_N$ to begin with, and at $\eta = M$, $\kappa_i = \kappa_o$. This provides the variation of κ_i from a value of λ_N at $\eta = 0$, to κ_o at $\eta = M$. This means that the plastic strains are zero at $\eta = 0$, and gradually increase to become maximum plastic strains at the critical state. Considering the value of k_i constant as that of isotropic swelling at critical state p' i.e., k_o the variation of l_i has been found to be as per this equation (Shantharajanna 1995)

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$$\lambda_i = \kappa_0 \left[\frac{\lambda_N}{\kappa_o}\right]^{\left(\eta/M\right)} \tag{4.46}$$

Combining Equations (4.45) and (4.46) the general expression would be

$$\frac{\kappa_i}{\lambda_i} = \left[\frac{\kappa_o}{\lambda_N}\right]^{\left(\eta/M\right)} \tag{4.47}$$

In Equation (4.47) in the left hand term, if λ_i is regarded as a constant λ_N, the variation of κ_i, with η is according to Equation (4.45). Further by regarding κ_i, as a constant, κ_o, the variation of λ_i with η is according to Equation (4.46).

In the use of this model, only the parameters used in the original Cam clay model, λ, κ, M, N, G, are needed. These parameters for a given soil can be evaluated by an isotropic loading/unloading test and one shear test. All parameters except κ are well defined. The slope of the unloading curve up to the p' value at the critical state, which may correspond to $OCR = 2$ in place of average slope κ, is used. With these parameters and the law for variation of κ_i the stress strain path can be predicted. In the case of an undrained situation, for an applied increment of stress dp', dq is evaluated using Equation (4.44). For these increments the elastic and plastic strain increments are computed using Equations (4.41) and (4.42) and the current stresses and strains are updated. While using Equation (4.44) it is to be noted that κ_i is to be computed using the current stress states thus updated for the next increment of stresses. In the case of drained tests the increment of stresses applied are known beforehand and only the strain components can be computed using Equations (4.41) and (4.42). Figures 4.17 and 4.18 show typical plots of experimental and predicted stress-strain-pore pressure/volumetric strain paths of kaolin and Weald clays.

Overconsolidated state: In the case of overconsolidated states, the stress state being within the yield surface, the Cam clay model predicts purely elastic strains until the stress state reaches the yield surface. In the variable moduli approach the plastic strains within the yield surface too must be considered. For such states it is observed that the total volumetric strain modulus is different from λ_N and it varies with OCR. So the appropriate value of λ_o, being the slope of the recompression path in $v - \ln p'$ plot, for that OCR is to be used in place of λ_N in the variational law of κ_i, which then gives correct results. Similar to Equation (4.47), Equation (4.48) accounts for the variation of κ_i from an intitial value of λ_o to a final value of κ_o at the critical state.

$$\frac{\kappa_i}{\lambda_i} = \left[\frac{\kappa_o}{\lambda_o}\right]^{\left(\eta/M\right)} \tag{4.48}$$

Detailed verification by Shantharajanna (1995), has shown that the comparisons of stress-strain predictions with experimental results of overconsolidated clays are quite satisfactory.

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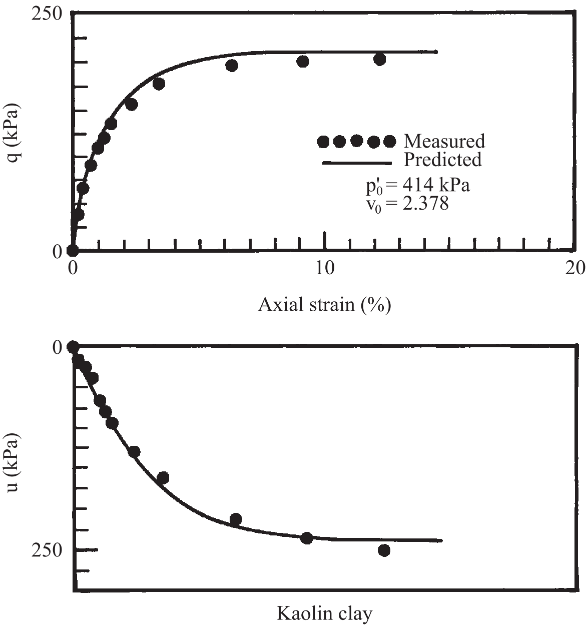

Figure 4.17. Experimental and predicted stress-strain-pore pressure plots of the CIU compression test on Kaolin clay (Vatsala et al. 1997).

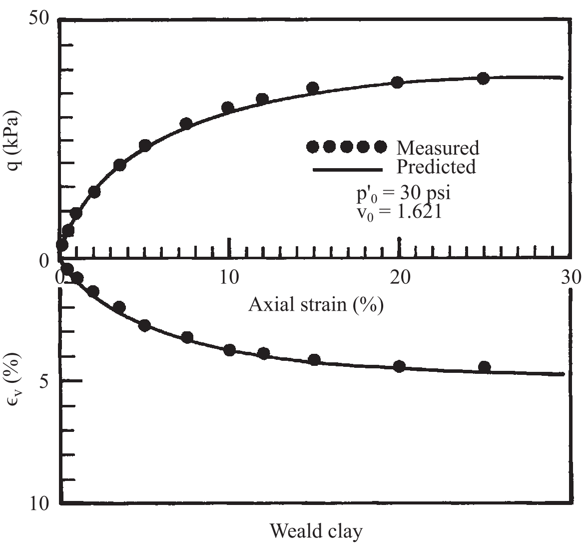

Figure 4.18. Experimental and predicted paths of stress-strain-volumetric strain plots of the CID compression test on Weald clay (Vatsala et al. 1997).

Obviously, it has not been possible to discuss in this section all the recent developments in constitutive modelling of saturated uncemented clays. It has also been not possible to address other aspects such as modelling of the behaviour of anisotropically consolidated states and that due to cyclic loading. The coverage provided indicates that due considerations of processes at the microlevel provides a basis for advancing suitable modifications to the original Cam clay model. It

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would be possible to retain the original Cam clay model yield surface with its isotropic hardening and associative flow rule.

4.6 STRESS-STATE PERMEABILITY RELATIONS

The significance and of need to assess the permeability characteristics of fine-grained soils is increasingly appreciated due to renewed interest in seeking acceptable solutions for environmental problems. The coefficient of permeability of coarse-grained soils has been related, with some sucess, to the characteristics of grain size distribution. Attempts to develop similar approaches have been less successful when extended to fine-grained soils due to interparticle forces arising due to clay-pore fluid interactions. The Kozeny-Carman relation (Equation 1.67), in which the fabric is accounted for by shape factors, does not serve the purpose of accounting fabric of clay. Further until recently it has not been possible to have stress-state permeability relations for prediction purposes. Although the mechanisms controlling permeability have drawn attention as early as 1971 (Mesri & Olson 1971) it has not been apparent how a generalized approach can be advanced for assessment purposes. It is known that the void ratio alone is not a sufficient parameter for assessment of the permeability of clays. Two soils with the same void ratio can have drastically different permeability values. It is equally possible that permeability of clays can be same with the void ratios being different. Thus it is obvious that the microstructure, or, more explicitly, the size and distribution of micropores, control the permeability and not just the total volume of voids.

4.6.1 *Normally consolidated*

With a void ratio corresponding to the water content at the liquid limit state as a reference parameter, it has been shown that the generalized soil state parameter reflects the clay fabric conditions rather that than void ratio (see Section 3.3.5). The advantage of using this generalized soil state parameter, e/e_L, in analysing the permeability data of clays and linking this with the consolidation stress merits examination. The permeablity data of soils compressed from their liquid limit state to different consolidation stress levels, determined by the variable head permeability test, have been examined (Nagaraj et al. 1993). In Figure 4.19 the data of $e - \log \sigma'_v$ and $e - \log k$ of soils considered for analysis are presented. It can be seen that at a particular consolidation stress level, say 100 kPa, (Fig. 4.19a) it can be seen that the void ratios equilibrated are different whereas the permeablity values at these void ratios are of the same order (Fig. 4.19d). This implies that the pore channels constituting the microfabric responsible for the same order of flow is of the same pattern, despite the void ratios being disticnctly different. This possibility is also depicted when the compression paths of the clays as well as the

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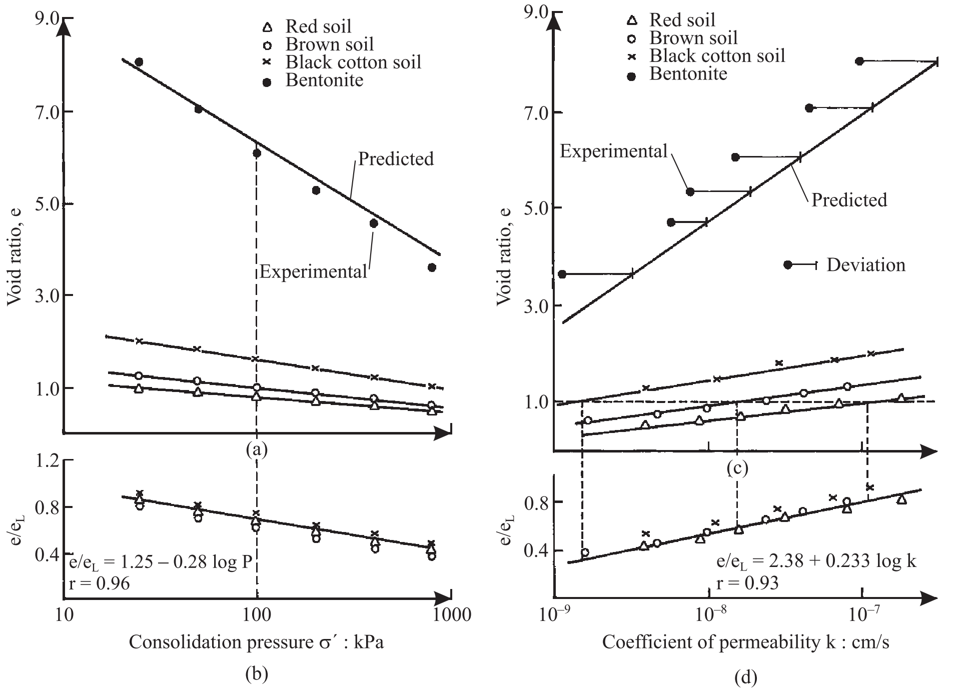

Figure 4.19. Generalized relations for stress-state permeablity relations (Nagaraj et al., 1993).

coresponding permeability paths are normalized by the respective void ratios at water contents corresponding to their liquid limit states. Linear regression analysis yield the following relations

$$\frac{e}{e_L} = a - b\log\sigma' \qquad\qquad (4.49)$$
$$\frac{e}{e_L} = c + d\log k$$

For the data analyzed

$a = 1.25$ $b = 0.280$ with $r = 0.96$

$c = 2.38$ $d = 0.233$ with $r = 0.93$

All the attributes for the water contents of clays at their liquid limit state to be taken as a reference parameter have already been discussed in Section 3.3.5. It has been shown in Table 3.3 that the permeability of clays at water contents corresponding to their liquid limit states is of the same order. Hence it is premised that the microfabric is of the same pattern. Another supporting evidence to this postulation is mercury porosimetry data of clays at their liquid limit. A reanalysis of

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limited pore size distribution data for four clays (liquid limit water content variation between 29 and 100%) from Griffiths & Joshi (1989) (see Fig. 3.12), in terms of intruded volume per unit volume, indicates that pore size distribution is of same pattern. It has been further reported that there exists a relation between the ratio of pore volume V_t to that at its liquid limit state V_{tL} and consolidation stress of the form

$$\frac{V_t}{V_{tL}} = 1.193 - 0.2060 \log \sigma \tag{4.50}$$

The implication of the above relation is that the intruded pore volume of the sample of a clay, under equilibrium at a specific consolidation stress, is proportional to its initial intruded pore volume at its liquid limit state. Hence V_t/V_{tL} is an indirect reflection of the micro-fabric of the clay. At an engineering level, e/e_L reflects the fabric of the clay. Depending upon the intrinsic potential of the clay, reflected by its e_L value, the equilibrium void ratio of the clay at a particular stress level would adjust itself such that the e/e_L ratio of different clays would be the same. This is to because pore channels available for flow per unit area would be of the same order and hence the same permeability.

Generally it is a misapprehension that montmorillonitic soils have a lower permeability than other soils of low plasticity. But the fact is that all clayey soils will have the same permeability (at corresponding states) ranging between 10^{-7} cm/sec at high water contents close to their liquid limit state and 10^{-9} cms /sec at water contents of about 0.5 w_L. For a soil of known liquid limit water content, the change in permeability with consolidation stress level can be assessed from the stress-state-permeablity relations discussed in this section. For a required permeability the consolidation stress level can also be computed. It is stressed that these assessments are valid only for slurry consolidated clays. The permeability of compacted soils will not follow the above relations since their micro-structures are different.

4.6.2 *Overconsolidated*

Most often soils in situ might have experienced different stress histories. In the case of overconsolidated clays devoid of cementation, to characterize the compressibility response it is necessary to assess the preconsolidation pressure. With the appropriate relation discussed earlier (see Equation 3.31) it is possible to trace the entire $e - \log \sigma'_v$ in the recompression, normally consolidated and rebound stages.

The permeability characteristics of samples with laboratory-induced stress histories have shown that it is solely dependent on the void ration irrespective of the past stress history effects (Fig. 4.20) (Nagaraj et al. 1994a). The preconsolidation stress level influences the level of permeability. For the entire data of normally consolidated and overconsolidated clays, the relation obtained is of the form

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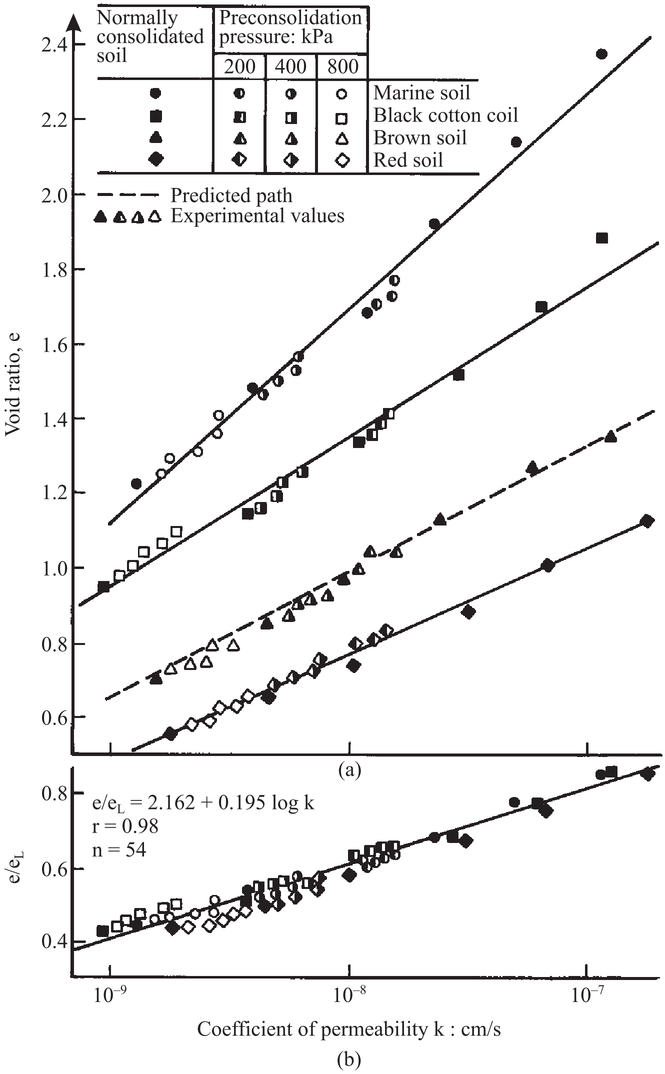

Figure 4.20. Void ratio and permeability relations of different clays (a) and their generalization (b) both in the normally consolidated and overconsolidated zones (Nagaraj et al. 1994a).

$$\frac{e}{e_L} = 2.162 + 0.195 \log k \tag{4.51}$$

with a correlation coefficient of 0.98 for the data examined, where *k* is in cm/sec.

It may be appreciated that, for a given soil, the permeability is mainly dependent on the void ratio and not on stress, while for different soils it is dependent on stress and on void ratio. For two soils under the same stress, the size of the large

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pores will be the same, and the volume of large pores in a given cross-sectional area will also not be very different, in spite of wide variation in void ratio. The differences in specific surface of the particles are responsible for the compatible void ratios at the same stress. Thus their permeability will be of the same order. With the increase in stress, there will be a reduction in the size and volume of large pores and hence a reduction in permeability. But for the same soil, at the same void ratio, under normally and overconsolidated states, as there will be no further change in size or volume of large pores during unloading and reloading, the permeability will be independent of pressure and will depend only on the void ratio.

4.7 CONCLUDING REMARKS

The discussions in this chapter deal mainly with soil behaviour governed by particulate considerations of the soil. Since the treatment pertains mainly to soils with an appreciable clay fraction, the considerations of interacting particulate systems discussed in Chapter 3 provide the basis for analysis and assessment of soil behaviour. The behaviour of in-situ soft clays which are insensitive or of very low sensitivity can be analyzed by the approaches discussed. Reclaimed soft clays, when subjected to precompression with or without sand or geodrains, can also be analysed for their compressibility, strength and permeability characteristics as the precompression progresses. In the analysis of soft sensitive clays the above analysis, based on the intrinsic potential of the clay, forms the basis in relation to which the deviations in the behaviour can be recognized and subsequently their engineering behaviour can be analyzed and assessed. Chapter 5 deals with soft cemented clays, their characteristics, analysis of their engineering behaviour, and assessment of the engineering properties wherever feasible.

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CHAPTER 5

Naturally cemented soft soils

5.1 INTRODUCTION

Although soils are particulate media, as elucidated earlier, stress, time and environment are the dominant factors in the formation of soft clay deposits (see Section 2.2). The environmental factor, in some cases, may be responsible for additional bonding to the fabric of clay through extraneous chemical substances. This in turn imparts specific characteristics to such deposits, such as sensitivity, low shear strength upon remoulding and high compressibility when loading exceeds cementation bond strength. Essentially it transforms the interacting particulate characteristics of the soft clay deposit to a non-particulate character. Hence the behaviour of such naturally cemented soft soils may have to be analyzed from combined non-particulate and particulate considerations depending upon the stress level in relation to the cementation bond strength.

There are many anomalies in the behaviour of naturally cemented soft clays when compared with the behaviour of uncemented soft clays. This may be because of the lack of a unique relation between the state and the conventional effective stress governed only by particulate considerations. In this chapter the contribution of the fabric of the soil and of the cementation component to the observed behaviour are examined in detail and appropriate methods are developed to consider their contribution at different stress levels. The approach is based on the finding that the load carrying capacity of a cemented soil is a superposition of two components, the resistance of the unbonded soil skeleton and the resistance of the cementation bonds. Further it is established that deformation is entirely due to changes in the stress on the unbonded skeleton, since the cementation bonds have a rigid response.

5.2 STRUCTURED SOFT CLAYS

Most of the soft and sensitive clays show the characteristics of natural cementation bonding between the particles. These soils include not only the brackish-marine Leda clays in Canada but also fresh water varved clays, the deep marine clays of Ariake Bay in Japan and others. The depositional and post-depositional

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aspects of geochemistry and their implications for engineering properties have been studied by several researchers (Skempton & Northey 1953, Kenney 1964, Rosenqvist 1953, Houston & Mitchell 1969, Ohtsubo et al. 1995, and others). In brief and in very general terms, the cementation bonds between the soil particles or their aggregates in saline environments are due to extraneous material of calcium carbonate from supersaturated solutions (Mitchell & Houston 1969) of aluminium and iron hydroxide precipitates (Soderman & Quigley 1965, Quigley 1968), organic compounds and amorphous manganese oxides (Loring & Nota 1968). Weathering, salt leaching, ion exchange and the formation or addition of dispersants have been reported as the post-depositional processes that influence the geotechnical behaviour of clays through the alteration of the geochemistry of clay sediments (Quigley 1980). For Ariake Bay clays with smectite as the main clay mineral, it has been shown that the remoulded shear strength decreased by salt leaching leading to the development of quick clays (Egashira & Ohtsubo 1982). In order to understand when and where the solid amorphous links are formed in the overall fabric so as to result in a structured soft clay, it is necessary to probe deeper into the possible fabric of the clay during formation of the clay deposit.

