Developments in Geotechnical Engineering 20

SOFT CLAY ENGINEERING

edited by

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ELSEVIER SCIENTIFIC PUBLISHING COMPANY
Amsterdam — Oxford — New York — 1981

ELSEVIER SCIENTIFIC PUBLISHING COMPANY
Molenwerf 1,
1014 AG Amsterdam, The Netherlands.

Distributors for The United States and Canada:

ELSEVIER/NORTH-HOLLAND INC.
52, Vanderbilt Avenue
New York, N.Y. 10017

Library of Congress Cataloging in Publication Data
Main entry under title:

Soft clay engineering.

(Developments in geotechnical engineering; v. 20)
Bibliography: p.
Includes index.
1. Engineering geology. 2. Clay. I. Brand,
Edward W. II. Brenner, Rolf Peter, 1937–
III. Series.
TA705.S583 624'.1513 79–13386
ISBN 0–444–41784–2

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Elsevier Scientific Publishing Company, Molenwerf 1, 1014 AG Amsterdam

Printed in Hong Kong by Libra Press

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Dr Brand has acted as a geotechnical consultant for large projects in many countries, and he has lectured to universities and professional bodies throughout the world. He is currently President of the Southeast Asian Geotechnical Society, and a member of the Sub-committees on Landslides and Site Investigation of the International Society for Soil Mechanics and Foundation Engineering.

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Dr Brenner is the author of numerous technical papers which deal with a wide range of topics, including soil dynamics, pile foundations and engineering geology. He is presently a member of the Sub-Committee on Soil Sampling of the International Society for Soil Mechanics and Foundation Engineering.

PREFACE

Soft clay deposits are widespread, and they present very special problems of engineering design and construction, many of which are not common to other earth materials. By definition, soft clays are of low strength and high compressibility, and many are sensitive, in that their strength is reduced by disturbance. Foundation failures in soft clay are comparatively common, and surface loading in the form of embankments or shallow foundations inevitably results in large settlements which must be accommodated for in design, and which invariably necessitate life-long maintenance of engineered facilities. Virtually all structures on soft clay except earth fills and very small buildings have to be supported on deep foundations, often of a sophisticated kind, and this invariably results in high substructure costs. As a material, therefore, soft clay warrants the state-of-the-art coverage given in this book to its engineering behaviour.

Soft clay is an especially interesting and rewarding material with which to work. It poses abundant engineering challenges whereby the designer must often use very low safety factors, and about which the decisions he takes can have large economic consequences for a project. There is also scope in the handling of soft clay problems for flair and innovation to an extent which is unusual in present day civil engineering. Further, it is precisely because these soils respond in a spectacular manner to stress changes that they offer the engineer-scientist special opportunities to evaluate his soil mechanics theories which are offered by no other earth material. This evaluation process has been particularly facilitated by a number of carefully planned full-scale field trials which have been carried out in recent years.

The origins of *Soft Clay Engineering* go back to 1977, when the Editors organized an International Symposium on Soft Clay in Bangkok, Thailand, which featured important state-of-the-art lectures delivered by individuals who were well-known for their work with soft clay. Eleven of these lectures, after revision and modification, have been transformed into chapters of the book. Collectively, these chapters are thought to provide a comprehensive coverage of the relevant subject matter. Individually, each chapter is intended to be as far as possible self-contained within its stated subject area. Some overlap between certain chapters is inevitable, but these small duplications of material will generally be found beneficial to the reader. It is felt that this book will be of particular value to practicing civil engineers and engineering geologists, but it will also be of considerable interest to postgraduate and senior undergraduate students in civil engineering, engineering geology and geology.

The main role of the Editors has been to decide on the overall subject matter of the book, as determined by the selection of the eleven chapter topics, and to unify the separate contributions into a common format and general style. The unification of language and style in a volume with twenty-two Authors of twelve nationalities has not been without its difficulties! In the process, some Authors have suffered in a most gracious and understanding manner certain linguistic and editorial indignities imposed on their manuscripts in the interests of uniformity, and the Editors wish to record their gratitude for the tolerance of the Authors throughout the book's long gestation period.

Acknowledgements are due to those who played some part in the realization of *Soft Clay Engineering*. The International Symposium on Soft Clay held in Bangkok in 1977 was sponsored by the Asian Institute of Technology and the Canadian International Development Agency, without whom the seeds of the book would never have been sewn. The Editors are also grateful for the painstaking way in which manuscripts were typed in Bangkok by Mrs Uraiwan Singchinsuk, and in Hong Kong by Miss Margaret Wong.

E.W. Brand
R.P. Brenner

July, 1981

Chapter 1

HISTORY OF CIVIL ENGINEERING IN SOFT CLAY

by

NILS FLODIN and BENGT BROMS

Nils Flodin began his career in 1937 as a field engineer under Walter Kjellman in what was then the Geotechnical Department of the Royal Board of Roads and Waterways, and which was reorganised into the Swedish Geotechnical Institute in 1944. He became Head of the Institute's Technical Secretariat in 1950. He remained in that post until 1975, when he moved to the Department of Soil and Rock Mechanics of the Royal Institute of Technology in Stockholm as Senior Research Engineer.

Mr Flodin was Assistant Secretary of the Swedish Geotechnical Society from 1956 to 1963 and has been the Secretary continuously from 1963. He has been involved in several committee activities of the Swedish Society and of the International Society for Soil Mechanics and Foundation Engineering.

Bengt Broms is Professor of Soil and Rock Mechanics at the Royal Institute of Technology in Stockholm. He graduated in civil engineering from Chalmers Technical University, Gothenburg, and then obtained MS and PhD degrees from the University of Illinois in the USA. From 1959 to 1964, he was Associate Professor of Civil Engineering at Cornell University, before he returned to Sweden as Director of the Swedish Geotechnical Institute, a post he held for ten years.

Professor Broms is the author of about 250 papers and books on reinforced concrete, rock mechanics, soil mechanics and foundation engineering. He has worked as a geotechnical consultant in more than 25 countries. During the period 1977 to 1981, he was Vice-President for Europe of the International Society for Soil Mechanics and Foundation Engineering.

Chapter 1

HISTORICAL DEVELOPMENT OF CIVIL ENGINEERING IN SOFT CLAY

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1.1 INTRODUCTION

The construction of buildings, roads, bridges, canals, harbours and railways in soft clays has always been associated with stability problems and settlements, and it is only gradually that man has been able to master, at least partly, some of these problems, mainly through experiences gained over many centuries. Still much remains to be done. The following historical review is only a brief account of this development. The history of geotechnical engineering is in fact a thrilling one, and those involved, particularly in the construction of railways and harbours during the industrial revolution, are worthy of our respect.

Soft clay is here defined as a clay with a shear strength less than 25 kPa, and this review covers mainly the period up to about 1948. This means that most of the recent excellent work in Norway, Canada, USSR, Mexico, Thailand, etc has to be saved for a future chapter in the history of geotechnical engineering.

The main regions with soft clay are the Nordic countries (except Denmark), Canada and northern United States (Chicago and Boston), where deposits of soft glacial and postglacial clays are often more than 100 m thick. This review is principally concerned with the soft clays in these regions, but other soft clays are also considered. Clays in these regions often have a high sensitivity and a low shear strength; they are called *quick* in the Scandinavian countries when the sensitivity ratio, S_t, (i.e. the ratio of the undisturbed and remoulded shear strengths) exceeds 50.

Other areas where deep deposits of soft clay occur are Mexico City, with its volcanic clay, and the deltas of the major rivers of the world, such as the Nile, Mississipi, Rhine around Rotterdam in the Netherlands, Elbe around Hamburg in Germany, Neva around Leningrad in USSR, Eufrates and Tigris Rivers in Iraq, Ganges around Calcutta, and the Yangtze River around Shanghai in China. The well-known Bangkok Clay and Boston Blue Clay should also not be forgotten.

This historical chapter is to a large extent concerned with the conditions in Sweden, where problems connected with construction in soft clay appeared

very early. Much of the early work on soft clay was done in Sweden during the first decades of this century. The investigations of the Swedish chemist Albert Atterberg (Fig. 1.1) on the consistency and classification of clays are well known, and the so-called *Atterberg Limit Tests* are today used all over the world. The work of the Geotechnical Commission of the Swedish State Railways under its chairman Wolmar Fellenius and, especially, its secretary John Olsson (Fig. 1.2) on the shear strength and settlement of soft clay was also of fundamental importance, as were the investigations of Knut E. Petterson and Sven Hultin on the stability of slopes and the Swedish slip circle method, and of Ernst Wendel on the bearing capacity of piles. The contributions of Walter Kjellman (Fig. 1.3) from the middle of the 1930s with respect to new field and laboratory test methods should also be mentioned.