As a consequence of clay-pore fluid interactions, even in very low concentrations of solid particles, clay particles approach each other to form stable clusters. Particle orientation within the cluster, their size and stability, are influenced by the physico-chemical forces arising out of pore fluid chemistry and the nature of clay and non-clay minerals in the deposit. A possible micro-fabric of uncemented clays arising out of clay-pore fluid interactions has been discussed in Section 3.3.4. Considering fluid content at the liquid limit as a reference state to reflect the potential of the soil as well as that of the pore fluid, the schematic picture of a same clay micro-fabric for the same coarse fraction for two clays with different clay minerals is provided in Figure 3.14. This possibility is strengthened by the observation of the same order of permeability i.e., 10^{-9} m/sec. for different clays at water contents corresponding to their respective liquid limit states. A detailed micro-mechanistic explanation for the above postulation has been presented in Section 3.3.5. Permeability data for clay suspensions (Pane & Shiffman 1997 – Fig. 11) (Fig. 5.1a) reveal that for Speswhite kaolin clay at $e = 1.3$ with $e/e_L = 0.94$, $k = 2 \times 10^{-9}$ m/sec and for Phosphatic clay at $e = 4.5$, with $e/e_L = 1.01$, $k = 1 \times 10^{-9}$ m/sec confirms that e_L can be used as a reference parameter for critical examination of the permeability data of these clays at very high void ratios in the early stages of the formation of the deposit. It is very interesting to note that analysis of the extracted data of Pane and Shiffman (1997 – Fig.11) in relation to the intrinsic parameter e/e_L yields the following relation.

$$\log\left\{\frac{e}{e_L}\right\} = 2.27 + 0.256\log k \tag{5.1}$$

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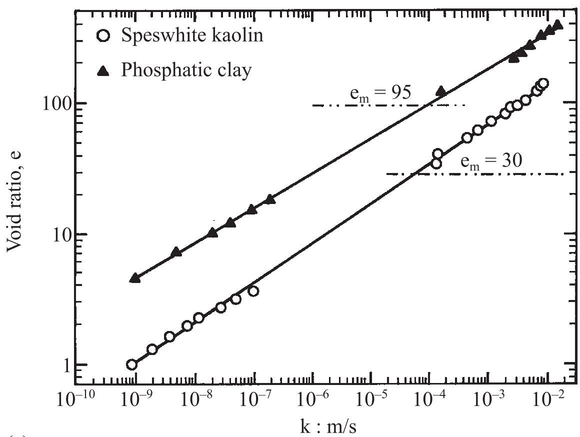
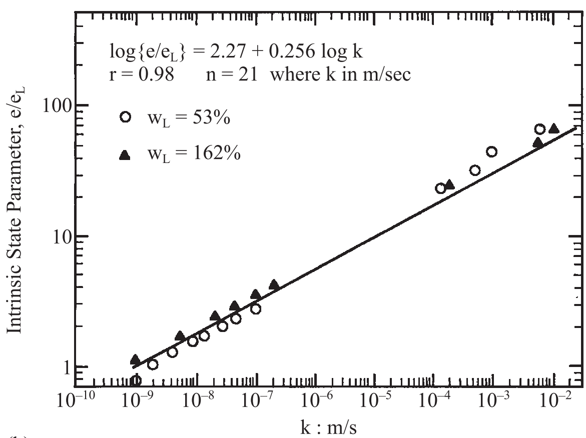
with a correlation coefficient of 0.98 where k values are reckoned in m/sec (Fig. 5.1b). Accordingly, at the specified permeability coefficient the void ratios of clays are proportional to their fluid holding capacity reflected in their liquid limit values. Since permeability characteristics indirectly reflect the micro-fabric of the clay, the above analysis suggests that the micro-fabric of the soft clay similar to that at their liquid limit state, is formed in the very early stages of clay deposit formation. Another observation that reinforces this inference in the uniqueness of the relationship between w/w_L and shear strength of extremely soft high water content Singapore marine clays as determined by the penetration method (Inoue et al. 1990, Tan et al. 1991). Although this micro-fabric continuously changes due to its own weight in the deposit, the characteristics are likely to be identical to that analyzed at the respective liquid limit state of the clays. This generalization needs



(a)



(b)

Figure 5.1. Log e versus log k plots (data from Pane & Shiffman 1997) and their normalized plot with their e_L values.

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further substantiation by generation and examination of permeability data from different clay suspensions in different physico-chemical environments.

Not withstanding the need for confirmation of the initial micro-fabric of clay suspensions, the response of the clay supensions as load increases can be examined. There would be continuous changes in the initial micro-fabric due to the effects of stress, time and cementation. As early as 1970 Pusch (1970), based on the data generated by the ultra-thin section technique and transmission electron microscopy, characterized the natural micro-structural pattern by a network of small aggregates connected by links of particles. This pattern is shown schematically in Figure 5.2.

According to the widely quoted fabric model derived from electron microscope studies by Collins & McGown (1974) there exists a possibility of clay pores being distributed among micro-pores surrounding assemblages of particles with submicroscopic pores within aggregations of particle groups (Fig. 5.3). This fabric fea-

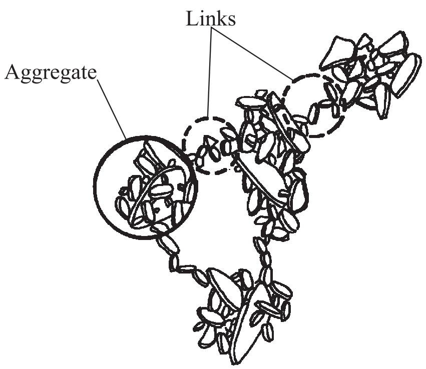

Figure 5.2. Schematic representation of the micro-structural network pattern of sensitive soft clays (Pusch 1970).

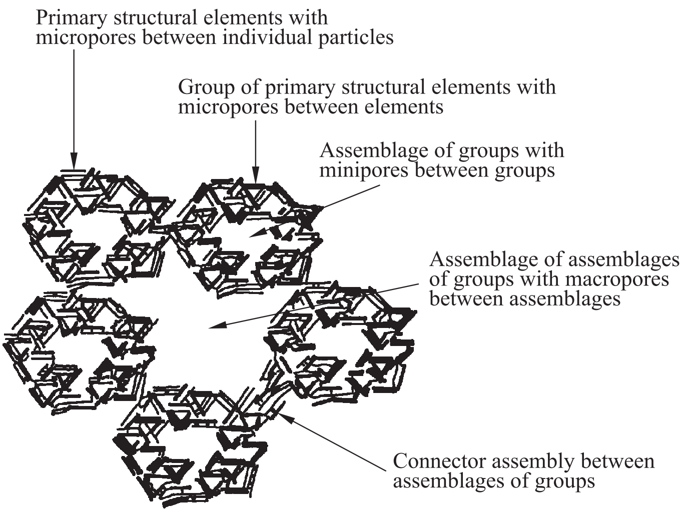

Figure 5.3. Fabric elements and pore types, interpreted from consideration of Collins & McGown (1974) by Griffiths & Joshi (1990).

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ture does not provide any clue to the deposition of cementation bonds in the fabric of natural clays. Perhaps the response of soft clay to stress changes in its undisturbed and remoulded states merits examination to delineate the microstructure of cemented soft clays.

Leroueil et al. (1979) refer to the cemented soils as 'structured soils' and consider the soil to be destructured once it is subjected to stresses much greater than its yield stress. That is, they attribute the difference in the behaviour of cemented and uncemented soils to the difference in the structure of the soil under these two conditions. It is necessary to probe what is meant by difference in structure and what type of changes can take place under different stress conditions.

It is well known that in a clay-water system the stress transfer takes place through an interacting fluid phase. The soil state realized is due to the equlibrium between long range forces and the externally applied stress. There is nothing in principle to bar the coexistence of long range forces and cementation bonds. The disposition of these features in the microstructure can only be delineated with detailed information about the geometrical aspects of microstructure. The pore size distribution and permeability data for cemented undisturbed soft Champlain sea clay and its remoulded state at the same void ratio was determined by Delage & Lefebvre (1984) and Lapierre et al. (1990) (Figs 5.4 and 5.5). It has been very interesting to note that no identifiable difference in pore size distribution and permeability was observed. This implied that the microfabric is of the same pattern. Based on the fabric for uncemented soft clay discussed in Section 3.3.5, the fabric

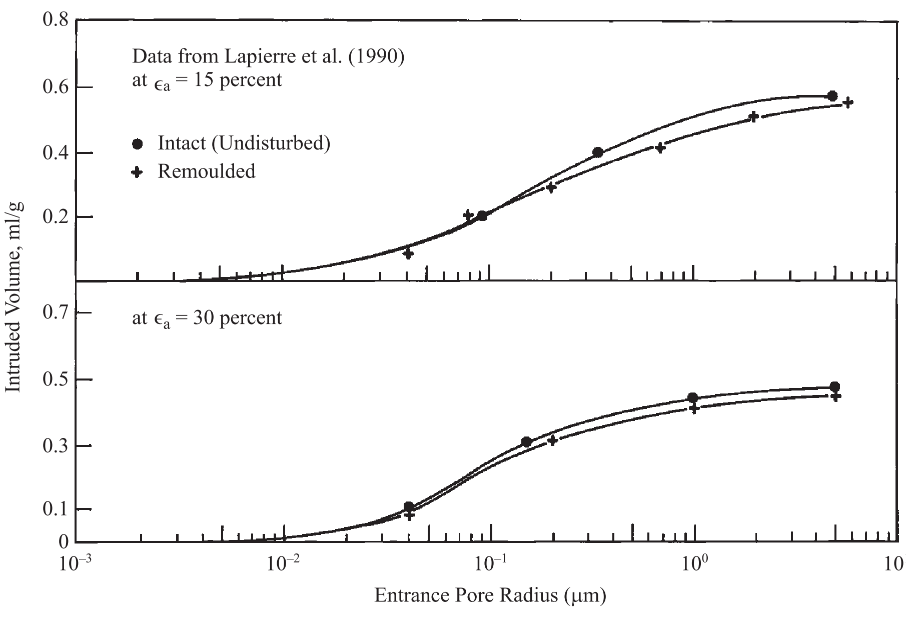

Figure 5.4. Pore-size distribution of intact and remoulded Champlain sea clay from (data Lapierre et al. 1990) (Nagaraj et al. 1998).

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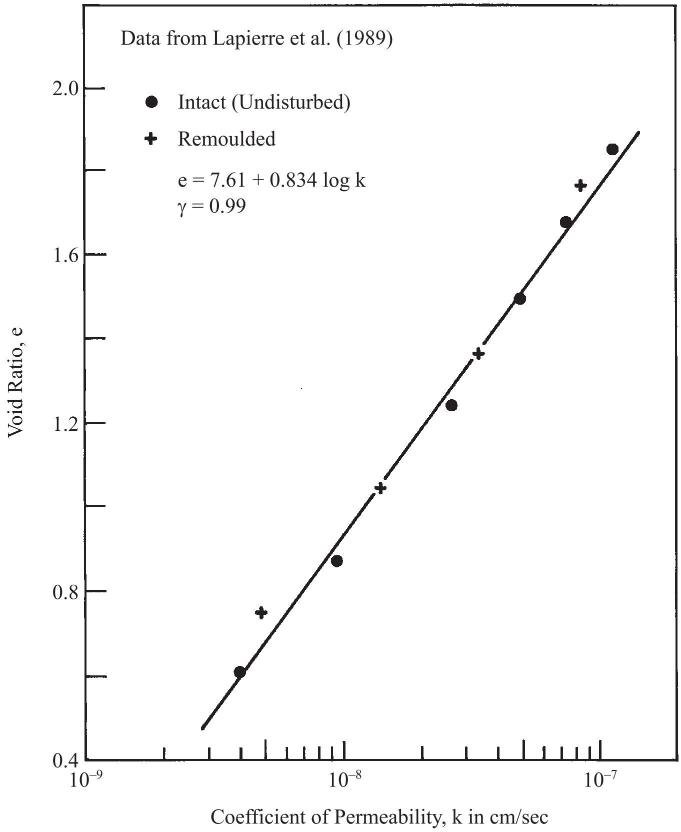

Figure 5.5. Plot of void ratio against coefficient of permeability for intact and remoulded Champlain sea clay (data of Lapierre et al. 1990) (Nagaraj et al. 1998).

pattern of naturally cemented soft clay is shown schematically in Figure 5.6. The implications of such a fabric for the compressibility and shear strength of naturally cemented soft clay will be discussed in later sections.

5.3 EFFECTIVE STRESS IN CEMENTED CLAYS

Apart from reverting back to the schematic compression path shown in Figure 2.4, the stress components responsible for different levels of compression are re-examined with the help of a schematic compression plot of both the undisturbed and the remoulded state of the same clay, shown in Figure 5.7. Compression is negligible up to a certain stress level, beyond which there is a sudden compression of relatively high magnitude indicated by a steep slope. This stress level, which marks the beginning of yielding, was designated as the apparent or quasi pre-consolidation pressure (Leonards 1972). Further, at any void ratio, σ_R is

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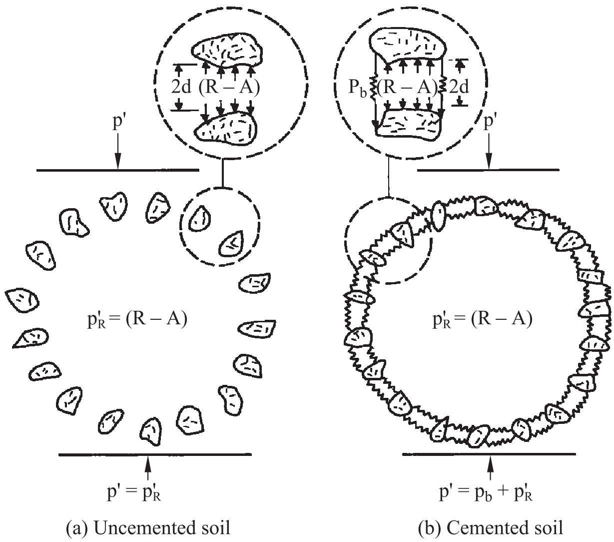

Figure 5.6. Possible microstructure of: a) An uncemented, and b) A cemented clay (Nagaraj et al. 1994, 1998).

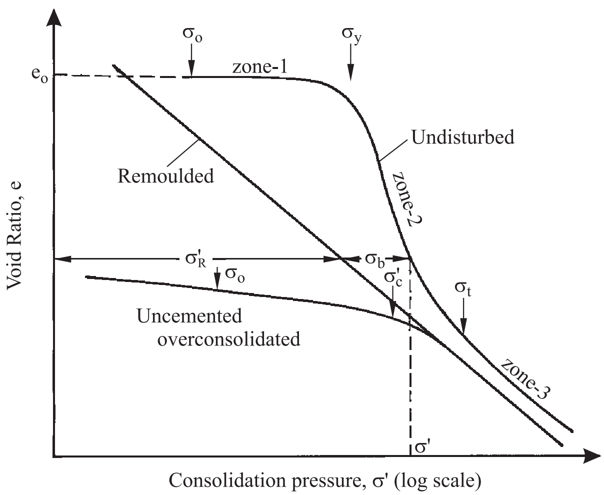

Figure 5.7. Typical compression path of soft cemented soil (Nagaraj et al. 1994).

the maximum stress that soil can carry without cementation bonds. The remaining stress component, σ_b, falling short of σ, is attributed to cementation bonds. Both interparticle forces and bond stress components are assumed to act simultaneously. The soil shows sudden compression beyond the yield stress. The general understanding is that the bonds break down beyond the yield stress and hence compression takes place.

For further elucidation let us consider the data for Louisville clay (Fig. 5.8) (Lapierre et al. 1990). The yield stress is about 180 kPa. Since the liquidity index of this clay is greater than 1.0 the clay derives additional resistance over that of the remoulded state due to natural cementation. The resistance of the remoulded

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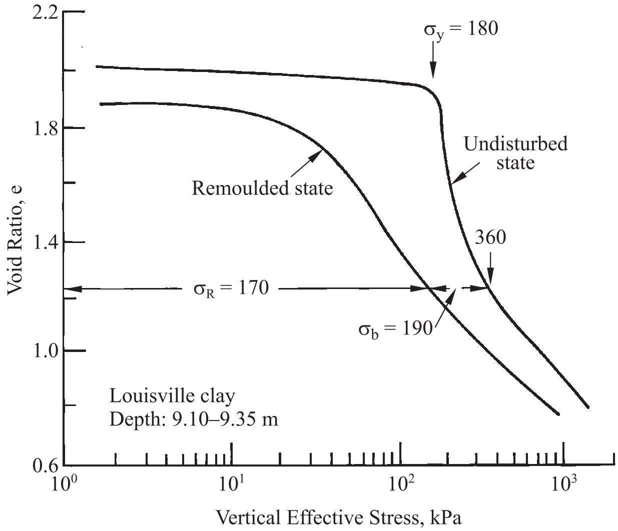

Figure 5.8. Compression paths of Louisville clay in undisturbed and remoulded states (Lapierre et al. 1990).

state at that high water content would be negligible for less than 5 kPa, a value at the liquid limit state of the clay. Now for a load increment of unity, i.e. 360 kPa, the remoulded component is about 170, which means that the cementation component is still 190 kPa. It can be further seen that this component is at no level less than 180 kPa. This indicates that during this pressure increment, though the load increment ratio on the soil was 1.0, it was actually much more literally from 5 to 170 kPa on the remoulded state. This is responsible for the high compressions and hence the reported high C_c values for the undisturbed soil. Similar has been the case with deformation features in the case of few more clays examined (Yong & Nagaraj 1977, Quigley & Thomson 1966) (Figs 5.9 and 5.10). This suggests that the deformation of the soil is possibly due to the stresses acting on the remoulded state. According to Terzaghi's effective stress principle, this stress component responsible for change in the state is the effective stress. The effective stress relation can be expressed as

$$\sigma' = \sigma_R = \sigma - \sigma_b - u \tag{5.2}$$

The stress-state relation would be the same as for an uncemented soil, i.e.

$$e = a - b \log \sigma_R = a - b \log(\sigma - \sigma_b - u) \tag{5.3}$$

The above observations and the ensuing inference would be valid for cemented soils provided the remoulded state does not redevelop some of the cementation bonds within the time required for laboratory testing, i.e. it should not exhibit some thixotropic characteristics but should represent a truly uncemented state. As

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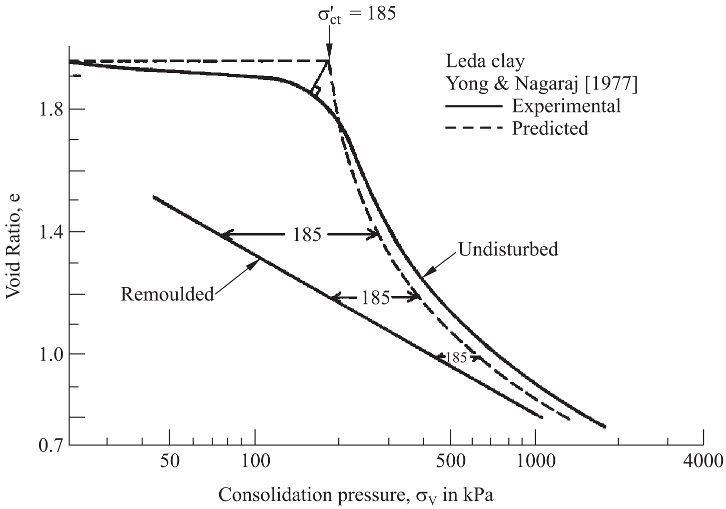

Figure 5.9. Prediction of σ_{ct} and most probable e versus log σ'_v using laboratory compression curve – Leda clay (data Yong & Nagaraj 1977).

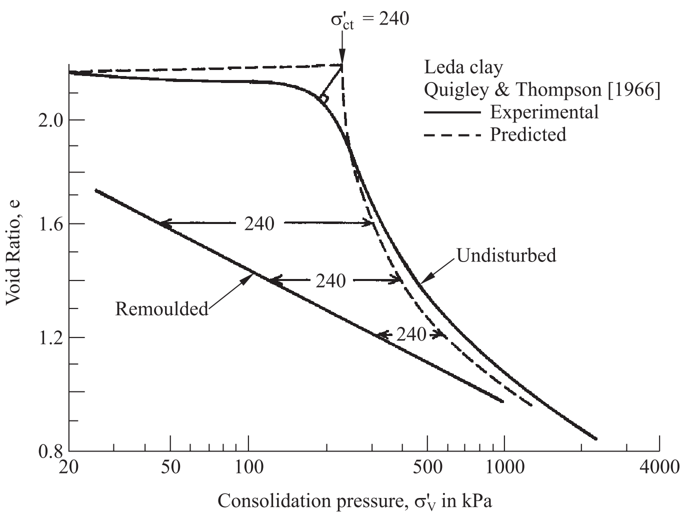

Figure 5.10. Prediction of σ_{ct} and most probable e versus log σ'_v using laboratory compression curve – Leda clay (data Quigley & Thomson 1966).

already cited (Delage & Lefebvre 1984, Lapierre et al. 1990) the pore size distribution and the permeability of the cemented undisturbed soil and of the remoulded soil at the same void ratio is identical, although the equilibrium stresses are different because of bond resistance. This is contrary to the earlier understanding that the micro-fabric of cemented soils is of the open fabric type. Since

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in remoulded soils the state is directly related to the effective stress, and for such a state to prevail in cemented soil, the same remoulded component of total stress can be assumed to be effective. The mobilization of uncemented skeleton resistance to shearing is also directly related to this effective stress σ_R, but with an additional shearing resistance q_b due to bonds.