Fig. 1.1 Albert Atterberg.

Fig. 1.2. John Olsson.

Fig. 1.3. Walter Kjellman.

Much of the early work on soft clays is, of course, connected with the name of Karl Terzaghi. A productive period in the history of soil mechanics is that of the 1930s when many distinguished international researchers worked together with Terzaghi in Vienna (see A. Casagrande, 1960). The results of this cooperation added much to the progress of geotechnical science and to its development in different countries. More recently, the works of A. Casagrande, A.W. Skempton, L. Bjerrum and others have also been of fundamental importance.

1.2 MAIN EVENTS AND CONTRIBUTIONS

1.2.1 Milestones in geotechnical engineering

A large part of the early theoretical works of such people as Coulomb in 1773 (1776) and Rankine in 1857 was concerned with the lateral earth pressure against retaining walls. In 1846, Collin presented his remarkable essay on the stability of slopes in clays, a work which only recently has been rediscovered. Several textbooks in civil engineering existed already in the 18th and 19th

centuries, and these reflect the state-of-the-art at that time with respect to construction in soft clay. These books were almost entirely based on practical experience. Examples of such textbooks are those by Carlberg (1792), Hagen (1841), Broch (1848), Stål (1854), Kolderup (1894), Strukel (1895), Willmann (1906) and H. Olsson (1915).

One turning point in modern soil mechanics was around 1913 when a number of landslides in different parts of the world indicated that the methods of analysis as they existed at that time were not adequate. In Sweden, a geotechnical commission was appointed by the Swedish State Railways to investigate the cause of a number of landslides that had recently occurred. In the United States, a special committee was set up within the American Society of Civil Engineers because the slope of one of the deep cuts in the Panama Canal failed and threatened to close the canal permanently (Binger, 1948). In Germany, a number of slides occurred during the construction of the Kiel Canal which prompted Karl Krey to initiate an extensive investigation into the lateral earth pressures on retaining walls and bulkheads.

Modern soil mechanics is intimately connected with Karl Terzaghi who, with his book *Erdbaumechanik auf bodenphysikalischer Grundlage* (1925a), initiated research in many parts of the world.

Of great importance for the development of modern geotechnical engineering has also been the International Conferences. The first conference was held in Cambridge, Massachusetts, in 1936 with Arthur Casagrande as secretary general and organizer. Terzaghi (1943) wrote that the proceedings from this conference alone contained a greater amount of quantitative information regarding soils and foundations than the entire literature up to 1910. Of importance for the development of geotechnical engineering, particularly in the United States, was the Purdue Conference on Soil Mechanics and Its Application which was held in 1940. The International Society for Soil Mechanics and Foundation Engineering was started in 1948 with Terzaghi as its president and D.W. Taylor as the secretary general. The international geotechnical journal *Géotechnique* was initiated in 1948 by the British Geotechnical Society.

The end of this first period of modern soil mechanics saw the publication in 1948–51 of the four textbooks:

(1) *Soil Mechanics in Engineering Practice,* by K. Terzaghi & R.B. Peck (1948).

(2) *Fundamentals of Soil Mechanics,* by D.W. Taylor (1948a).

(3) *Subsurface Exploration and Sampling of Soils for Civil Engineering Purposes,* by M.J. Hvorslev (1949).

(4) *Soil Mechanics, Foundations and Earth Structures,* by G.P. Tschebotarioff (1951).

The contributions of A.W. Skempton in 1948 should also be mentioned in this connection, since he presented no less than seven important papers that year.

Some general notes on Russian contributions to the history of soil mechanics, with special reference to soft clay, are to be found in the interesting paper by Tsytovich & Zaretsky (1969) on the theory of soil consolidation in the USSR. Gersevanov (1931, 1934, 1937), who co-operated with Terzaghi, is regarded as one of the founders of the Soviet school of soil mechanics. The work of Tsytovich (1934, 1940, etc), Maslov (1936, 1949) and Florin (1938, 1948) should also be mentioned. An interesting historical review of soil mechanics in the USSR was made by Ter-Stepanian (1963).