5.4 ESTIMATION OF SAMPLING DISTURBANCE

Although great strides have been made in the in-situ testing and evaluation of engineering parameters for practice, laboratory testing cannot be dispensed with owing to specific advantages in exercising better control over boundary conditions and simulation of subsequent loading and other environmental conditions. Hence undisturbed sampling of soils is inevitable. If the natural water content of in-situ deposits is such that the liquidity index is greater than unity and the yield stress is only in the range of 25 to 50 kPa, such soft and sensitive deposits are prone to greater sample disturbance than stiff cemented soils. Since sampling disturbance is unavoidable, the laboratory testing is most often on a partially disturbed sample, the specific requirements for assessment of sample quality and evaluation are:

1. Quantification of the sample disturbance from the laboratory test on undisturbed samples suffered unavoidable mechanical disturbance.
2. The ease with which corrections can be made to other test results obtained by tests on such partially disturbed samples.

5.4.1 *State of-the-art*

Extensive investigations have so far been reported in the literature on different methods of sample quality evaluation based on tests on undisturbed samples (Hvorslev 1949, Ladd & Lambe 1963, Nelson et al. 1971, Okumara 1971, Anderson & Kolstad 1979, Nagaraj et al. 1990a, Onitsuka et al. 1995, Onitsuka & Hong 1995a, Shogaki & Kaneko 1994, Shogaki 1996, and others). Although sample disturbance, in the absence of macro-fabric features, implies de-structural effects, only very few attempts can be traced to analyses of disturbance to a reference state of the soil not prone to any mechanical disturbance. The liquid limit state is due to an internal stress field, which is only a function of the water-holding capacity of the clay, and the micro-fabric would remain the same as long as the water content is unaltered and hence is not prone to any mechanical disturbance. As discussed in Section 2.3.2, the conventional laboratory compression paths of clays from a water content corresponding to their liquid limit state is devoid of any stress history, secondary time effects and natural cementation and hence forms a reference path.

By undisturbed sampling at various depths, apart from undisturbed samples for testing, the information available would be that of the overburden pressure. The information from the sample would be the natural water content and the bulk den-

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sity. This does not directly provide any information about the structured state of the clay as a result of ageing and cementation. Hence the practical way appears to be to examine the dominance of any of the three effects, stress, time and cementation, responsible for the present state of the clay deposit. How far the relation between the state of clay in its reconstituted state and the consolidation stress would provide any help in determining the dominance of any one of the three factors merits examination.

A detailed examination of the basic framework, already developed and discussed in Chapter 3, would help as a reference stress-state path. The schematic e/e_L versus log σ' compression paths of clays and the average rebound and recompression paths at different consolidation stress levels, have already been shown in Figure 3.28, with the combined functional relation given by Equation (3.31). What is depicted by these compression paths is the reference template. What has been considered is only the physico-chemical parameters of the clay and non clay solid constituents and their synergy with the pore medium. The specific advantage of this generalization is that the slopes of compression paths prior to and after the transitional stress level would aid in the analysis of soft clay behaviour as influenced by any other dominant factor, such as time and cementation subduing the effects of mechanical stress history. Soft clay deposits data from one hundred locations all over the world (Table 3.1) (Nagaraj et al. 1997a), collected and collated for subsequent analysis, plotted on the w/w_L plot of Figure 3.30, reveal that practically all clays plot above the intrinsic compression line AB. Particularly with clays whose w/w_L is greater than unity, the microstructure is in a metastable state, since the stable compatible state would be on the intrinsic compression line. Hence soft clays of such an initial state are prone to mechanical disturbance during sampling and handling.

5.4.2 *A quantitative approach*

Shogaki & Kaneko (1994) and Shogaki (1996), for their detailed study on sampling disturbance assessment, developed a simple laboratory device to impart various degrees of mechanical disturbance. Undisturbed samples after imparting different levels of disturbance were subsequently tested for their strength and compressibility characteristics. Figure 5.11a and b shows the compression paths obtained. As the authors have rightly stated, the degree of rigidity (non-particulate response reflected in yield stress level) decreases as the degree of disturbance increases. In these plots the predicted compression paths of the clay from its liquid limit water content of the clays (ICL paths) are shown. These paths do not exhibit any yield stress and hence forms a reference compression path. It can be seen that the degree of rigidity reflected by the compression path up to the yield stress, and the value of the yield stress itself, reduces as the sample disturbance increases and moves inwards and towards the completely remoulded path (Nagaraj et al. 1998b, 1999). This suggests the possibility of

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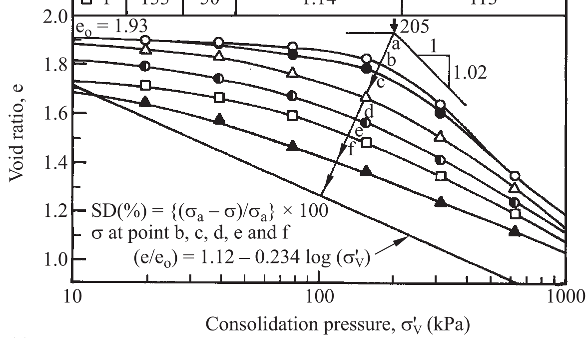
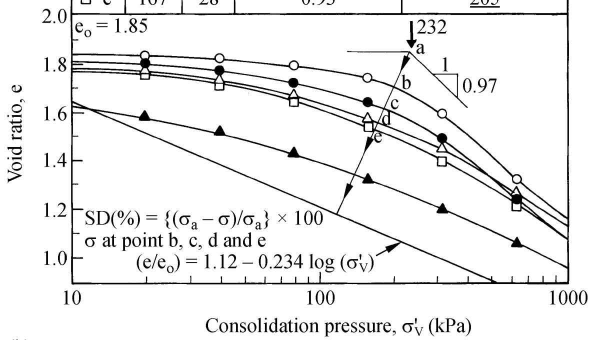
	σ_y (kPa)	SD (%)	$C_{cm} = \frac{C_c}{1 - SD/100}$	$q_{um} = \frac{q_u}{1 - SD/100}$ (kPa)
○ b	180	12	1.05	163
● c	172	16	0.98	116
△ d	157	30	0.87	105
◐ e	142	35	1.06	117
□ f	133	50	1.14	113



(a)

	σ_y (kPa)	SD (%)	$C_{cm} = \frac{C_c}{1 - SD/100}$	$q_{um} = \frac{q_u}{1 - SD/100}$ (kPa)
○ b	200	14	1.04	225
● c	182	22	1.05	238
△ d	173	25	0.86	220
□ e	167	28	0.93	205



(b)

Figure 5.11. a and b) Analysis of data of Shogaki & Kaneko (1994) for sample disturbance (Nagaraj et al. 1998a).

building up a scale to assess the degree of sample disturbance. This is similar to the approach suggested for field samples (Nagaraj et al. 1990a). But recently the re-examination has been on a more detailed scale and on samples progressively mechanically disturbed intentionally.

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The most probable yield stress (at point 'a') and sample disturbance (SD) have been determined in relation to the predicted remoulded path, free from any disturbance. The lines of reduction in yield stress are shown by arrows along the path perpendicular to the intrinsic compression line, as in the figures. The degrees of disturbance, as per the relation indicated (Nagaraj et al. 1990a) for each of the paths, are shown in the figures, and also the modified values of the compression index and unconfined compression strength values for the test data on partially disturbed samples are indicated. It can be seen that, except for the values underlined, for most of the cases there is convergence towards the most probable values corresponding to the undisturbed (least disturbance) case.

5.4.3 *Suggested approach*

It is believed that an examination of extensive compressibility and strength data of undisturbed samples of soft and sensitive clays along these lines would provide additional credibility to the above approach. Hence, based on the basic considerations explained earlier and the analysis carried out, the following tentative step-by-step procedure has been suggested for analysis and assessment of sample disturbance and accounting for its effects on other engineering properties.

The data required are: in-situ water content and overburden pressure, index properties of the clay, compression path of the undisturbed clay (partially disturbed sample) and vane strength data.

Step 1 – Assess the location of the w/w_L versus overburden pressure point on the ICL plot in Figure 3.30. If the point plots above the ICL line, sampling disturbance effects can be dominant.

Step 2 – Plot the $e - \log\sigma$ path of the sample tested and on this superimpose the laboratory compression path of remoulded clay, from its liquid limit state of the predicted compression path of the clay as calculated from the equation:

$$\left\{\frac{e}{e_0}\right\} \text{ or } \left\{\frac{e}{e_L}\right\} = 1.23 - 0.28\log\sigma'$$

(e/e_0) is used if e_0 is greater than e_L, otherwise e/e_L is used. Draw a horizontal line from the in-situ void ratio as well as a line perpendicular to the predicted compression path. This defines the direction and magnitude and the path of reduction of yield stress due to sample disturbance. From the relative location where this line cuts the compression path of the partially disturbed sample, the degree of sample disturbance can be assessed as illustrated in Figure 5.11. A crosscheck obtained graphically to confirm the yield stress is possible if the yield stress can be assessed independently. This will be examined in the next section.

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Step 3 – Using the value of the sample disturbance, the compression index and undrained strength value corresponding to the undisturbed condition can be assessed as indicated in the inset table in Figure 5.11a and b.

5.5 MOST PROBABLE COMPRESSION PATH

Within the stress range of engineering interest, from the considerations detailed in Section 5.3, it is indicative that at any void ratio e, the equilibrium stress σ' on the undisturbed soil, at least to a minimum extent, is in excess of the stress on the same soil, completely remoulded and monotonically compressed to the same void ratio e, by the magnitude σ_b such that

$$\sigma' = \sigma_R' + \sigma_b \tag{5.4}$$

The total resistance to compression can be considered to be the sum of two components, the cementation resistance and the unbonded skeleton resistance. At every level of void ratio it should be possible to scale the magnitude of these two components to trace the most likely compression path. Hence assessment of the most probable compression path of a naturally cemented clay involves two steps, assessment of the yield stress and the compression path of the same clay in its uncemented state. This can be regarded as giving an upper bound value of compression index.

5.5.1 *Modification of laboratory compression curve*

As has been discussed in the previous section the locus of σ_y for different degrees of sample disturbance is a straight line perpendicular to each of the e – log σ' curves at their points of maximum curvature (Fig. 5.12). In the limiting case this line is likely to be perpendicular to the remoulded path, which corresponds to a hundred percent disturbance. To fix up the magnitude of yield stress point falling along the straight line identified by the above method, the in-situ load settlement observations made by Pelletier et al. (1979) and Folks & Crooks (1985) are helpful. It has been observed that the settlements are negligible up to the yield stress beyond which a marked increase in settlements took place. With the above two findings, the most probable value of yield stress can be predicted from the laboratory compression curve of the field sample at the point of intersection of a horizontal line from the point σ'_0, e_0 and the normal to the e versus log σ' curve at the point of maximum curvature (see Fig. 5.12). This procedure is illustrated using both the field and laboratory compression curves for four soils (Fig. 5.13).

5.5.2 *Using field vane strength*

Alternatively, in the absence of a laboratory compression curve, the field yield stress can be determined using field strength data. This is a useful method since it

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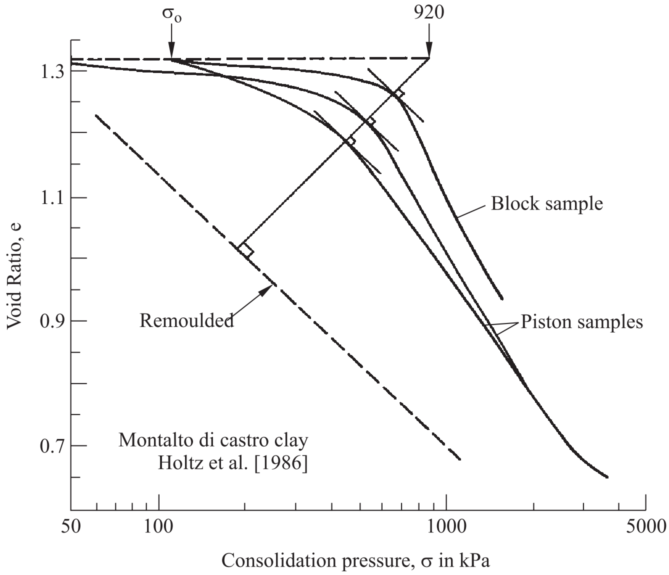

Figure 5.12. Effect of sample disturbance on the path of e versus log σ curves of Montalto Castro clay (data after Holtz et al. 1986) (Nagaraj et al.1990a).

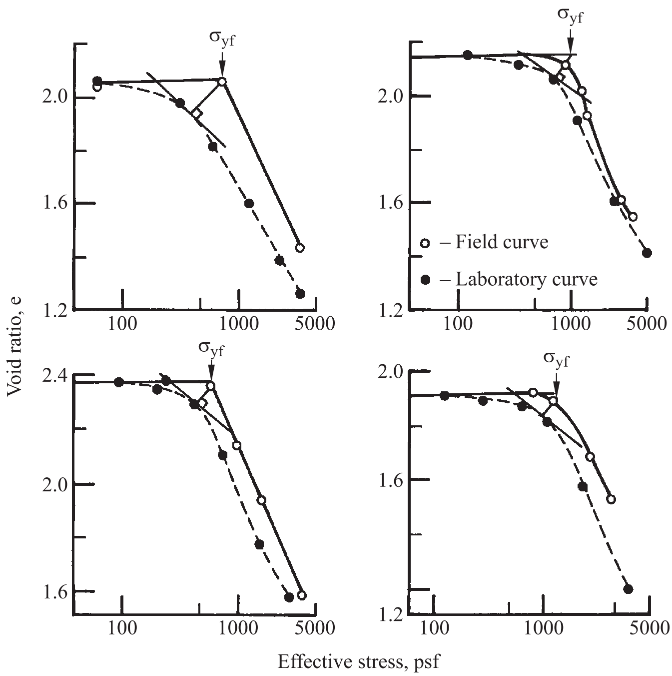

Figure 5.13. Comparison of predicted σ'_{ct} with field values from the data of Pelletier et al. (1979) (Nagaraj et al. 1990a, 1994).

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is a routine practice to determine the vane strength of soil during site investigations and hence this data would always be available. Since both the yield stress σ_y (σ_b) and the undrained strength q_b are reflections of the same bond strength, it is logical to relate these two parameters. In fact, due to the non-particulate nature of cementation bonding, the bond resistance in compression bears a relation to the vane shearing resistance (Fig. 5.14) which in turn is independent of the intrinsic particulate characteristics of the clay, reflected in terms of index properties. Detailed analysis of the published data for many sensitive Canadian soils has resulted in a functional relationship of the form

$$\sigma_{Y,\text{field}} = 3.78 S_u + 7 \tag{5.5}$$

with a high degree of correlation of 0.98 (Nagaraj et al. 1990a), S_u being in kPa.

Once the bond resistance is known, the most probable compression path can be assessed. For this the compression path of the clay in its reconstituted state is needed. This path can be determined by a laboratory test. It is also possible to estimate its path from the generalized relation in Equation (3.29). If the remoulded water content is higher than the liquid limit water content of the clay then, instead of the void ratio corresponding to liquid limit water content, the void ratio corresponding to the natural water content, a reflection of the potential, has been suggested as applicable. In tracing the probable path, the equilibrium stress at any void ratio beyond the yield stress is obtained by adding σ_b to the stress on the uncemented σ'_R at that void ratio. Figures 5.15 to 5.18 show the assessed and experimental paths. In general, it has

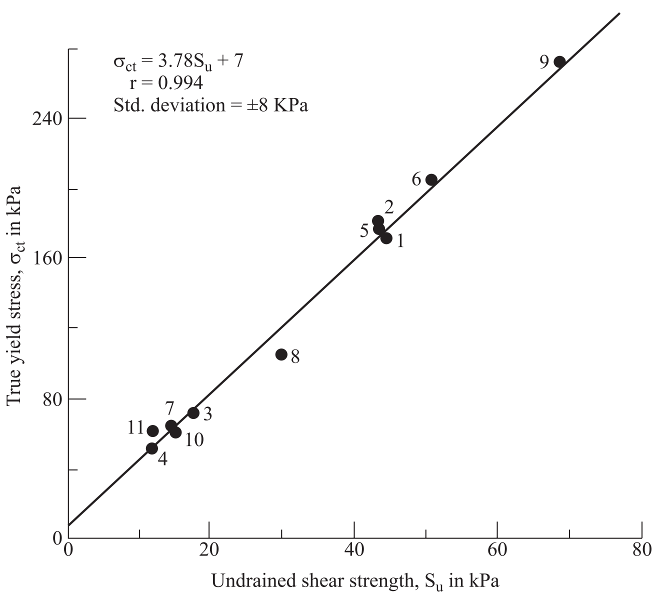

Figure 5.14. Relation between the yield stress and vane strength of different clays (Nagaraj et al. 1990a).

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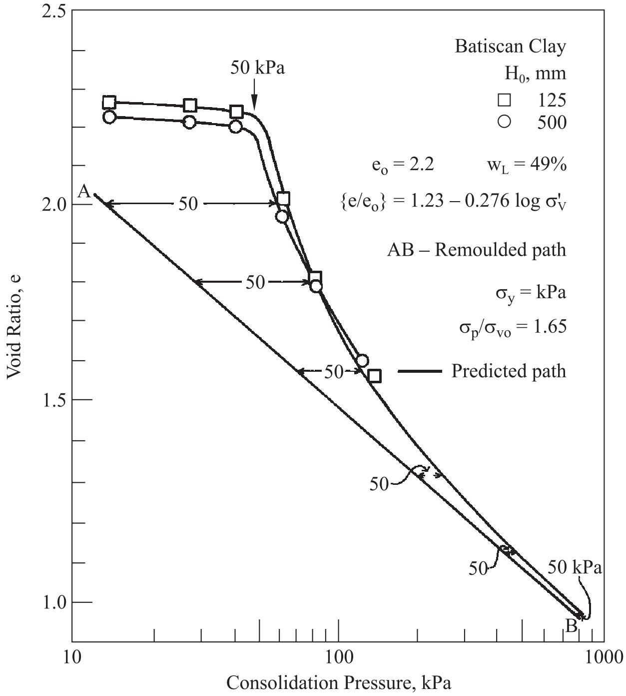

Figure 5.15. Prediction of σ_{ct} and most probable *e* versus log σ'_v using laboratory compression curve – Batican clay (data Mesri et al. 1995).

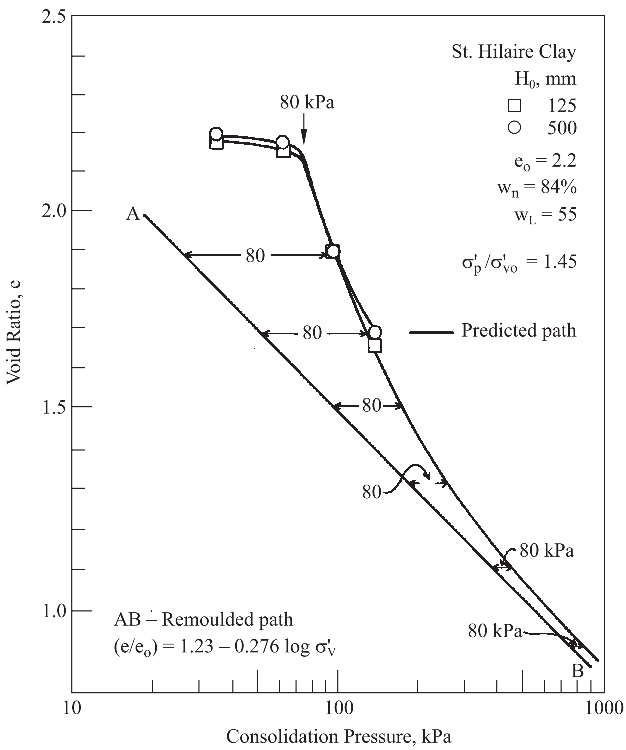

Figure 5.16. Prediction of σ_{ct} and most probable e versus log σ'_v using laboratory compression curve – St. Hilaire clay (data Mesri et al. 1995).