Of particular interest from a general historical point of view is the paper by Glossop (1968) with the title *The Rise of Geotechnology and Its Influence on Engineering Practice.* Also Feld (1948) and Verdeyen (1959) published reports on the history of soil mechanics. The book by Legget (1973), *Cities and Geology,* is of great interest in this connection, not least with regard to foundation problems in soft clay. The paper by Bjerrum & Flodin (1960) on the history of soil mechanics in Sweden 1900–1925 should also be mentioned. A further excellent source for historical studies in geotechnical engineering is *A Century of Soil Mechanics,* published in 1969 by the Institution of Civil Engineers in London, which contains 16 classical British papers from 1844 to 1946 and a bibliography for the period 1836–1948. Similar works, entitled *Contributions to Soil Mechanics* were issued by the Boston Society of Civil Engineers in three volumes in 1940, 1953 and 1965, and contained 43 valuable papers from 1925 to 1962.

1.2.2 Early committees and organizations

The first geotechnical committee concerned particularly with soft clay was appointed in Norway as early as 1726, some decades after a large landslide at Sarpsfossen on the river Glomma. This commission consisted of two generals, another officer of senior rank *(quartermaster general)* and an engineer captain (Nygard, 1944). Another early commission, also in Norway, was appointed in 1838 with the task to investigate the cause of slope failures in Drammen and to prevent future slides. A third Norwegian committee, the so-called Large Engineering Committee, was appointed in 1864 with representatives from different authorities in the civil engineering field: its activities have been described in Polyteknisk Tidskrift (1866). A geologist was later added to the committee to assist with the interpretation of the results from penetration tests (G. Holmsen, 1955).

The so-called Bohus Committee was appointed in Sweden in 1908 because of the many severe landslides which had occurred along the main railroad along the west coast of Sweden (Bjerrum & Flodin, 1960). The committee concluded that it was in many cases not possible to predict the bearing capacity of soft ground or to tackle the stability problem technically. Professor

G. De Geer, chairman of the committee, compared the behaviour of a deposit of very soft clay, with its often thin, dry crust, with that of a lake covered by a thin ice sheet. The saturated clay was so soft, according to De Geer, that it ought to have been called "water with clay particles".

The Bohus Committee is regarded as the forerunner of the Geotechnical Commission of the Swedish State Railways, which was appointed because of repeated slides along the main railroad lines in Sweden. This committee was convened in December 1913 and worked between 1914 and 1920, most of its work being done during World War I. Its *Final Report* (Swedish Geotechnical Commission, 1922) is regarded as one of the milestones of modern soil mechanics. The term *geotechnics* (Swedish *geoteknik*) was coined by the Commission. Its recommendations were adopted almost immediately in Sweden, Finland and Norway (see also Section 1.7.2.3).

A geotechnical commission patterned after the Swedish Commission was appointed by the Finnish State Railways in 1919 after a landslide at Hoplax, west of Helsinki. The Secretary of the Finnish Geotechnical Commission was the geologist Thord Brenner; he is probably best known for his investigations of the physical properties of soils.

Besides the Swedish Geotechnical Commission, a special Harbour Committee was set up in Gothenburg in Sweden in 1916 because of the failure of the Stigberg Quay. The analysis of this slide resulted in the Swedish Slip Circle Method with Sven Hultin, Knut E. Petterson and, later, Wolmar Fellenius (Fig. 1.4) as its originators (S. Hultin, 1916; Petterson, 1916; W. Fellenius, 1918, 1926, 1927).

Fig. 1.4. The fathers of the Swedish Slip Circle Method. From left, Knut E. Petterson, who established the circular shape of the slip surface, Sven Hultin, who developed the analytical method, and Wolmar Fellenius, who extended the method to soils with both cohesion and internal friction.

A geotechnical governmental committee was also appointed in the Netherlands in 1920 following the disastrous failure of an embankment at Weesp;

Professor A.S. Keverling Buisman was one of the members of the committee (Glossop, 1968).

A geotechnical laboratory was set up within the Swedish State Railways after the Swedish Geotechnical Commission had finished its work in 1920 (with John Olsson as its chief engineer). This laboratory was probably the first of its kind in the world. A geotechnical department was organized at the Norwegian State Railways at about the same time (1922) with A.L. Rosenlund and S. Skaven Haug as heads, and the Finnish Geotechnical Committee was reorganized in 1926 into the Geotechnical Department of the Finnish State Railways under Thord Brenner.