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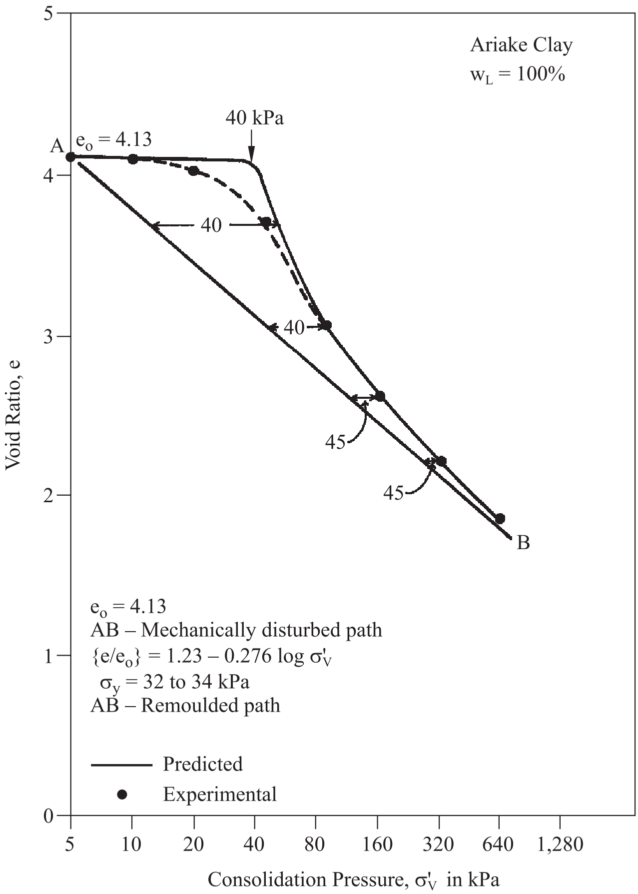

Figure 5.17. Prediction of σ_{ct} and most probable e versus log σ'_v using laboratory compression curve – Ariake clay (data Koumoto & Otsuka 1992).

been observed that as the in-situ water content is higher than the water content at the liquid limit, the assessed path is in very good agreement with the experimental path. As the in-situ water content is closer to the liquid limit water content, or lower, then the experimental path would be to the right of the paths predicted. The resistance to compression would be higher than that assessed.

5.6 ASSESSMENT OF COMPRESSIBILITY

It is quite evident that the compressibility characteristics of naturally cemented soft clays cannot be quantified by a single value for the entire stress range. As indicated in Figure 2.4 the laudable approach is to characterize the compression index zone-wise, and hence three values applicable to the three zones have to be

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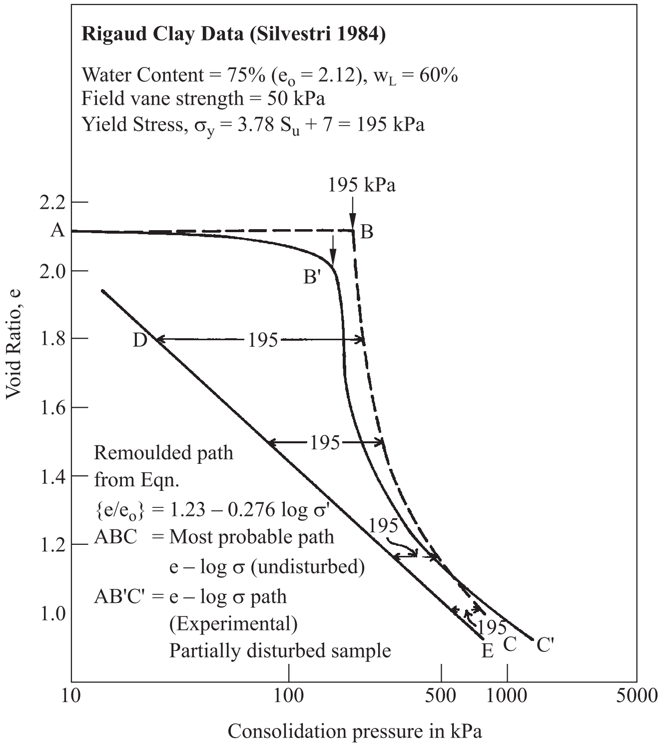

Figure 5.18. Prediction of σ_{ct} and most probable e versus log σ'_v using laboratory compression curve – Rigaud clay (data Silvestri 1984).

evaluated. For zone one, up to the yield stress the compression path would be close to horizontal. The compression index C_{c1}, although close to zero cannot be regarded as such, since the soils undergo a slight degree of compression due to sampling disturbance. Hence, for all practical purposes, on the conservative side its value can be taken to be that computed for the recompression path as influenced by the stress history effects.

$$C_{c1} = 0.05e_L \tag{5.6}$$

The compression index for zone two cannot be obtained from the index properties. The magnitude depends upon the yield stress level, sample disturbance and by how far the in-situ void ratio of the undisturbed clay is higher than the void ratio at the liquid limit. For this it is preferable to trace the compression path from the in-situ water content of the clay superimposed by experimental or predicted path of the remoulded clay from the liquid limit water content of that clay. The procedure outlined in Section 5.5 would enable us to trace most probable compression path. The compression path between the yield stress level and zone three can

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be used to compute the compression index. The compression index can be computed by the relation

$$C_{c2} = \left\{ -\frac{de}{d\log \sigma'_v} \right\} \tag{5.7}$$

where *de* is the difference in the void ratio between that corresponding to the yield stress and the second transitional stress level. σ'_v is the stress increment between the yield stress and that at the point where the second transition takes place, as depicted in Figure 2.4. If this transition is not apparent then the stress difference corresponds to one log cycle considering the appropriate void ratio difference.

The compression index in the third zone, if applicable, can be determined from the relation

$$C_{c3} = 0.28 e_L \tag{5.8}$$

5.7 ANALYSIS OF SHEAR STRENGTH CHARACTERISTICS

As early as 1966 Andresen & Sollie (1966) at the Norwegian Geotechnical Institute developed an inspection vane borer for field determination of the undrained shear strength of clay. This vane borer, with the range of strength it can cover being as high as 200 kPa, was in use in trenches and excavations at shallow depths. Subsequently field vanes capable of use in the high strength range were developed for use at greater depths (Leussink & Wenz 1967). In this method the vane, together with its casing and external drill rod, must first be pressed into the soil up to a depth 50 cm above the measuring point. Then the vane, with the internal drill rod which has freedom to move and rotate inside the external casing, has to be pushed down to the required depth. With the help of a gearbox, the internal rod and with it the vane can be rotated so as to cause failure of the sample in about three minutes, with the torque required being measured. This reference time to failure is very relevant, since the time effects on vane shear strength and consequently the sensitivity assessment are dependent on rate effects. According to the analysis of the shear strength results from laboratory investigations by Sridharan & Madhav (1964), the strength was found to be as high as 60% when the rate of rotation was increased from 1.2 to 30° per minute. Sensitivity was also reduced as the rate of rotation decreased. To determine the remoulded strength of the soft clay, the test was repeated after the first failure. By using a series of vanes of different height/diameter-ratios (Aas 1967), an approximate assessment of the ratio between the shear strength acting along the horizontal and vertical failure surfaces could be made.

A useful but approximate correlation between the undrained remoulded shear strength and the liquidity index was proposed by Wroth & Wood (1978) and Wroth (1979). It was based on the observation that for remoulded clays the re-

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moulded undrained strength at the liquid limit is about 1.7 kPa while at the plastic limit it is approximately 170 kPa. Assuming a linear relationship between water content and the logarithm of undrained shear strength, the following relationship was proposed:

$$s_u^* \text{ (kPa)} \approx 170 \exp(-4.6\, I_L) \tag{5.9}$$

where s_u^* is the undrained shear strength of remoulded soil and I_L is the liquidity index. Being aware of the fact that ageing and natural cementation not being considered, apart from macro-features such as fissures, Wroth (1979) considers this relationship to be useful only as a framework against which observed shear strengths from undisturbed samples can be judged for their consistency and reliability. Not necessarily considering the linear relationship as the basis, as was the case for the data collated by Leroueil et al. (1983) (see Fig. 5.19) the following relation has been suggested:

$$c_u\text{(kPa)} = \left\{\frac{1}{(I_L - 0.21)^2}\right\} \tag{5.10}$$

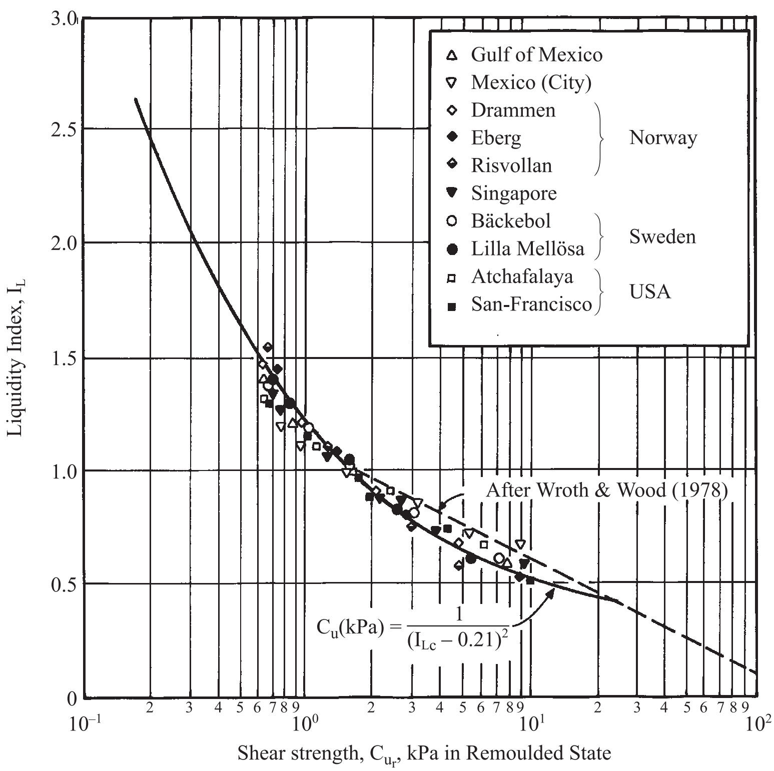

Figure 5.19. Relation between liquidity index and remoulded shearing resistance (Leroueil et al. 1983).

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In Japan, the undrained shear strength s_u of cohesive soils is commonly obtained from the unconfined compression (UC) test. Half the compressive strength, $q_u/2$, from the UC test is undrained shear strength. Unless high quality samples are taken by appropriate sampling tools, the values may not be realistic. Another factor affecting the values determined is the uncertainty of the residual effective stress in the exposed state. In order to assess the relative efficacy of the field vane shear FVS and UC test methods, Tanaka (1994) analyzed data on seven Japanese marine clays obtained by him. The suggestion by Tanaka (1994) is that when reviewing the results from the field vane shear and unconfined compression tests, it must be borne in mind that the UC test tends to underestimate the in-situ shear strength, due to sampling disturbance. On the other hand the FVS overestimates, due to friction between the vane rod and soil or between the outer and inner rods. At present most countries except Japan seldom use the UC test for evaluating the undrained shear strength of soft clays.

Bjerrum (1973) proposed the correction factor, μ, for the shear strength obtained from the FVS test, based on analysis of many failure cases. In his proposal, as the effect of anisotropy and strain rate are considered, μ is determined from the plasticity index, I_p (Fig. 5.20). Analysis of strength properties of Japanese marine clays (Hanzawa & Tanaka 1992) reveals independence from I_p, whereas most soft clay data for deriving Bjerrum's correction factor were from Scandinavia. This suggests that Bjerrum's correction factor merits re-examination. This is also required in the framework of earlier discussions as to how strength arising out of cementation has a bearing on index properties.

It is obvious that, by the methods discussed above, the emphasis has been on finding the undrained shear strength values without any due consideration of deformation levels and consequent pore water pressure mobilization. Although shear strength and compressibility are frequently considered as quite separate aspects in conventional approaches, in fact both are measures of the resistance of soils to de-

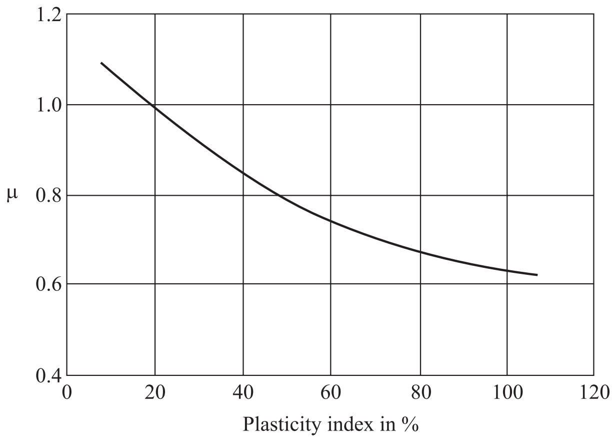

Figure 5.20. Bjerrum's correction factor (Bjerrum 1973).

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formation, but with different boundary conditions. It is basically a reflection of the resistance offered by the micro-fabric of the soil.

At the outset, the shearing behaviour of sensitive clays appears to be similar to that of overconsolidated soils. The stress-strain response exhibits a steep rise in strength or a high tangent modulus at low strains, and a strain softening behaviour beyond the peak strength mobilization. The base for comparison is the stress-strain behaviour of the same clay, sheared from its monotonically loaded state to the same initial conditions with respect to its void ratio state. In fact, the shear strength of soft clay on shearing gradually approaches that followed by a monotonically loaded sample at large strains. Hence most of the studies of the shear strength of soft clays has been based on the premise that their shear behaviour can be characterized by considering them as being subjected to certain degree of overconsolidation. This would be in order if a method of identifying which factor, stress, time or environment (cementation), was dominant during the formation of soft clay deposits is not available.

Since the earlier analysis of the strength of naturally cemented clays was examined only within the framework of particulate mechanics, no other implications were recognized. As such the two methods that were developed in the 1970s as a representative method for obtaining in-situ undrained strength of soft deposits are as follows.

1. Bjerrum's method (Bjerrum 1973): In this method, the in-situ undrained strength of a clay is obtained from the K_0 consolidated undrained strength test, in which the specimen, prepared from an undisturbed sample, is re-consolidated at the same stresses it carried in the field. Tests with longer consolidation times, spanning even up to 10 days and beyond, are recommended in order to achieve complete restoration of the original structure of the specimen in the field.
2. SHANSEP, the Stress History and Normalized Soil Engineering Properties approach (Ladd & Foott 1974): In this method, the measured undrained shear strength is normalized with respect to the effective overburden pressure and related to the overconsolidation ratio OCR. A high quality, undisturbed sample is consolidated, under conditions of no lateral strain (K_0 condition), to a vertical effective stress in excess of the in-situ effective overburden pressure (1.5-2 times σ'_{vc}), and then to point B, to establish the normally consolidated behaviour as illustrated in Figure 5.21, then unloaded to the appropriate vertical stress to give a value of OCR required (A to D SHANSEP – CK_oU test). The sample is then loaded to failure to obtain the undrained strength, S_u.

In Figure 5.22 the typical normalized strength plots are indicated. It has been categorically stated that many clays have been found to exhibit normalized behaviour, although cemented clays or highly structured clays are exceptions. It is not apparent how it was ensured that the clays considered for examination exhibited only particulate behaviour. The following is the relationship between OCR and the ratio of the normalized shear strength.

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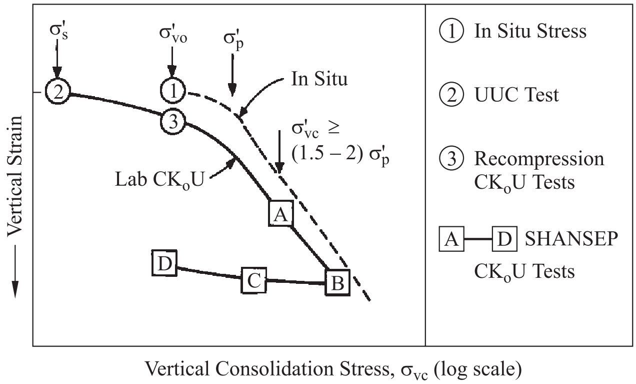
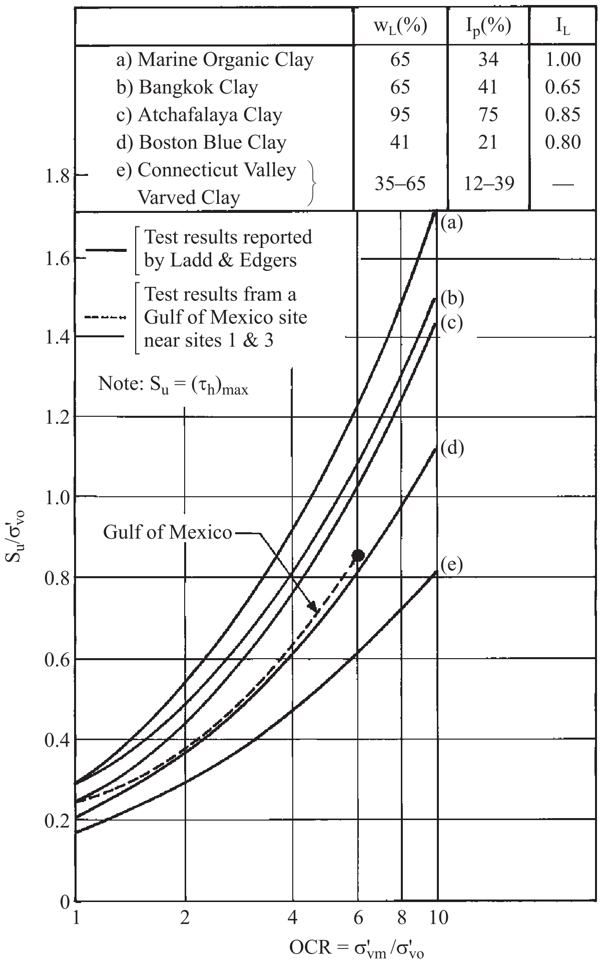


Figure 5.21. Different steps in the SHANSEP tecnique (Ladd & Foot 1974).



	w_L(%)	I_p(%)	I_L
a) Marine Organic Clay	65	34	1.00
b) Bangkok Clay	65	41	0.65
c) Atchafalaya Clay	95	75	0.85
d) Boston Blue Clay	41	21	0.80
e) Connecticut Valley Varved Clay	35–65	12–39	—

Figure 5.22. Normalized undrained strength data (Quiros et al. 1983).

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$$\left\{\frac{\left(s_u/\sigma'_{vo}\right)_{oc}}{\left(s_u/\sigma'_{vo}\right)_{nc}}\right\} = (OCR)^m \tag{5.11}$$

where m is about 0.8. The parameter m can be related to the parameters of critical state soil concepts (Worth 1984).

The difference in the approaches of the above two methods is that an attempt is made in Bjerrum's method to consider the ageing effect in evaluating the shear strength of a clay in the field. In Ladd's method this does not receive any attention. SHANSEP assumes mechanically overconsolidated behaviour to represent all preconsolidation mechanisms, and hence involves obvious errors with highly structured, sensitive clays and with naturally cemented deposits (Ladd 1986).

SHANSEP may tend to underestimate strengths in deposits having significant ageing effects with natural cementation. Perhaps the strength behaviour characterized by $m = 1$ in Equation (5.11) leads to a constant s_u/σ'_p. The value of this constant, as proposed by different investigators, although not the same, varies over a narrow range. Mesri (1975) proposed that the mobilized shear strength s_u can be related to σ'_p the preconsolidation pressure, independent of I_p as

$$s_u = 0.22\sigma'_p \tag{5.12}$$

This relation in Equation (5.12) suggests that the increment of strength ratio is 0.22. In Japan, on the other hand, an increment strength ratio of 0.3 is usually adopted in practice (Tanaka 1994). In Equation (5.5), ignoring the constant value of 7 kPa, the increment strength ratio is 0.26. It is indicative that σ'_p, can be considered as the yield stress in the K_0 compression test, σ'_y, which may have no direct relation with the plasticity characteristics of soft clay nor with the overburden pressure.

5.7.1 *Strength Parameters*

Conventionally, for a clay the shear resistance τ at the micro-fabric level is dependent on the normal effective stress σ' and the coefficient of friction ϕ, by the relation

$$\tau = \sigma' \tan\phi \tag{5.13}$$

The possibility of the existence of additional source of shear resistance, termed as 'bonds' was recognized by Kenney (1968) as early as 1968. The above shear strength Equation (5.13) was modified into the form

$$\tau = b + \sigma' \tan\phi \tag{5.14}$$

The fact that the bonds are brittle and shearing resistance is dependent on the strain level was also appreciated. The present level of clarity regarding the role of cemen-

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tation bonds did not then exist, nor it was clear that these bonds provided additional shearing resistance apart from the resistance arising due to the soil fabric.