In the Netherlands, the Laboratorium voor Grondmechanica in Delft was established in 1934 with Professor Buisman as Director (van Mourik Broekman, 1948). The Geotechnical Department of the National Swedish Road Authority was organized in 1936 under Walter Kjellman, and this was reorganized as the Swedish Geotechnical Institute in 1944. Its brother organization in Norway, the Norwegian Geotechnical Institute, was set up in 1953 with Laurits Bjerrum as Director. The National Geotechnical Laboratory of Finland was organized in the same year as part of the Technical Research Centre and was headed by K.V. Helenelund. In the USA, special committees to deal mainly with soils related to highways existed as early as the 1930s: the ASTM Committee on Soils for Engineering purposes was established in 1936.

1.3 AREAS WITH SOFT CLAY

A very brief account is given below of the geological conditions of regions with soft clays. For details of the geological conditions in the Nordic countries, the reader is referred to the *Proceedings of the European Symposium on Penetration Testing (ESOPT)* held in Stockholm in 1974.

The geological conditions during and after the last glacial period have had a decisive influence on the soil conditions in the Nordic countries as well as in Canada and in the northern United States. Of special importance in Finland, Norway, Sweden and Canada has been the isostatic uplift (land upheaval) which began about 10,000 years ago when the continental ice sheet started to melt. The depression of the land caused by the ice was several hundred metres in Scandinavia. The land upheaval, of up to nearly 200 m so far, is still going on; this is about 5 mm/year in Stockholm and about 10 mm/year in northern Sweden. This land upheavel has affected the stability of the slopes along the coast and the major rivers below the highest coastline. It has also contributed to an apparent lowering of the ground water level in many parts of the Nordic countries, e.g. Stockholm.

The glacial clays are often varved, while the younger post-glacial clays are generally uniform with a frequently high organic content; strata of silt and sand frequently occur. The soft clays are as a rule normally consolidated or

slightly overconsolidated. The sensitivity is normally between 10 and 15, which is often regarded as high for countries outside these regions. Quick clays, mainly caused by leaching of salt, are often found in the form of layers or lenses. The upper part of the soft clay, the so-called dry crust, with a thickness of generally 1 to 2 m, is usually weathered. In the natural slopes of the Leda Clay in Canada (see below) a weathered crust of 10 m is often found (Eden, 1975).

The term *quick clay* can be traced to the first half of the 18th century. It was used in Norway as early as 1736 (Senneset, 1974). The Norwegian Geotechnical Commission of 1726, however, used the old Norwegian term *qvegleer* which is similar to *kvikkleire, quick clay* (Jørstad, 1981); this is the first time that the term *quick clay* appears to have been used. A description of quick clay is found in a Swedish calendar of 1767. In this book, as well as in modern dictionaries, the terms *quick clay* and *quick sand* have practically the same definitions: "A soil consisting of a fine sandy clay which when saturated forms a dough-like, almost flowing mass; flow-clay; fermenting clay" (*Sv.Akad.Ordbok,* Vol. 15, 1939). This old definition was also used by Reusch (1901) who was the first to translate the Scandinavian word *kvicklera* into the English term *quick clay*. Other strange definitions can be found (e.g. Kolderup, 1894, p. 11). It was after 1923, when 'undisturbed' samples could be taken, that the term was first used in its present sense (Terzaghi & Peck, 1948).

In *Finland,* soft clays are mainly found in the southern and south-western parts of the country. The average thickness is about 10 m but, in valleys and along the coast, up to 50 m thick layers are common. Soft clay deposits of up to 70 m (in the Åbo area) and up to more than 100 m (in the Salo area), however, have also been found (Juhola, 1977). The shear strength is often 10 to 30 kPa. The sensitivity, S_t, is generally about 10, but values exceeding 50 have been reported (cf. the soft clay of eastern Sweden as described below and in Fig. 1.6). Reference should also be made to Korhonen & Leskela (1968) and Gardemeister (1975).

In *Norway,* nearly the whole population lives along the coast and fiords in the central and northern parts of the country, and in the midlands and valley of the southeastern part. These areas represent about one quarter of the total area of Norway, and they are also the regions where the country's geotechnical problems are concentrated. The soft clay deposits are up to 100 m thick and the 2 m to 5 m thick dry surface crust is generally thicker than in Finland or Sweden (G. Holmsen, 1938). The clays are invariably of a high sensitivity, and the number of landslides has been very large over the years. A typical profile at Drammen is shown in Fig. 1.5 (Lunne et al, 1976).

In *Sweden,* the soft clays in the eastern parts of the country around Stockholm and Lake Mälaren are frequently varved. These clays, which were deposited mainly in fresh or brackish water, are similar to those in Finland. They are more fine-grained than the clays in Norway. The shear strength is normally between 10 and 30 kPa, and the sensitivity is between 10 and 20.

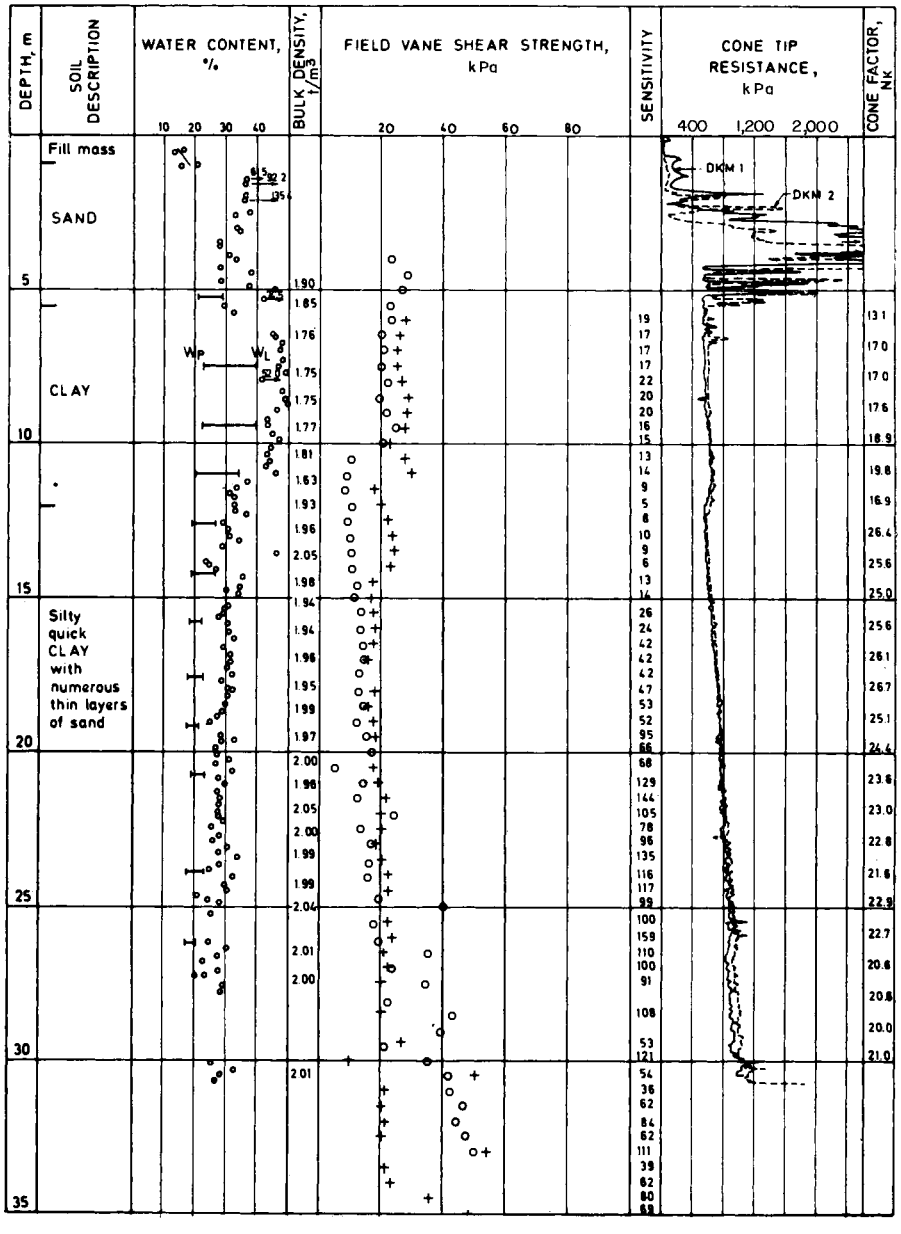

Fig. 1.5. Soil profile from Drammen, Norway (Lunne et al, 1976).

The depth is generally about 10 m, but deposits of up to 30 m depth occur. Quick clays have been found also in these regions. A lowering of the ground water level by tunnels and other factors has recently caused large settlements and extensive damage on even small, single-family houses in Stockholm. A profile at Skå-Edeby near Stockholm is shown in Fig. 1.6.