Assessment of the strength parameters of naturally cemented clays is often required for design needs in solving stability problems. It is necessary to examine whether the approach of deriving parameters from conventional triaxial test data by the modified Mohr Coulumb diagram (Fig. 1.19b), as discussed in Section 1.6.2, can be adopted. Hanzawa & Kishida (1981) and Hanzawa et al. (1990) have examined strength development due to ageing. Accordingly the undrained strength has been considered to be a combination of two strength components, i.e. the strength developed by primary consolidation under effective stress acting on the clay as external stress such as σ_{vo}, and the strength developed by the ageing effect, σ_{ua}. This second strength component has been designated as internal effective stress, $\sigma'_{\text{int.}}$ arising due to ageing with and without cementation. For example, secondary compression is a phenomenon where the void ratio decreases under constant external stress, while, due to natural cementation, the bonding structure is developed under a constant void ratio. It is difficult to evaluate the contribution from each of the two phenomena separately. Hence it has been suggested that the total effect of ageing on the micro-structure may have to be assessed from strength tests. For aged clays the relationship between undrained strength and yield stress is schematically shown in Figure 5.23. In this figure the strength contribution due to ageing and natural cementation is superimposed on that available due to the changes in the void ratio as applied consolidation stresses change. The ratio between the in-situ undrained strength S_{uf}, and σ'_{vy} is assumed to be constant up to the yield stress σ_{vy}, irrespective of the type of ageing effects. This constancy has been seen up to the yield stress due to resistance offered by cementation to compression. Beyond the yield stress level, strength mobilization is assumed to be in accordance with that due to the normally consolidated condition. The typical

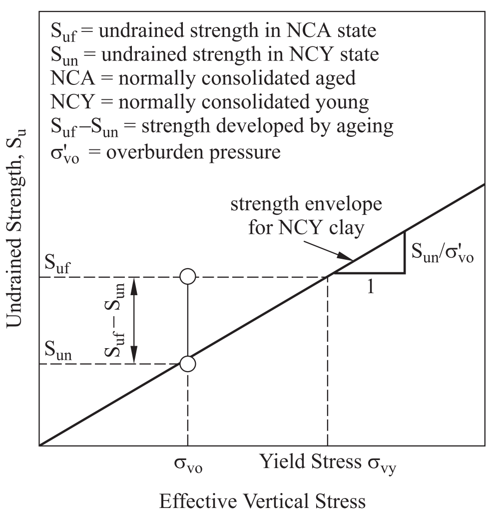

Figure 5.23. Conceptualized path for undrained strength versus vertical stress.

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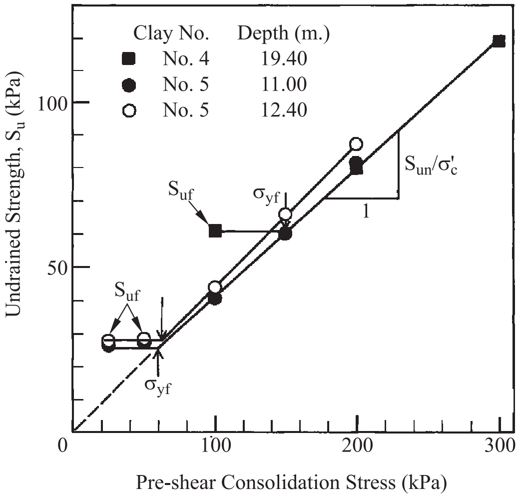

Figure 5.24. Typical strength data to validate the schematic representation in Figure 5.23 (Onitsuka & Hong 1995).

relationship for Ariake clay between undrained strength and pre-shear consolidation stress obtained from the triaxial compression test is shown in Figure 5.24. It can be seen that when the consolidation stress is less than the yield stress, σ_{vy}, the undrained strength is independent of the consolidation stress. This result is also consistent with that of the undrained strengths which are identical for all confining pressures less than the yield stress, from the extensive test data reported by Bozozuk on the Gloucester fill (Nagaraj et al. 1990a). Onitsuka et al. (1995) reiterate that the undrained strength depends upon natural cementation bonding up to respective yield stress levels, beyond which the undrained strength mobilization is in accordance with the slope of the line representing the relationship between undrained strength and consolidation stress, as in the normally consolidated condition.

5.8 CONSTITUTIVE MODELLING OF CEMENTED SOFT CLAYS

During the last four decades, a wide spectrum of constitutive models for cohesive soils (see Sections 1.7 and 4.5) has been developed, with varied degrees of success. As a first step in the analysis of the stress-strain response of naturally cemented soft clays, it is necessary to examine the specific cases which warrant modifying the constitutive models already available.

5.8.1 *Analysis of stress-strain response*

A closer examination of the stress-strain-pore pressure response of clays subjected to only stress dependent overconsolidation, and that of naturally cemented sensitive clays, reveals that both responses are distinctly different (Fig. 3.3). In cemented soft clays strain softening is observed invariably in undrained shearing,

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but in drained shearing softening may occur only at very low confining pressures. In stress-dependent overconsolidated clays, a peak in the stress-strain response is observed only in drained shearing, the more so at higher overconsolidation ratios. Strain softening in undrained shearing on naturally cemented clays occurs under all confining pressures beyond the level of yield stress. Further, softening in overconsolidated clays is associated with volumetric dilation or negative pore water pressure, whereas in naturally cemented soils the response is that of continued volumetric compression or positive pore water pressure. These are clear indications that the volume change in drained shear, and pore water pressure mobilization in undrained shear, of stress-dependent overconsolidated clays are markedly different from those of naturally cemented clays.

Further, the responses of naturally cemented clays during shear are amplified by consideration of the compression paths of constant p' drained tests and undrained shear tests at different confining pressures, as shown schematically in Figure 5.25. Obviously, at large shear strains, when the bonds are disrupted the soil tends to reach the same failure states on the critical state line as that which the same soil in its uncemented states would reach under similar conditions on the critical state line. That is the same void ratio in the undrained test, and the same confining pressure and stress path in the drained test corresponding to the equivalent uncemented state of the same clay. It can be seen why the volumetric compression increases for tests with increasing confining pressures up to the yield stress, and then decreases (3-3″ > 2-2″ > 1-1″ and 3-3″ > 4-4″ > 5-5″). It is also clear that the magnitude of the pore pressure or volume change for naturally ce-

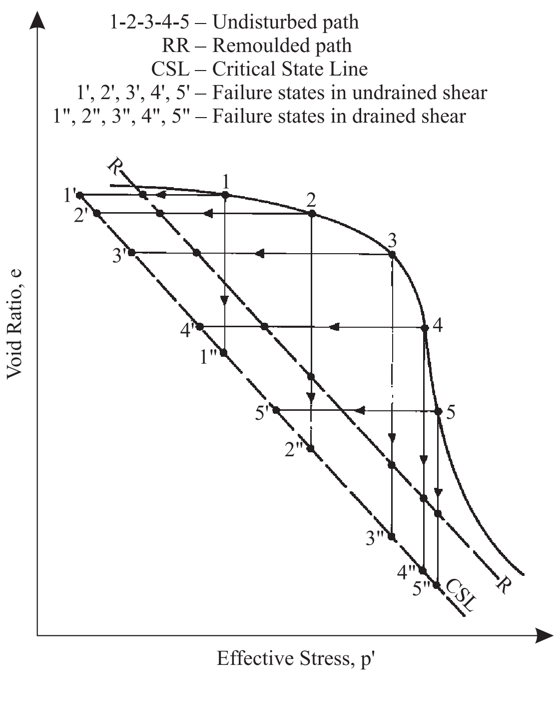

Figure 5.25. Drained and undrained shear test paths of naturally cemented clays in the v – p plane (Vatsala 1991, Nagaraj 1995).

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mented states is far greater than that for the corresponding uncemented state of the same clay (Vatsala 1991, Nagaraj 1995).

It appears that all the above features observed during shearing of naturally cemented clays can be explained by extending the hypothesis advanced in Section 5.5 to analyzing compressibility behaviour. It is assumed that during shear of a naturally cemented clay the yielding or deformation is entirely due to changes in the component of stresses excluding the bond resistance acting on the unbonded micro-fabric. In other words, the actual shearing resistance of the soil at any strain level is the sum of the unbonded clay fabric resistance and cementation bond resistance, i.e. at a strain level i,

$$q_i = q_{Ri} + q_{bi} \tag{5.15}$$

This mode of superposition was suggested by Conlon (1966) and then by Feda (1982). If the above proposition of the existence of two components and their superposition is very likely, the mechanism of the shear behaviour of naturally cemented clays can be delineated.

The unbonded component q_R increases hyperbolically with strain, similarly to an uncemented normally consolidated clay, drained or undrained as the case may be. The cementation component q_b mobilizes to reach its peak value at very low strains, usually being in the range of 1 to 1.5%. With continued shearing, both in drained and undrained conditions, the bonds are gradually disrupted, resulting in the reduction of this component. A breakdown of the bonds during shearing may mean a reduction in the number of bonds per unit volume, contrary to the possibility of an increase in bonds per unit volume during compression under the K_0 condition. Simultaneously, during undrained shearing the unbonded component q_R increases with strain, the rate of increase being markedly lower with strains these in the range of 4 to 6%. In drained shearing, the unbonded component increases markedly even beyond strain levels, say of the order of 15%. Thus the total strength, q, which is the sum of these two components, may or may not show a peak and strain softening, depending on whether or not the rate of increase of the unbonded component is less than the rate of decrease of the cementation component at any strain level. This explains why generally a peak in strength mobilization is markedly observed in undrained shearing, followed by strain softening compared to that observed in drained shearing. Ultimately, at large strain levels, when the bonds are completely broken, q_b tends to reduce to zero and hence the strength of the soil might reach the magnitude mobilized in the case of the remoulded soil.

To obtain the stress-strain response, it is necessary to estimate the two components independently at any strain level and consider their sum.

The shear stress paths of a cemented soil in critical state line are shown schematically in Figure 5.26. For any given state $C(e, p')$ of the clay, there is a corresponding uncemented state $E(e, p'_R)$. Mobilization of the uncemented shearing resistance component due to micro-fabric, with shear strain, will be ε_s for this initial

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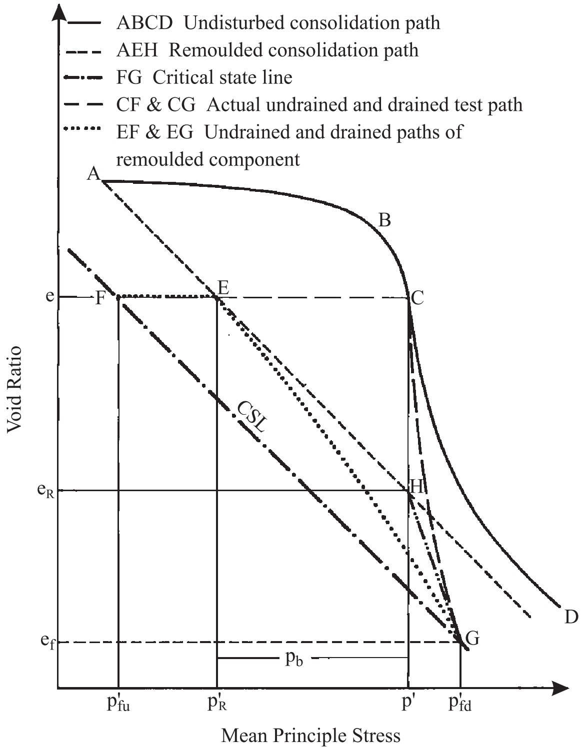

Figure 5.26. Shear stress paths of naturally cemented soil on the $e - p'$ plane (Nagaraj et al. 1991a, 1994).

state. Simultaneously the bond strength q_b mobilizes to its peak value, $q_{b\max}$, at low strain levels and decreases thereafter due to disruption of cementation bonds (Fig. 5.27). With the increase in the deviatoric cementation component q_b, there would also be an associated increase in the mean principal stress, of the order of $q_b/3$. As the cementation bonds are progressively stressed to their capacity, the bonds are incapable of resisting additional mean stress $q_b/3$. Added to this the stress-carrying capacity of these bonds itself reduces from its initial value p_0 ($= p_{b\max}$) to zero at large strains, due to disruption of bonds. Hence the resultant stress components, $q_b/3$ and ($p_{b0} - p_{b\varepsilon}$), will have to be transferred either to the remoulded state of the clay, if drainage is permitted, or to the pore water pressure in an undrained situation. This is in effect equivalent to an external loading of the same magnitude on to the uncemented soil state. At large strains, since the bonding reduces practically to very low levels, the whole of p_{b0} would have either to be transferred to pore pressure or to be borne by the soil fabric. The clay would ultimately reach the same critical state $F(e, p'_{fu})$ in undrained shearing as the state of an uncemented clay would reach upon shearing (see Fig. 5.26).

We now examine why higher magnitudes of pore pressures are mobilized during undrained shearing compared to those mobilized in the case of the remoulded state of the same clay at the same void ratio. Figure 5.28 shows the undrained stress path of a naturally cemented soft clay, along with the stress path on the unbonded clay

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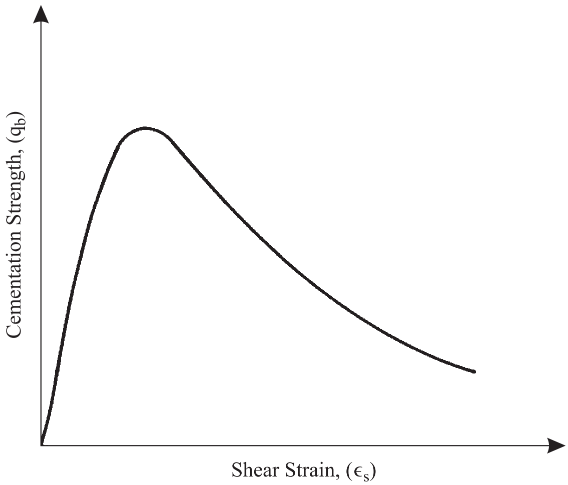

Figure 5.27. Degradation of cementation bond strength with shear strain. (Nagaraj et al. 1994).

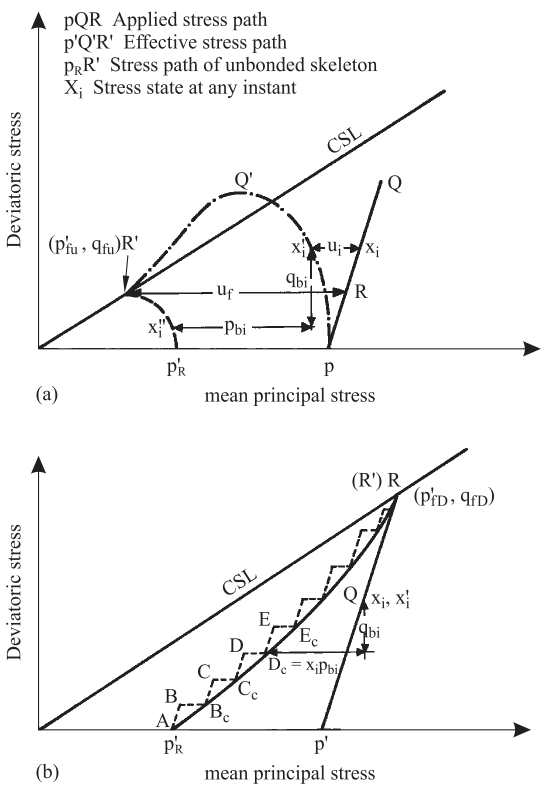

Figure 5.28. a) Undrained, and b) Drained shear stress paths of naturally cemented clay on $q - p'$ plane (Nagaraj et al. 1994).

fabric, explicitly depicting the two components of resistance. At any instant, for X_I the applied total stress X'_i is the effective stress and X''_i is the corresponding stress state on the unbonded skeleton. The pore water pressure component ($= p' - p'_{fu}$) in the cemented state is greater than that in the uncemented state of the same clay ($p'_R -$

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p'_{fu}), both reckoned at the same void void ratio (see Fig. 5.26). The total resistance $X'_i(q, p')_i$ is the sum of the unbonded fabric resistance $X''_i(q_R, p'_R)_i$ and the bond resistance (q_b, p_b) (see Fig. 5.28).

Similarly, in shearing under drained conditions, as the cementation bonds are disrupted, the clay would reach the failure state (e_f, p'_{fd}) as the remoulded clay would reach (e_R, p') under the same stress path (see Fig. 5.26). During the drained condition, at each stage of shearing the stress components $q/3$ and u_b ($= p_{b0} - p_{b\varepsilon}$) would be transferred to the uncemented micro-fabric as an effective stress component. Hence the initial uncemented state at (e, p'_R) is subjected to a loading path different from the actual applied path (see Fig. 5.28). That is, the initial clay fabric minus cementation bonds at (e, p'_R) is subjected to greater change in stresses (p'_R to p'_{fd}) than is apparent (p' to p'_{fd}), and hence to greater volumetric compression.

The above discussions permit us to advance the following postulations. In undrained shearing, at any given state, the total shearing resistance is made up of two components,

1. the shearing resistance of the unbonded soil skeleton, q_R, for the initial state (e, p'_R), and
2. the bond resistance, q_b.

$$q = q_R + q_b \tag{5.16}$$

The three components making up the total pore pressure u, are

1. due to an uncemented fabric component of pore pressure u_R, corresponding to q_R,
2. a component $q_b/3$ equal to the increase in the mean principal stress due to an increase in deviatoric stress, q_b, and
3. a component u_b equal to the decrease in p_b caused by the disruption of bonds.

$$u = u_R + u_b + q_b/3 \tag{5.17}$$

In drained shearing, the total shear strength, q, at any instant is the sum of the cementation component q_b, and the unbonded skeleton component q_R, with its deviator stress path to include the stresses $q_b/3$ due to the dissipation of u_b arising from stress transfer due to the disruption of cementation bonds throughout the shearing process. The cementation component has a rigid response. Hence, there is only one component of volumetric strain ε_{vR}, which is due to the changes in the stresses acting on the uncemented clay fabric. Figure 5.29 schematically depicts the cemented, uncemented and net components of shear strength with strain, along with the components and net mobilized pore water pressures in undrained shearing and the volumetric strain component in drained shearing.

5.8.2 *Constitutive modelling*

Although the above analysis provides a detailed discussion of the stress-strain-pore pressure or volumetric changes when naturally cemented soft clays are sheared, it does not provide comprehensive formulations, which could be directly implemented

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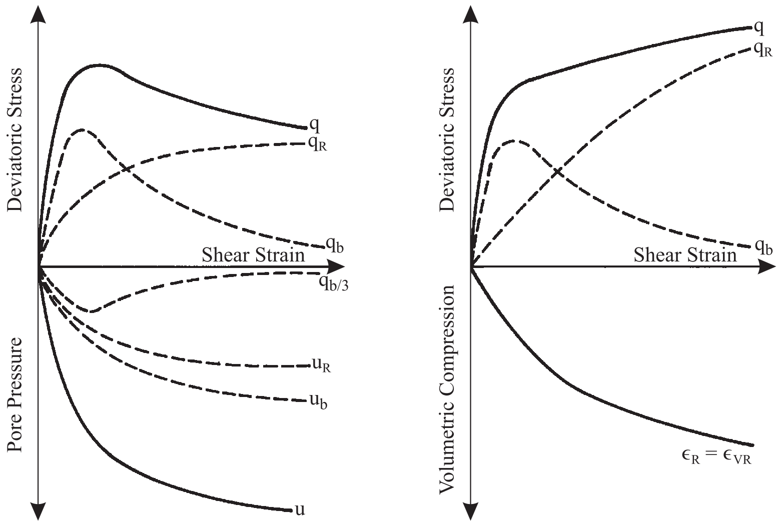

(a) (b)

Figure 5.29. Cemented, uncemented and net components of shear strength with strain. a) Undrained, and b) Drained (Nagaraj et al. 1991a).

into finite element code. A variety of hardening plasticity models have been proposed in recent years with varied degrees of success, to characterize the behaviour of cemented geo-materials (Gens & Nova 1993). These models are based on the concepts of initial void ratio and its subsequent changes with stress history effects. It is apparent from the discussions in various sections of this chapter that in the case of naturally cemented soft clays, consideration solely of the initial void ratio and its subsequent changes due to stress changes is not adequate to quantify components of strength and stiffness during the stress regime of engineering interest.

Conlon (1966), Feda (1982) and Oka et al. (1989) have indicated that the shear strength of cemented soils can be split into frictional and cementation components. The frictional component explains the usual resistance of clay to deformation without cementation. For stress increments within the yield stress, the applied stress increments will be carried entirely by cementation bonds, with negligible deformation. Beyond the yield stress level some of the bonds, which are stressed to their optimal capacity, may yield or be disrupted. The load carried by these disrupted bonds will be transferred to the clay fabric. This is possible only after compatible volume changes take place while drainage is permitted. In case drainage is not permitted, the additional stresses will be transferred to the pore water pressure. More specifically, the focal points for development of constitutive modeling of cemented soft clays by Vatsala (1989) are:

1. The resistance or the stress-carrying capacity of a bonded/cemented soil is the summation of two components, i.e.

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a) the stress carried by bonds developed due to cementation, and
b) the stress carried by a clay fabric devoid of cementation bonding.

2. The deformation of the clay is always associated with the changes in the stresses acting on the unbonded clay fabric, with the cementation bonds providing additional resistance at any strain level.

To obtain the stress-strain response, any elasto-plastic hardening model, for example the modified Cam clay model, exclusively stress-strain relations of the equivalent unbonded clay fabric, can be considered. At each strain level, to get the total response the bond resistance should be added. This can be represented mathematically as:

$$\begin{aligned} \sigma_{ij} &= \sigma_{Rij} + \sigma_{Bij} \\ \varepsilon_{ij} &= \varepsilon_{Rij} = \varepsilon_{Bij} = C_{Rijkl}\sigma_{Rkl} \end{aligned} \tag{5.18}$$

This concept is equivalent to having two stiffnesses acting in parallel for the same strain response. Separate stress-strain laws can be derived for each of the components.

5.8.3 *Description of the model for cementation component*

Within the framework of analysis of the observed stress-strain-pore pressure/volumetric change of naturally cemented soft clay during shear, it is necessary to discuss the development of analytical models to characterize the bonding component, and then that due to the clay fabric. This would allow us to examine the possibility of superposition of the same. Modelling the bond component is a new concept and will be described here in detail.

The yield surface for the bond component can be defined by the general expression represented by

$$f_B = f(q_B, p_B) - N = 0 \tag{5.19}$$

where p_B is the bond component of effective mean principal stress, q_B is the effective deviatoric bond stress and N is the hardening parameter reflecting the degree of bonding. The initial yield curve can be obtained by a detailed experimental programme. To circumvent the difficulty associated with an elaborate experimental procedure, a generalized expression has been proposed (Nagendra Prasad et al. 1998) for the yield curve based on the stability analysis at microstructural level, corresponding to experimentally determined yield points of several cemented clays. The yield curve obtained from such an analysis is a rectangle (Fig. 5.30) oriented along the K_0 compression path. This yield curve is approximated to an ellipse in order to avoid the corners (Equation 5.20).

$$\left\{\frac{(p_B/\sigma_y - k_1)\cos\theta + (q_B/\sigma_y - k_1)\sin\theta - k_2}{a_1}\right\}^2$$

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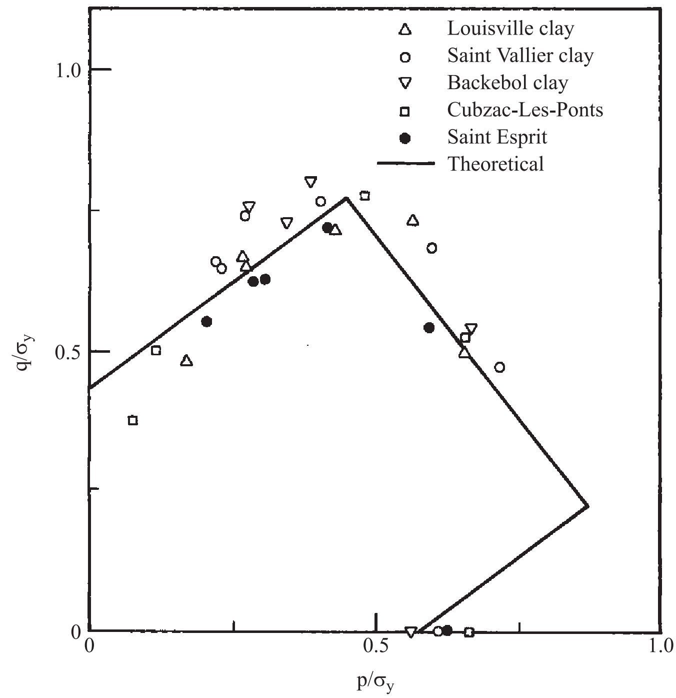

Figure 5.30. Generalized yield curve for the bond component (Nagendra Prasad et al. 1998).

$$+\left\{\frac{\left(q_B/\sigma_y - k_1\right)\cos\theta + \left(p_B/\sigma_y - k_1\right)\sin\theta - k_2}{b_1}\right\}^2 = 1 \tag{5.20}$$

The elliptic parameters, namely a_1, b_1, k_1, k_2 and q, could be readily defined for a given value of K_0. The input parameter to completely define the yield curve by the above Equation (5.15) is the yield stress value in compression, either by the isotropic or one-dimensional compression test and the K_0 value of the clay.

Based on the test results of several cemented clays, the yield curve is assumed to harden with plastic volumetric compression and to soften with plastic shear strains. For the linear hardening modulus K_B^p and an exponential law of softening with shear strains, the following expression has been advanced by Vatsala et al. 1998). The expression for hardening or softening cementation component can be written as:

$$dN = \left\{\frac{K_B^p}{p_y}\right\} d\varepsilon_v^p - \alpha N d\varepsilon_s^p \tag{5.21}$$

or Equation (5.21) can be split into two parts and expressed as

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$$dN = \left\{ \frac{K_B^p}{p_y} \right\} d\varepsilon_v^p$$

$$dN = \alpha N d\varepsilon_s^p$$

where α is softening and N hardening parameter, p_y is yield stress corresponding to isotropic compression.

The plastic strain rates are obtained by the usual methods of hardening plasticity. For the present case of the associated flow rule, the plastic strain rate takes the form

$$d\varepsilon_{rs}^p = \left\{ \frac{1}{H} \frac{\partial g_B}{\partial \sigma_{Brs}} \frac{\partial f_B}{\partial \sigma_{Bij}} \right\} d\sigma_{Bij} \tag{5.22}$$

where $H = \dfrac{\partial f_B}{\partial N} \dfrac{\partial N}{\partial \varepsilon_{rs}^p} \dfrac{\partial g_B}{\partial \sigma_{Brs}}$ is the hardening modulus.

The elastic strain rates are given by

$$\begin{aligned} d\varepsilon_v^e &= \frac{dp_B}{K_B} \\ d\varepsilon_{rs} &= \frac{d\sigma_{Brs}}{2G_B} \end{aligned} \tag{5.23}$$

where ε_{rs} and σ_{Brs} are the deviatoric strain and deviatoric stress components respectively.

With these defined features, the stiffness matrix D_B is obtained for the cementation component. The stiffness matrix for the uncemented component D_R is obtained by adopting the modified Cam clay model. The overall stiffness matrix for the soil is then obtained by adding the stiffness of the two components as:

$$D = D_R + D_B \tag{5.24}$$

The stress-strain relations for the soil are worked out incrementally, updating the stresses, strains and hardening separately for the two components at each stage.

The model developed here is very simple and the input parameters required are minimal, well defined and easily determinable. The input parameters for the uncemented soil component are the usual Cam clay parameters, i.e. λ, κ, Γ, M, and G, which can be determined from typical consolidation and shear tests on the remoulded soil. The additional parameters for the cementation bond component are K_0, σ_y, p_y, K_B, G_B, K_B^p, α, all of which can be determined from isotropic

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compression and typical shear tests on undisturbed soil. The predictions yielded by this model have been examined by Vatsala et al. (1998). Figures 5.31 and 5.32 show the experimental paths of deviatoric stress and pore water pressure development due to undrained shearing in Osaka and Saint Esprit clays respectively, along with the paths predicted by the above model. Figure 5.33 shows the drained test results of Saint Esprit clay with volumetric stain data, along with the paths predicted by the above model. The model parameters used for predictions for the Osaka clay are indicated in Table 5.1.

The close agreement between the experimental results and the predictions for the entire stain range validates the constitutive model developed.

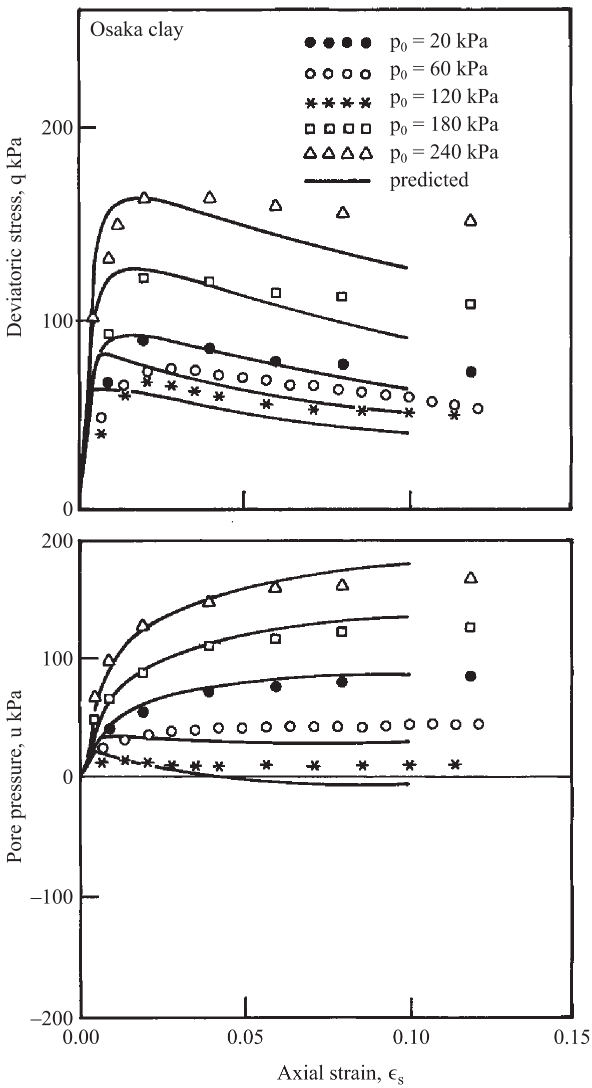

Figure 5.31. Experimental and predicted paths of undrained shear test results of Osaka clay (Vatsala et al. 1998).

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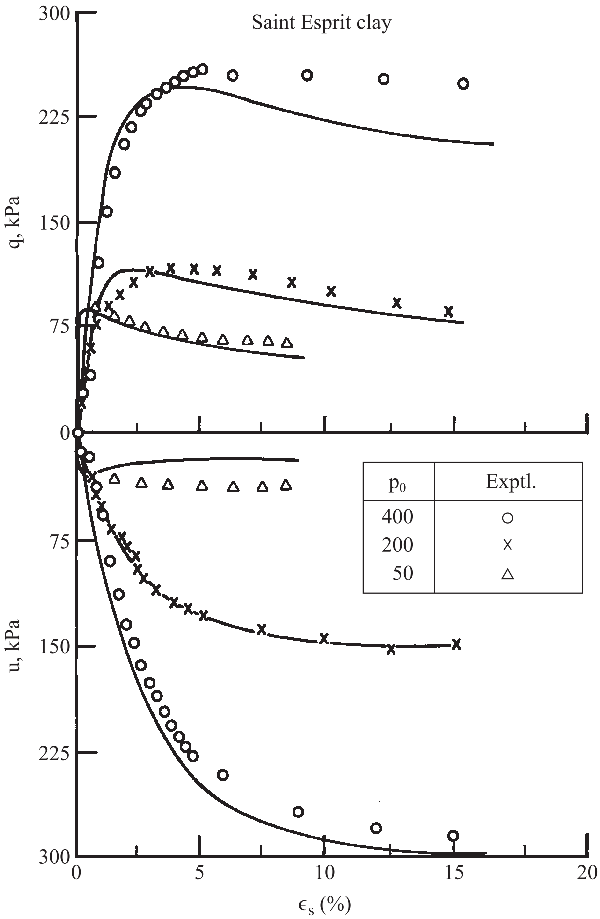

Figure 5.32. Experimental and predicted paths of undrained shear test results of Saint Esprit clay (Vatsala et al. 1998).

Table 5.1. Model parameters.

Property	Osaka clay	Saint Esprit clay
λ	0.194	0.217
κ	0.02	0.07
M	1.41	1.25
p_y (kPa)	83	80
K_B (kPa)	1200	1200
G_B (kPa)	2300	2300
$K^p{}_B$ (kPa)	240	600
σ_y (kPa)	94	130
α	5	20

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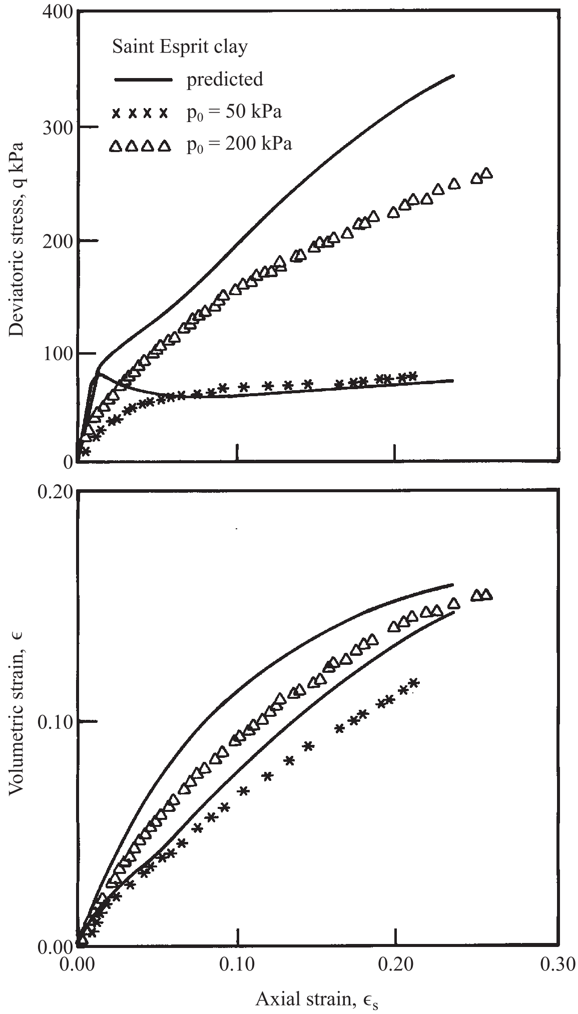

Figure 5.33. Experimental and predicted paths of drained shear test results of Saint Esprit clay (Vatsala et al. 1998).

5.9 STRESS-STATE PERMEABILITY RELATIONS

In this section an examination of the permeability characteristics of naturally cemented soft clays, similar to that in the case of saturated uncemented clays in Section 4.7, will be outlined. Such an attempt is needed to circumvent the time-consuming experimental determination of the permeability of clays and their variation with stress. It would also help to have an independent method of assessing the same, with minimum input parameters normally determined in routine investigations. Tavenas et al. (1983a) studied the variation in the coefficient of permeability with void ratio for Champlain, Canadian and Swedish clays. They found that there exists a linear relationship between the void ratio and the logarithm of the coefficient of permeability of the form

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$$\Delta e = C_k \, \Delta \,(\log k) \tag{5.25}$$

The slope of this relationship has been termed as the permeability index C_k, which is similar to the compression index C_c and is dependent on the initial void ratio. Mesri & Roshar (1974) observed that the ratio of these two indices is in the range of 0.5 to 2.0 for a wide variety of natural clays. Tavenas et al. (1983b) and Leroueil et al. (1990), after analysis of considerable volume of data on sensitive clays with liquidity index greater than unity, despite scatter in the results (see Fig. 5.34), as a first approximation suggest an equation of the form $C_k = 0.5\ e_0$.

The viability of the above form of relation within the framework of the stress-state permeability relations discussed in Section 4.7 merits examination. Consider Equation (4.46). In this relation e_L reflects the potential parameter of the clay. In the case of sensitive clays, the in-situ void ratios are often more than the liquid limit void ratios. These higher void ratios are possibly attained due to the thixotropic nature of the physico-chemical interactions due to the nature of pore fluid. Since the clays are in equilibrium without pore water separation even at higher water contents, these initial void ratios can be regarded as reflecting the potential, and in turn form the reference parameter to reckon the permeability. In the equation, in place of e_L the in-situ void ratio e_0 can be considered. Hence the modified relation would be

$$\frac{e}{e_0} = c_1 + d_1 \log k \tag{5.26}$$

to represent the permeability behaviour of sensitive clays. Although for different clays the permeability coefficients are different even for the compression of these clays to the same e, with respect to the normalized values with respect to their in-

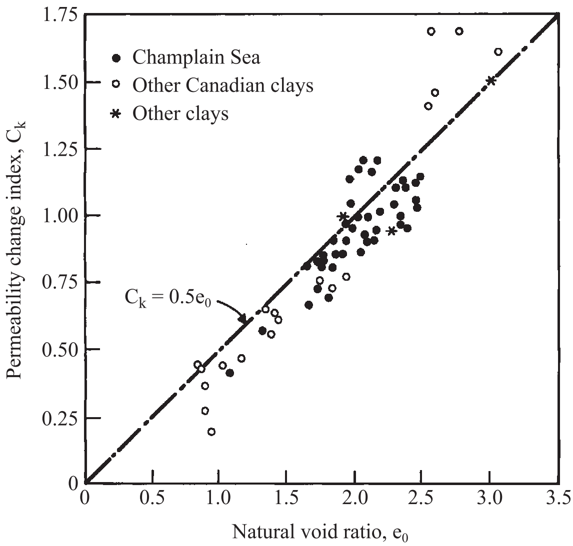

Figure 5.34. Relationship between intial void ratio and permeability index (Tavenas et al. 1983b).

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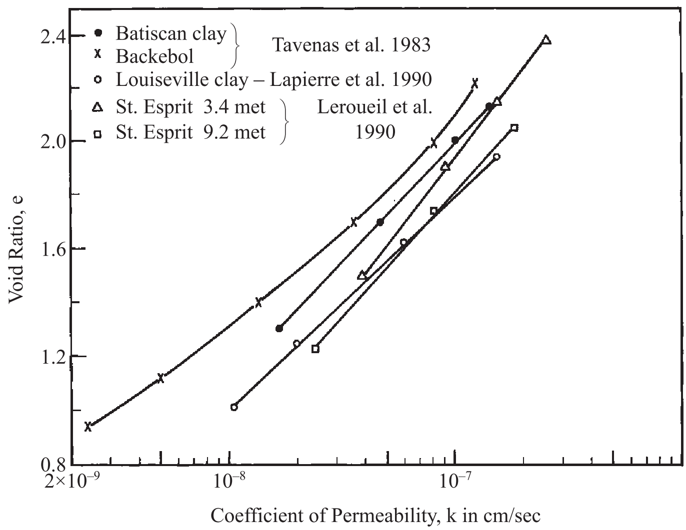

Figure 5.35. Void ratio versus permeability data analyzed by Sivakumar Babu et al. (1993).

tial void ratios a unique relation exists. The parameter (e/e_0) is an indirect reflection of the microfabric of the clay.

The differentiation of the normalized Equation (5.26) with respect to e yields

$$\frac{de}{e_0} = d_1 d(\log k)$$

$$\frac{de}{d(\log k)} = C_k = d_1 e_0 \tag{5.27}$$

The analysis of data of sensitive clays (Tavenas et al. 1983b) (Fig. 5.35) by Sivakumar Babu et al. (1993) yields the value to be 0.41. Further examination of the data of sensitive clays (Tavenas et al. 1983b, Lapierre et al. 1990, Leroueil et al. 1990) (see Fig. 5.36) on normalization, with their respective values of the initial void ratios, indicate the unique path which can be expressed as

$$\frac{e}{e_0} = 3.208 + 0.331 \log k \tag{5.28}$$

with a correlation coefficient of 0.95. In order to examine the applicability of Equation (5.24), two e – log k paths from the published literature are chosen. These pertain to Matagami clay (Tavenas et al. 1983) and Saint Esprit clay at 6.9 m depth (Leroueil et al. 1990). Table 5.2 shows the coefficient of permeability values as measured, and computed using Equation (5.28).

Since there is close agreement between the experimental and predicted values, the predictive capability of the permeability model developed is established.

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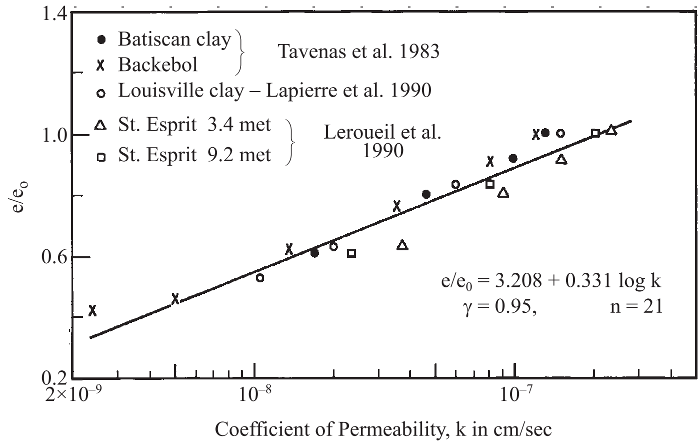

Figure 5.36. e/e_0 versus permeability (Sivakumar Babu et al. (1993).

Table 5.2. Comparison of experimental values with that predicted using Equation (5.28).