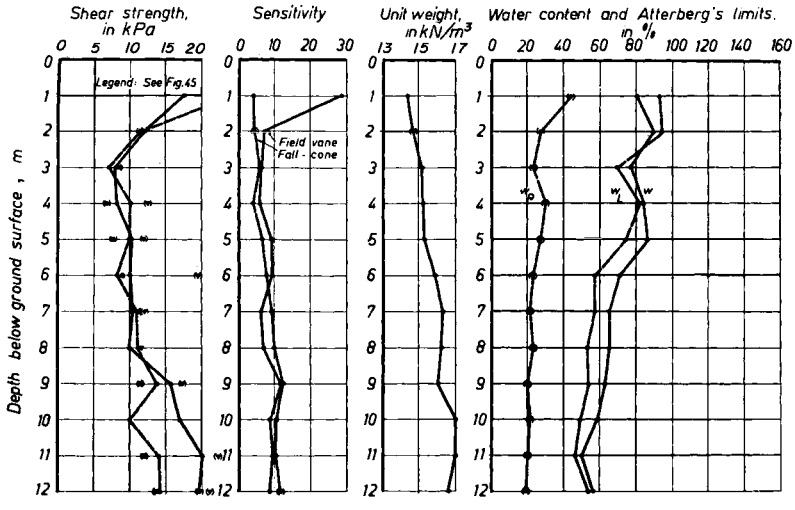

Fig. 1.6. Soil profile from Skå-Edeby, eastern Sweden (Hansbo, 1960).

Soft clays which have been deposited in salt water are common along the west coast of Sweden, including Gothenburg and the Göta River Valley. This valley is especially known for its many landslides (Frödin, 1919). The depth of the clay deposits exceeds 100 m in several places in Gothenburg, which became very early a 'geotechnical town'. The clay has here a shear strength of about 20 to 60 kPa and a sensitivity of about 10. Quick clays are common in the Göta River Valley; in the Lidan River region, northeast of the Göta River Valley, sensitivities of about 200 have been found (Odenstad, 1951). Considerably higher values of S_t have been found in Sweden (e.g. Söderblom, 1969). A clay profile at Gothemburg is shown in Fig. 1.7.

In *Canada,* soft marine clays, which are often quick, occur in the eastern part of the country. These clays, which are sometimes covered with a mantle of sand, were deposited during the late Pleistocene Period in the so-called Champlain Sea (Fig. 1.8a). This clay is perhaps best known under the name *Leda Clay,* given to it by the Canadian Geologist J.W. Dawson (Logan, 1863) because of the fossils it contained. In recent times, however, this name has been questioned, since parts of the deposit do not contain Yoldia (Leda) fossils (Gadd, 1960). The name *Champlain Clay,* suggested by Gadd, might thus be more appropriate (LaRochelle et al, 1970).