Void ratio e	k (exp.) cm/sec	e/e_0	k (predicted) cm/sec
Matagami clay	$e_0 = 2.54$	(Tavenas et al. 1983)	
2.54	2.5×10^{-7}	1.000	2.13×10^{-7}
1.80	7.0×10^{-8}	0.708	2.80×10^{-8}
0.88	5.0×10^{-9}	0.346	2.25×10^{-9}
St. Esprit clay at 5.9 m depth $e_0 = 2.03$ (Leroueil et al. 1990)			
2.03	1.85×10^{-7}	1.000	2.13×10^{-7}
1.72	8.00×10^{-8}	0.847	7.36×10^{-8}
1.40	3.75×10^{-8}	0.690	2.47×10^{-8}

However, a larger volume of data has to be analysed to re-establish numerical coefficients.

5.10 CONCLUDING REMARKS

The discussions in this chapter point to the fact that, in characterizing the engineering behaviour of naturally cemented soft clays, both particulate and non-particulate characteristics should be considered, depending on the stress level. Based on the fundamental study of the mechanisms of stress transfer and the deformation in naturally cemented clays, it has been established that the strength of cemented clays can be considered to be made up of two parts:

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1. the frictional strength of the uncemented component, and
2. the cementation bond strength.

Clays under K_0 compression exhibit different levels of resistance dependent upon the yield stress level. During shear deformations of clay it is still associated with the uncemented component of stresses, in the same way as in a remoulded clay. The cementation bonds offer additional resistance at any strain level. The permeability characteristics have been found to be affected predominantly by the void ratio, as in the case of uncemented clays. In the last chapter, the discussions will be about the possibility of enhancing this non-particulate cementation component in clays, by induced cementation using cementing agents from external sources.

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CHAPTER 6

Induced cementation of soft clays

6.1 INTRODUCTION

Soft clays in-situ are most often encountered in their naturally cemented state. The physico-chemical environment in which the deposition takes place, along with the extraneous cementing agents present in-situ and the time that has elapsed in the geological time scale in the formation of in-situ deposits, can mask the effects of stress history in these deposits. This is mainly reflected by the sensitivity exhibited by the natural soft clay deposits encountered. Even in such states, up to a particular stress level, most often unrelated to the overburden pressure, the clay exhibits resistance to compression, beyond which the clay compresses to such an extent that the equilibrium condition under the imposed stress level is attained. Depending upon the degree of this meta-stable state, compression can be of such magnitude that it may no longer be possible to transfer the superstructure loads within tolerable limits of settlements. No doubt if such settlements were to be tolerable, the clays would acquire desired levels of shear strength to ensure stability against shear failure.

Precompression of such deposits with geodrains can pre-empt the large compression potential that would otherwise occur when subjected to superstructure loads. The stable state reached would satisfy the practical need to ensure stability, both from the point of view of shear strength and deformation. This mode of tackling the problem is time-consuming, and cannot always be employed. Another alternative which merits examination is to induce cementation with supplementary cementing agents, such as lime and cement, introduced into soft clays in-situ. The resistance to compression and the strength developed by adding cementing agent to high water content clays takes place after a due rest period. To distinguish naturally cemented clays from soft clays where cementation is due to addition of cementing agents over a very short span of time compared to the geological time scale, soft clays of this type are designated as *Induced Cemented Soft Clays*. The term soft clays is intended to reflect the high potential of clays for compression beyond yield stress levels, due to their high in-situ initial water contents which could be higher than the water content at their liquid limit state.

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6.2 RECENT DEVELOPMENTS

Despite a lack of in-depth understanding of the development of strength when soft clays are mixed with cementing agents, the practical application of these methods can be traced over the past two decades. In 1975, two papers on the Deep Mixing Method were presented and discussed during the Fifth Asian Regional Conference on Soil Mechanics and Foundation Engineering, held at the Indian Institute of Science, Bangalore, India. One of them was on the Swedish Lime Column method developed by Broms & Boman (1975) and the other was on the Japanese Deep Lime Mixing Method developed by Okumura & Terashi (1975). The general recognition was that a new soft ground improvement method was being introduced to the profession. Development of Japanese deep mixing originally started in the 1960s at the Port and Harbour Research Institute, Tokyo, using either granular or powdered lime as cementing agents (Okumura et al. 1972). The method was called the Deep Lime Mixing method (DLM). Subsequently the development and practical use of slurry deep mixing methods such as DCM and CMC occurred (Kawasaki et al. 1981). Stimulated by the success achieved with these, variations of the methods were developed, particularly by the use of Portland cement. This method is the Cement Deep Mixing method (CDM) (Terashi et al. 1979, Terashi & Tanaka 1981). The basic aspects of incorporation of cementing agents in-situ to high water content clays have been discussed briefly in Section 2.9.

In general, the strengthened soil produced by in-situ mixing with cement binders is a composite ground with columnar inclusions, although block, wall, and grid types are also widely used to meet various practical needs (Yonekura et al. 1996). As there has been an extensive record of the successful application of this mode of ground improvement, these techniques can be seen as well established. The basic technique has undergone rapid changes to enhance its scope and versatility. For example, by providing vertical mixing vanes and a gearbox in addition to the usual horizontal blades in the mixing units, a monolithic rectangular shaped mixing zone can be realized (Watanabe et al. 1996). By using a superjet, a large diameter (up to 5 m) soil improvement, extending to desired depths, is possible with a small bore hole (Yoshida et al. 1996). A new system, JACSMAN (Jet and Churning System MANagement), which combines mechanical mixing (churning) and a jetting system, with the distinct advantage of creating a uniform diameter mixing system, has been developed (Miyoshi & Hirayama 1996). It uses a cross-jetting system which gives uniform diameter of mixing irrespective of the relative strength and stiffness of the soil layers with depth. Very extensive use of in-situ deep cement slurry mixing of soft clay in the Trans-Tokyo Bay Highway Project has been reported by Tatsuoka et al. (1997).

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6.3 PRACTICAL SIGNIFICANCE

In broad perspective, the main objectives of improvement are to increase strength, to control deformation and to alter the permeability of loose compressible soils. The basic aspects, and an overview of deep mixing technology, have been provided by Porbha (1998). More specifically the technique has been used extensively to achieve various objectives such as

1. increasing bearing capacity,
2. reducing settlement,
3. prevention of sliding failure,
4. protecting structures surrounding the excavation site,
5. controlling seepage and as a cut-off barrier,
6. preventing shear deformation such as in liquefaction mitigation,
7. increasing the ability to tunnel in soft ground, and
8. ground anchorage.

A number of applications related to the above purposes, already completed, have been discussed in detail by Porba et al. (1998). Deep mixing techniques have been used extensively to strengthen the substratum below the foundation of structures such as tanks, towers and bridge abutments, embankments, underground facilities, retaining structures and high rise buildings. In the area of marine and waterfront structures, this technique has been used for construction of quay walls, wharf structures and breakwaters. This technique is used as a cutoff wall for dams, dykes and river banks. Shield tunnelling in soft clay, cement anchors for soil nailing, and vibration reduction by wave-impeding blocks, are all innovative applications of in-situ deep mixing technology (Porbaha et al. 1998). The unique characteristics of this method, by which solidification of soft soil is possible within a practical rest period after cementing, have been entirely responsible for such a wide spectrum of practical applications.

6.4 NEED FOR BASIC WORK

In retrospect, developments in the field techniques, plant and machinery and extensive practical applications have far surpassed the basic understanding of strength development in soft clays due to cementation. Consequently, the Parametric assessment needed for rapid implementation of the deep in-situ mixing method in the field is at present a difficult task. Since the zone of influence of deep mixing in the field can be predefined, the volume of soft clay involved can be assessed. Hence the unit quantity of cement (percent by dry weight of clay or kg per cubic meter) to be intermixed can be utilized. The rest period, which is the number of days allowed for soil strength to reach the desired level before any construction activity can be undertaken, is also controlled by field engineers. Thus, the identification of various parameters affecting strength development

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merits identification, and appropriate laboratory studies under simulated conditions can provide the answer. Analysis of data from such studies might produce a simple method by which parameters to be used in the field could be adopted after due verification.

6.4.1 *Specific questions*

The questions which must be answered in order to develop a practical method to implement deep mixing methods are:

1. What is the difference between conventional stabilization with binders like lime and cement, as is usually done in embankment construction and highways, and in-situ deep mixing of high water content soft clays with lime or cement, and allowed for different rest periods for strength development at its initial state?
2. Can the strength development due to induced cementation be predicted? If so, what are the dominant parameters influencing specified strength development?
3. Can the level of admixtures for a defined rest period be assessed to attain a desired level of strength development? Is inter-changeability possible between the level of admixture and the rest period to reach the same level of strength?
4. Can the engineering properties of induced cemented soft clays be assessed? What are the limitations?

6.5 COMPACTED CEMENT ADMIXED CLAYS VERSUS INDUCED CEMENTED SOFT CLAYS

6.5.1 *Admixed clays*

In the conventional method used by highway engineers, clay in its relatively dry powdered state is thoroughly mixed with a predetermined lime or cement content using either a disc harrow or a ripper. With this method it is difficult to mix the cementing agent deeper than about 300 mm. Then the moisture content of this clay mix is made up to its optimum moisture content so as to be able to compact it to its maximum dry density. At this stage, when the clay is mixed with cementing agent it is still a modified clay. The clay-cementing agent interaction results in the grouping of particles into clusters. The stability of these clusters depends upon the dosage of the cementing agent and the time lapse after preparation of the mix. Upon compaction of such a mix, a clay-water-air system with a specific microstructure is formed. The engineering properties of the compacted mass are governed by the pre-effective stress locked in the compacted condition of the soil. It has been found that the synergistic effects between clusters are markedly influenced by the delay in compaction after the mixes have been prepared (Nagaraj et al. 1981). Unless the desired levels of synergy between units of different clay

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clusters are realized by compaction, the overall strength and stability of the compacted medium cannot be obtained due to cementation with time.

In the case of in-situ mixing of lime or cement with a clay which already has a water content at the liquid limit level or higher, it is not the clay with which cementing agents are mixed, but with an interacting clay-water system. At this stage, matric suction of the order of 5 to 6 kPa (internal effective stress) is already operative to balance the forces of interaction. This results in a specific pattern of the clay fabric. Hence the cementing agents would have freedom to drift to different cementing sites in the fabric so as to result in a structured state with a definite initial fabric pattern.

For example, it has been noticed that when a black cotton soil (w_L = 75%) is mixed with 4% lime in its dry state and its liquid limit is determined, the value was reduced to 61%, whereas with the same percentage of lime added to the same clay at its liquid limit state, the value did not alter as a result of this admixture (Muttaram et al. 1996). This suggests that in the first case, due to aggregation of clay particles, the potential might have been reduced, resulting in lowering of water content at its liquid limit. However, in the second case the liquid limit water content did not change, as particle aggregation was inhibited possibly due to the formation of the initial fabric by the clay-water system. It can be inferred that when cementing agents are added to a clay in its dry state it is to a clay material, whereas in the case of high water content clays the cementing agents are added to a stable clay fabric in equilibrium under matric suction. The need for this distinction is again discussed while characterizing strength mobilization due to induced cementation. The observation by Locat et al. (1996) regarding the index properties of clay at initial water contents of 341 and 351%, far higher than its liquid limit of 67%, mixed with higher lime contents of 5 and 10% and prolonged curing for 100 days, is different. The liquid limits of the clay under these conditions are far higher, being 181 for 5% lime and 213 for 10% lime admixture. According to the surface areas of clays reported in untreated and lime mixed states, this can be attributed to the growth of stable clusters during the curing period, which itself holds considerable non-participating water reflected in much higher water contents than at their liquid limit state.

In place of hydration of cement, improvements in the engineering characteristics of lime-soil mixtures can be attributed to three basic reactions: cation exchange, flocculation and pozzolanic reactions. The relative significance of these reactions has been brought out in detailed laboratory investigations by Narasimha Rao et al. (1993), Narasimha Rao & Mathew (1995, 1996), Rajasekaran & Narasimha Rao (1996) and Mathew & Narasimha Rao (1997, 1997a). X-ray diffraction and electron microscopy data has provided additional information in the analysis of the data.

6.5.2 *Diffusion*

Lime piles, which essentially consist of holes in the ground filled with lime, have been successfully used to transform soft clay soil into a composite soil. Lime piles

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are formed both by compacted quicklime or from lime slurry. When there are local differences in concentration in an otherwise uniform body of solution, these differences tend to decrease with time and finally disappear. This process is diffusion. The analysis by Rogers & Glendinning (1996) has advocated that migration of lime from piles into the surrounding clay provides the major stabilizing mechanism. The lime column and lime slurry injection methods were used to study the diffusion of lime radially from the source. In the lime column method, the unslaked lime, calciun oxide, was placed dry or in the form of slurry in holes punched in the soil by an end closed pipe. In the lime slurry method, injection is done under pressure. The migration distance of lime has been studied using pH measurements (Narasimha Rao & Mathew 1996). The formation of various cementation compounds due to soil-lime reactions were identified by X-ray diffraction studies. The test results indicate that a sufficient amount of lime is diffused into the soil systems with time (Rajashekaran & Narasimha Rao 1996a). It is not yet clear to how the diffused lime would impart strength to the clay through the micro-fabric of clay already prevalent in its uncemented state at liquidity indices greater than unity.

6.6 CHARACTERISTICS OF INDUCED CEMENTED CLAYS

The obvious fact of induced cementation is to impart enhanced strength to the soft clay. The determination of the strength developed would have sufficed if the deformation characteristics did not play any significant role. The void ratio of the in-situ induced cemented state of the clay due to high initial water content is not likely to be very different from that of its initial state, except in the case of added quicklime. Added to this, the loading of the composite ground can be far higher than that of the cementation bond strength. Hence, understanding the basic mechanisms of induced cementation and analysis of cemented soft clay behaviour linked with deformation, is advantageous for practical exploitation and in the development of appropriate constitutive modelling.

A general examination of the compression paths of soft clay in different states (see Fig. 6.1) enables us to make the following observations:

1. In the completely mechanically remoulded state, the compression path is linear as in the case of the intrinsic compression path.
2. Due to natural cementation as in the case of soft and sensitive clays, the clay offers resistance to compression up to a particular stress level, beyond which, as the stress level increases, the clay undergoes pronounced compression far higher than that of normally consolidated path. This happens to reach a compatible equilibrium state with the stress level. As explained earlier, the yield stress of induced cemented clays is distinctly different from the preconsolidation pressure due only to the effects of stress history. In all such cases the compression path is characteristically to the right of the intrinsic compression path.

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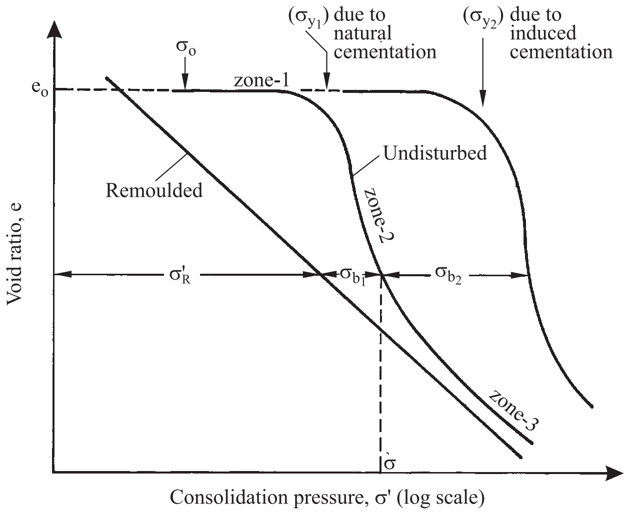

Figure 6.1. Compression paths of normally consolidated, naturally cemented and induced cemented states of the same clay – schematic representation (Nagaraj et al. 1997).

3. By in-situ deep mixing, the undisturbed clay is completely disturbed mechanically, but the water content is not altered to any recognizable extent. What is mainly lost is only one part of the clay structure, i.e. the bonding. Due to the loss of anisotropic characteristics, if any, of the undisturbed clay, the fabric is altered only to a minor extent. For all practical purposes, as long as the water content is not altered, the clay fabric can be regarded as being the same as in the case of the undisturbed situation without cementation bonding. At this stage the soft clay is mixed with cementing agents. After some time, the new cementation bonding develops strength in the fabric far higher than that of the naturally cemented state. The magnitude of yield stress due to induced cementation depends on the water content, binder content and the rest period. Characteristically the compression path is further right than that of the naturally cemented state.

6.6.1 *Microstructural state*

Analysis of experimental investigations into the compressibility and permeability characteristics of clays, both in their uncemented and induced states, along with the pore size distribution data (Nagaraj et al. 1995, Yamadera et al. 1998) enable us to advance the following inferences (see Figs 6.2 and 6.3).

1. The induced cemented clays exhibit the same order of permeability as in the uncemented state, when both are reckoned at the same void ratio. This indicates that the pore geometry in both cases is of the same pattern.

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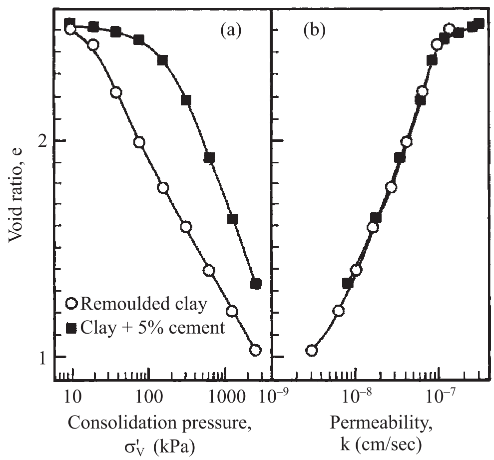

Figure 6.2. Permeability of reconstituted and cemented Ariake clay (Yamedara et al. 1998).

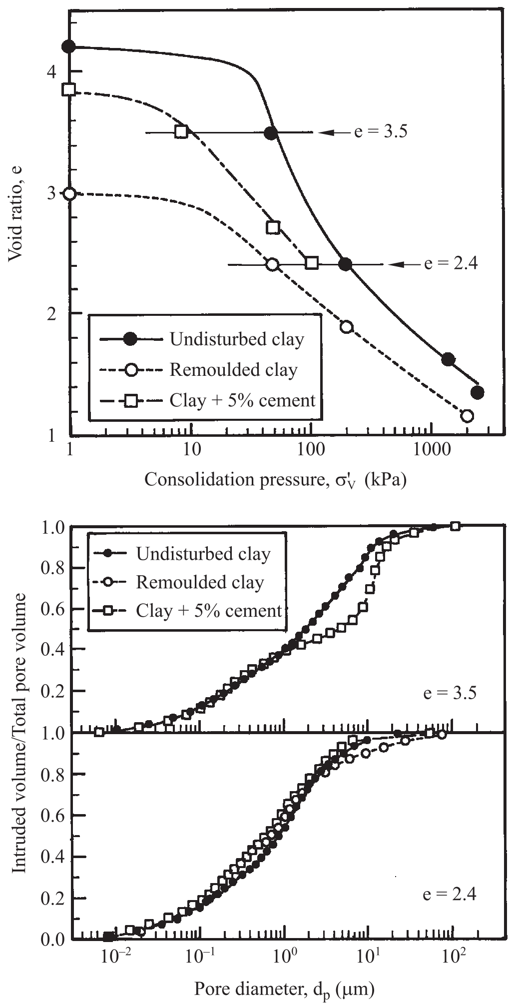

Figure 6.3. Pore size distribution in Ariake clay at the same void ratio in the undisturbed, remoulded and cemented states (Yamadera et al. 1998).

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2. Hence, the role of induced cementation is to weld the fabric. This results in additional resistance, as observed in its resistance to compression up to yield stress.
3. As a consequence, the total resistance of induced cemented clay arises due to the internal stress field, a distinct characteristic of interacting particulate materials, to create the fabric which is being further strengthened by cementation bonds (Fig. 6.4).
4. The resistance to compression and shear strength increase, with the rest period, due to enhancement of bonding for the same level of cementing agents.

Similar analyses for lime-admixtured soft clay or lime-diffused clays have not yet been advanced due to lack of the appropriate data. However, the attention of the reader is drawn to the recent comprehensive laboratory investigations reported by Locat (1995), Locat et al. (1990, 1996) on strength, compressibility and permeability characteristics of inorganic clays treated with lime.

It is interesting to observe that the compression paths of treated Louiseville clay with increasing percentage of lime moves to the right of the compression path of untreated clay, similarly to those of clays treated with cement (see Fig. 6.5). The variation of permeability at void ratio of 2.0 as the lime content increases is also indicated. It is interesting to note that even though resistance to compression increases to the extent of 50 times that of the uncemented state, the variation of permeability is very limited, well within the same order. It can be inferred that the fabric is strengthened by calcium carbonate cementation in a similar way as with hydration of Portland cement in the intercluster spacings. This is

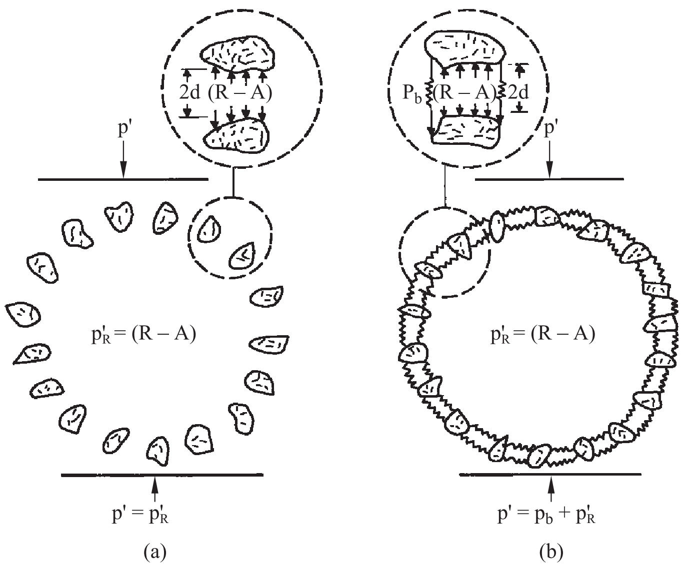

Figure 6.4. Possible clay fabric and its cementation. a) Mechanically disturbed state, b) Induced cemented state.

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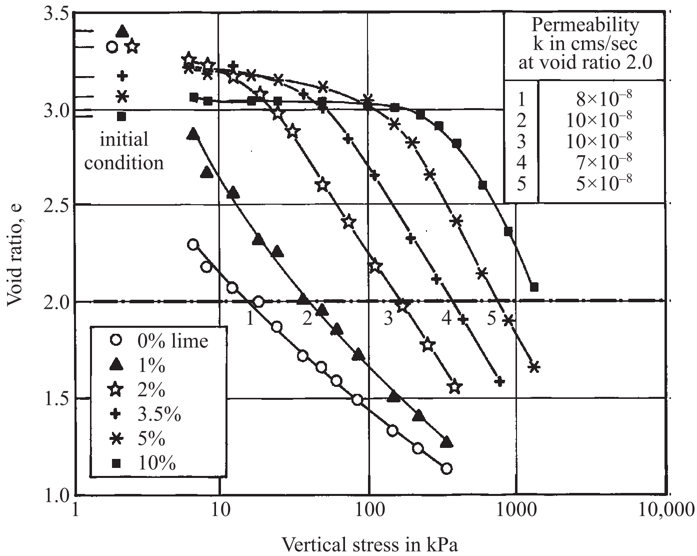

Figure 6.5. Compression paths of samples of Louiseville clay with distinct increase in yield stress as the lime content is increased (Locat 1995).

further to be reinforced by additional information on pore size distribution data at the same void ratio but with lime as the admixture.

6.7 STRENGTH DEVELOPMENT

A practical method for predicting the gain in strength after different rest periods has been developed (Nagaraj et al. 1996). The method derives its basis from Abrams' law and its generalization (Nagaraj & Zahida Banu 1996), which are extensively used in concrete technology.

Abrams' Law forms one of the basic principles on which the development of strength in concrete at a specific water/cement ratio is analyzed. It states that:

> *For given concrete ingredients, age and curing conditions, the STRENGTH of hardened concrete is determined exclusively by the ratio of free water content (kg/m^3) to the cement content (kg/m^3) in the mix. Strength is independent of the absolute contents of free water and cement in the mix (Abrams 1918).*

Subsequently Abrams' law was expressed in the functional form by Bolomey (1927) as:

$$S = A\left\{\frac{c}{w}\right\} + E \tag{6.1}$$

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Considering cement-based composites as chemically-bonded ceramics (CBC), the consequent strength development with age is essentially a constant volume solidification process, such that the hydrated gel particles fill the space resulting in compatible gel/space ratios. By analysing the of extensive data used in the graphical method of British mix design (Teychenne et al. 1988) (see Fig. 6.6), a generalization of Abrams' law has been evolved (Nagaraj & Zahida Banu 1996). By considering the strength at a water/cement ratio of 0.5 as the reference state, to reflect the synergy between the different constituents of concrete for a situation where the coarse aggregate characteristic strength exceeds the cement mortar matrix strength, the generalized relation is of the form:

$$\left\{\frac{s}{s_{0.5}}\right\} = a + b\left\{\frac{c}{w}\right\} \tag{6.2}$$

In this relation the strength ratio $S/S_{0.5}$ reflects the same order of gel space ratios, at a particular age, compatible with the physico-chemical characteristics of cement and water for a particular set of concrete ingredients.

In the case of cement-based composites there is continuity in the structure of hydrated cement, with the coarse and fine aggregates being embedded in the cement paste matrix. Hence, the water/cement ratio is the dominant factor controlling strength development. On the other hand, in the case of clays with in-situ water content in the same range as their liquid limit water contents, since the clay fabric would have been formed due to internal stress fields, the role of cementing agents is to weld the fabric at inter-cluster sites. The water content at liquid limit

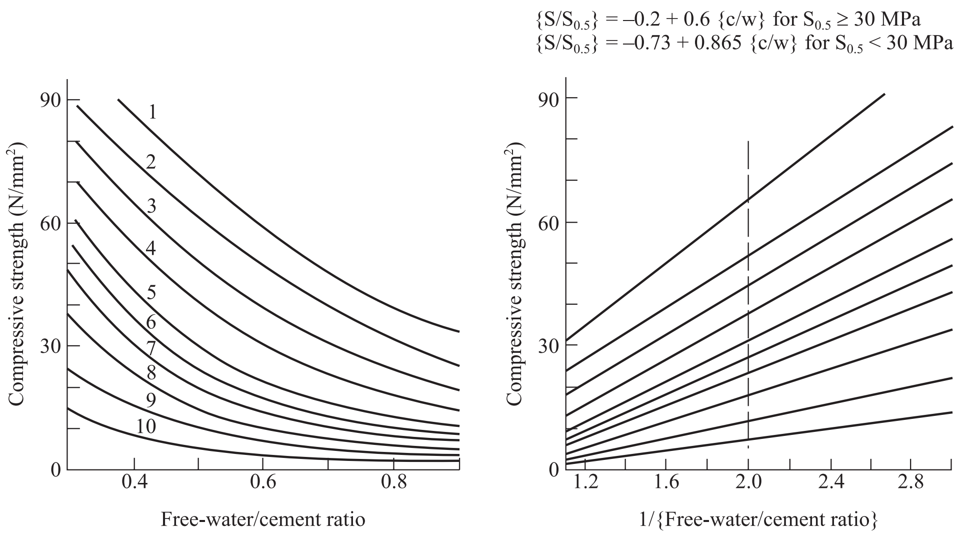

Figure 6.6. Compressive strength of concrete versus free water/cement ratio relationships and their generalization (Nagaraj & Zahida Banu 1996).

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would reflect the magnitude of inter-cluster sites and not the water/cement ratio Nagaraj et al. (1996). As the liquid limit water content of the clay increases the number of inter-cluster sites to be welded increases, and the strength mobilized is decreased. Conventionally, the lower level of strength development is attributed to the increase in the specific surface of the clay. The work of Lee et al. (1997) is an attempt to identify the role of cement/soil + cement ratio at different water/soil ratios in strength development. A simple method to determine the transitional combination of soil, cement and water, when either water content or water/cement ratio has to be identified and considered, has yet to be evolved in developing a method to assess strength development. Typical strength plots for variation of the liquid limit water content of inland as well as marine clays are shown in Figure 6.7. The following relationship between different parameters can be advanced by the multiple linear regression analysis of data both for land and marine clays.

$$\left\{\frac{S_D}{S_{14}}\right\} = a + b\ln D + \left\{\frac{c}{w_L}\right\} + dp \tag{6.3}$$

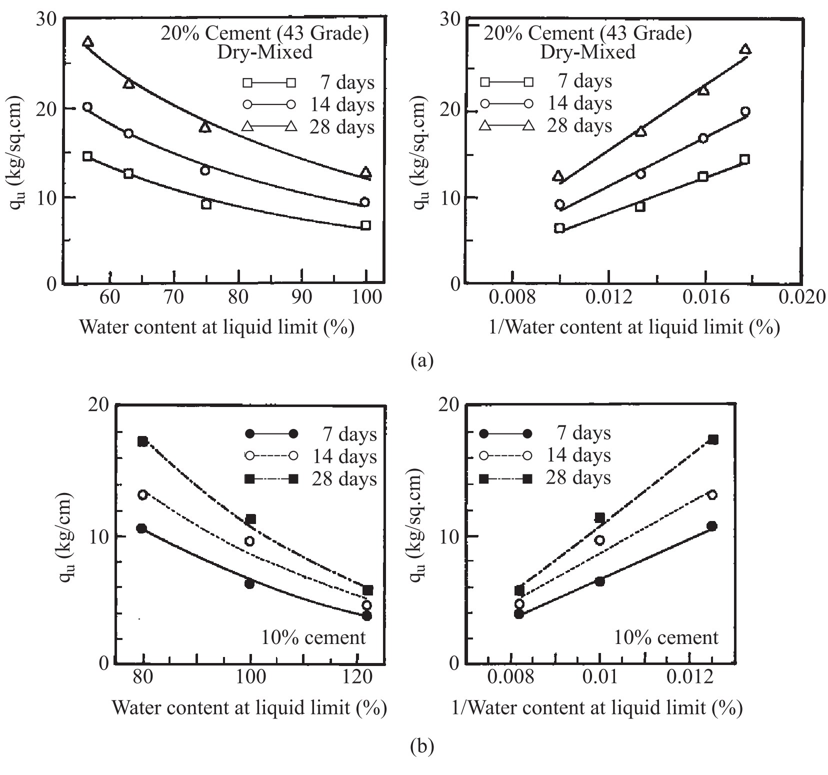

Figure 6.7. Strength versus liquid limit water content and inverse of liquid limit water content of clays (Nagaraj et al. 1998b). a) Inland clays, b) Marine clays.

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where S_D is the strength after D days of curing time, S_{14} is the strength for 14 days rest period, w_L is the liquid limit water content of the clay in percent, P is cement by percent by dry weight of the clay and a, b, c, d are constants with values for the data analyzed being $a = 0.189$, $b = 0.298$, $c = 3.546$ and $d = -0.00062$. By giving due weight to all four terms, it can be seen that strength contribution to the extent of 98% is due to the first two terms with these constants. Hence the relation gets reduced to:

$$\left\{\frac{S_D}{S_{14}}\right\} = a + b\ln D \tag{6.4}$$

Even though the water content and the cement content account for the number of sites to be cemented and the availability of the cementing agent per inter-cluster site, the development of strength during the rest period takes care of the contribution of all factors to a significant degree. It is not adequate if the investigations are terminated at this stage, since cement content as a distinct parameter is not accounted for. Such an attempt would enable calculation of the cement content in practice.

6.8 INTER-RELATIONS BETWEEN STRENGTH AND REST PERIOD, CEMENT CONTENT

Consistent with the above analysis of strength development in induced cemented soft clays, we need to examine whether it is possible to estimate how far one can reduce curing time by enhancing the percent of cement admixture. For this, the inter-relationship between strength, cement content and rest period must be elucidated, similarly to the connections between strength, age, and the water/cement ratio in concrete technology (Nagaraj et al. 1997). For this purpose the strength data for different inland and marine clays mixed with different percentages of cement and left for the same rest period are examined (Fig. 6.8). On interpolation, the linear plots intercept the x-axis at different values of water contents. This implies that for every percentage of cement content there is a particular cut off liquid limit water content of the clay beyond which no strength development takes place. It means that the cement content per inter-cluster site is so little that no cementation bonding takes place. The minimum percentage of cement to initiate cementation, as determined by iteration of the data on the cut-off liquid limit against the percentage of cement, results in the following relation for both inland and marine clays (Nagaraj et al. 1998a).

$$\left\{\frac{P_e}{w_L}\right\} = 0.3864 + 0.00215P_e \tag{6.5}$$

where: P_e is the minimum percentage of cement and w_L is the percentage liquid limit water content.

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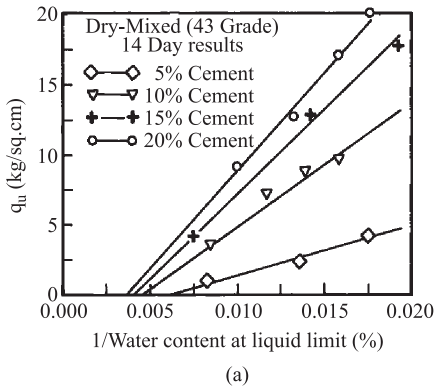

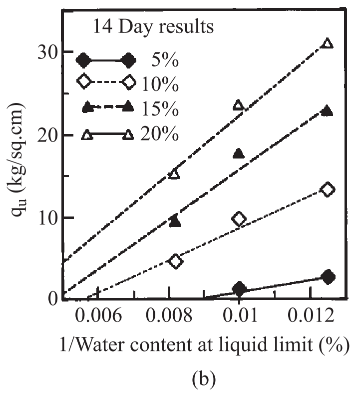

Figure 6.8. Strength versus inverse of water content at liquid limit for different percentages of cement for 14 day rest period (Nagaraj et al. 1997, Yamedara et al. 1998). a) Inland clays, b) Marine clays.

To obtain the functional relation between strength ratio and reduced percentage of cement, for example 10 r (for 10% cement admixture the reduced percentage is according to the value of w_L), the strength data of different clays with a 10% cement admixture can be analyzed. Out of the two relations obtained for inland and marine clays, the following has been advanced for further analysis.

$$\left\{\frac{S_{Pr}}{S_{10r}}\right\} = -0.0014 + 0.1003P_r \tag{6.6}$$

By neglecting the constant and rounding the value of the slope to 0.1 for all practical purposes, the above relation can be expressed as

$$\left\{\frac{S_{Pr}}{S_{10r}}\right\} = 0.1P_r \tag{6.7}$$

The specific advantage of Equation (6.7) is that a reduced percentage of cement need not be considered separately for laboratory experimental work, since the strength ratio at different percentages of cement and the cement percent ratios happens to be a constant.

$$\left\{\frac{S_{P_1}}{S_{P2}}\right\} = \left\{\frac{P_1}{P_2}\right\} = aP_1 \tag{6.8}$$

The above synthesis of the parameters responsible for strength development suggests that the slope to define strength mobilization for a constant curing time as the cement content increases, has to be determined by laboratory investigations. For example, a clay at a water content close to its liquid limit is mixed

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with 7.5 or 12.5% cement by dry weight of clay, and left for a 14 days rest period. Then the unconfined compressive strength of the cemented clay is tested. This is all the information to be generated in the laboratory. To estimate the strength increase, it is necessary to find the slope, *a*, of the line to define the incremental increase in strength as the cement content increases. Since S_{P2} would have been determined experimentally for P_2, the inverse of the reduced percentage corresponding to P_2, from Equation (6.8), itself would be the value of the slope defining the rate of strength development as the cement content varies, for the same curing time.

Combining Equations (6.4) and (6.8) yields a general equation with all the parameters:

$$\left\{\frac{S_{P_1,d}}{S_{P,14}}\right\} = \frac{P_1}{P}(0.189 + 0.298 \ln D) \tag{6.9}$$

Where $S_{P1,d}$ = strength of induced cemented clay to be estimated at a cement percent P_1 and a curing time of D days, $S_{P,14}$ = strength of induced cemented clay at P percent after 14 days curing time (trial set data).

For 14 days curing time and $P = P_1$, the right side expression reduces to unity, and the strength mobilized is equal to the reference strength data obtained from trial test data.

At this juncture it would not be possible to advance similar inter-relationships between different parameters for analysis of strength development in soft clays mixed with different percentages of lime. However, Locat et al. (1990) after analysis of strength developed as a function of water content and quicklime concentration after 30 days of curing, have shown that it follows a power law of the form

$$S_u = aw^b \tag{6.10}$$

where *a* and *b* are constants that for a given isograde (curves at constant lime concentration) are influenced by the nature of the clay, curing time and lime concentration. This mode of characterization of strength development suggests that similar relations (Equation 6.9) as developed for cement-admixed soft clays appears to be a possibility, provided appropriate data considering all the required parameters is generated and analyzed.

Another factor that merits consideration in the practical use of this parametric approach is the need to arrive at a target strength, for which the cementing agents and the curing period are to be computed. There is adequate evidence that, due to the large volume of soft clay involved in deep mixing, whatever may be the refinements in the techniques employed, non-uniformity in mixing occurs which results in variation in strength development. Hence the targeted value would be higher than the value assumed in the design, for which appropriate parameters from the above parametric assessment can be arrived.

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6.9 CONCLUDING REMARKS

The basic analysis of induced cemented clays presented in this chapter, and development of a simple method to arrive at appropriate parameters for field application, would help in making engineering decisions where in-situ deep mixing methods are used to create a composite soft ground with columnar inclusions. With more field data being generated on marine clays of different regions, and used to refine the analysis, the fond hope is that this approach will gain momentum for soft ground improvement. Still many of the aspects related to induced cementation have not been addressed. The following aspects merit intense examination in order to enhance the applicability of the method and fix the various parameters in the use of the deep mixing method.

1. Applicability of the relations to soft clays whose in-situ water contents are far higher than water content at their liquid limit. In practice, such situations would be encountered where jet grouting is resorted to.
2. The identification of transition combination of clay, cement, and water content where appropriate consideration of either the water/cement ratio or water content can be made in analysing strength development with rest period.
3. The validity of relations when different cementing agent, such as lime or a combination of cement and lime, are resorted to.
4. A methodology to fix the upper limit of the cementing agent, beyond which no appreciable strength gain takes place.
5. For any particular level of cementing agent, a methodology to determine the upper limit of cementation bond strength with rest period.
6. Validation of the method to obtain parameters for the same level of strength for soft clays with different liquid limit water contents.

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Epilogue

Urbanization, industrial development and oil exploration, due to logistics, most often take place in the coastal regions. Hence soft clays are invariably encountered in geotechnical engineering practice. Stress, time and environment play a dominant role in the formation of soft clay. They are mutually exclusive processes, with the principle of superposition of their effects not being tenable. The shear stress-strain behaviour of in-situ soft clays, although seemingly akin to that of stress-dependent overconsolidated clays, in reality is quite different when stress-strain-pore pressure or volumetric strains are considered in totality. With the developments presented in this treatise, the geotechnical engineer has a means to examine soft clay behaviour which varies from the classical treatment. It also provides a methodology to analyze and assess soft clay behaviour rapidly in order to meet time constrains and also to enable him to have an independent check on the test results provided by using minimum input parameters generated in routine investigations.

The classical developments in soil mechanics rely heavily on the analysis of soft clay behaviour, linking its state with the overburden stresses. This is not always appropriate. In fact, the effects of time and environment are subdued in the natural state of clay encountered. The basic framework developed and discussed in Chapter 3 considers the response of clays devoid of stress history and ageing effects. The intrinsic state/effective stress relations form the reference framework. In relation to this, the data on the prevalent overburden stress and the intrinsic state (present physical state normalized by the void ratio at the liquid limit of clay, which is a reflection of the potential of the clay-pore fluid interactions) can be analyzed. This enables identification of which of the three factors, stress, time and environment, predominantly control the responses of the clay due to subsequent imposed loading. With this identification, the consequent engineering properties of soft clays can rapidly be estimated for integration with the analysis, so as to arrive at an acceptable design of a substructure. The need for an analysis, a methodology to analyze the constitutive relations in the case of soft clays with natural cementation, have been elucidated. This situation arises since the responses of natural soft clays are governed by non-particulate and particulate considerations dependent upon the stress level considered. The combined influence of non-particulate and particulate characteristics has been integrated in such a way that

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the direct implementation of constitutive relations into finite element codes becomes viable when seeking solutions to soil-structure interaction problems.

The brief discussions of different methods, detailed in Chapter 2 on soft ground engineering, identify situations where analysis and assessment of soft clay properties would facilitate the adoption of an appropriate methodology. The assessment methods detailed in this treatise would also further enable us to embark on the observational approach to estimate the degree of ground improvement realized at the various stages of implemention of ground improvement techniques. More specifically, the discussions of induced cemented clays would enable consideration of the interplay of various factors, such as dosage of admixtures and the rest period, on strength gain in high water content soft clays. This would facilitate engineering decisions on the use of the in-situ deep mixing methods presently so extensively used in many parts of the world.

The authors are aware that there remain several other aspects of soft clay behaviour, not addressed here, which merits elucidation. Even in the case of those discussed, it is still possible that the inferences arrived at need further substantiation to ensure their validation in a general sense. It is believed that this treatment of the subject, different from the usual conventional one, will perhaps draw the attention of geotechnical engineers, for critical examination and lead to subsequent rigorous studies.

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