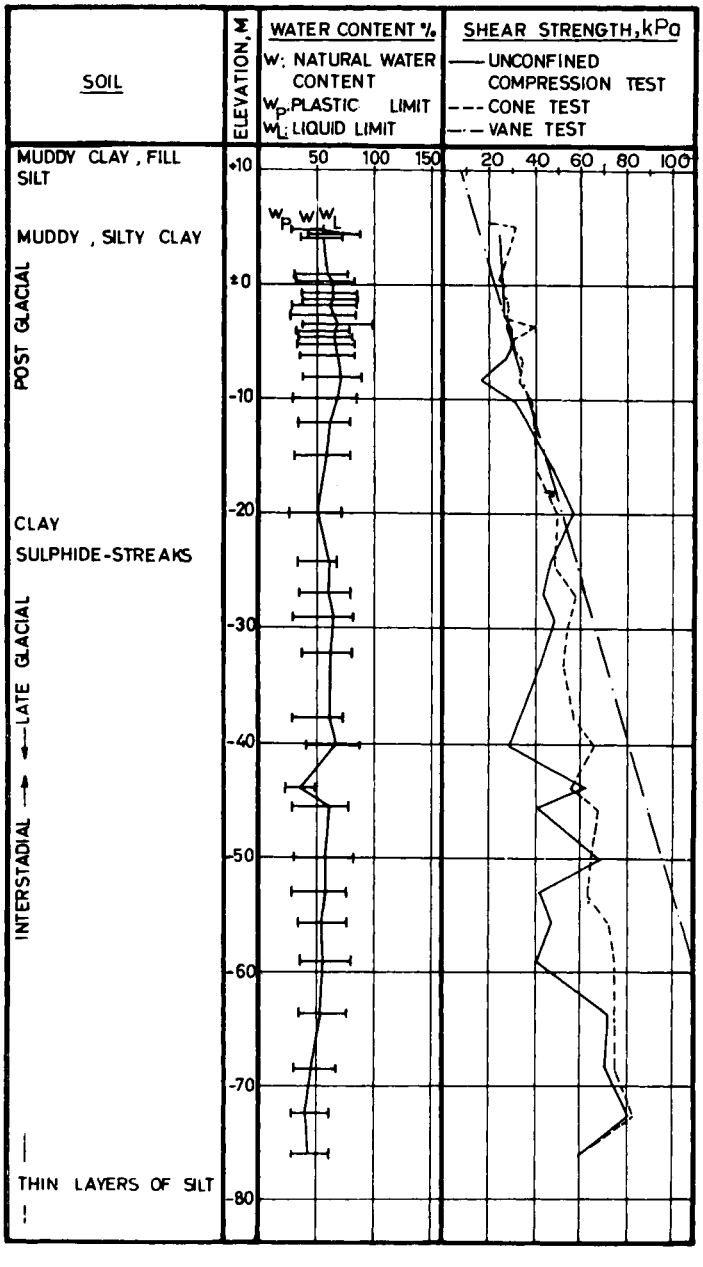

Fig. 1.7. Soil profile from Gothenburg, Sweden (Brink, 1967).

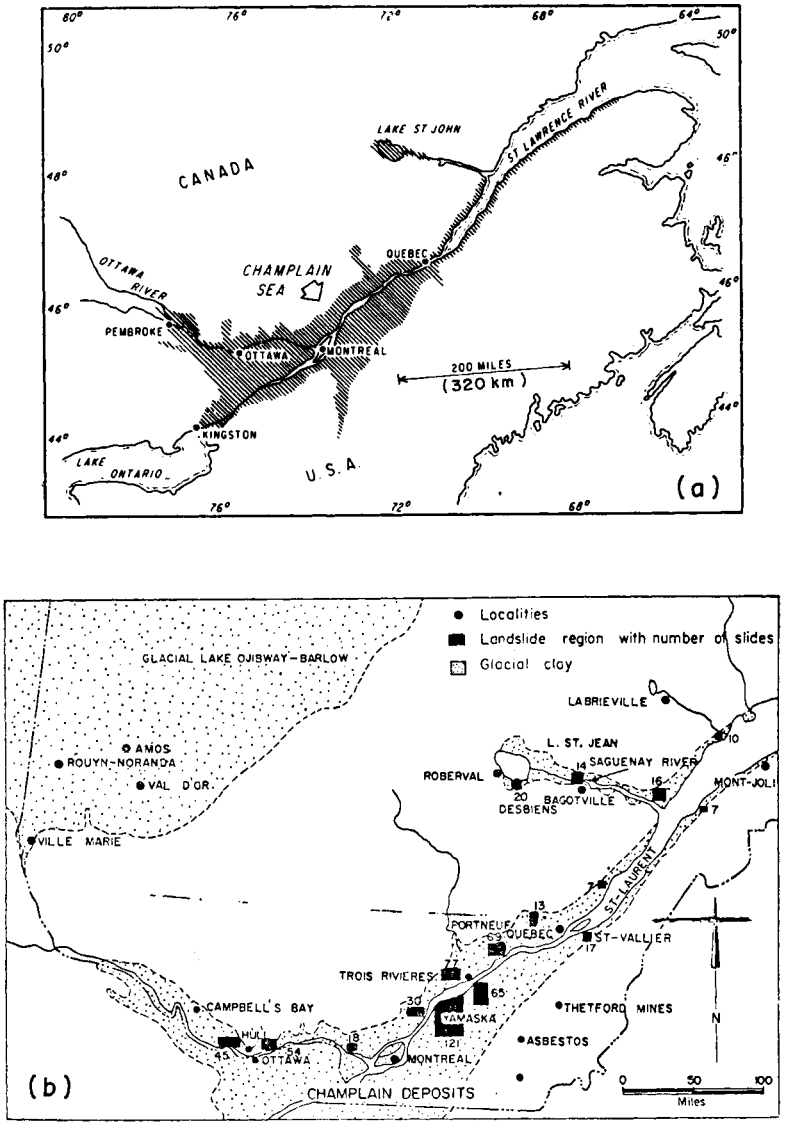

Fig. 1.8. The Champlain Sea, Canada: (a) location and extent of the Champlain Sea (Crawford, 1968), (b) distribution of glacial clay and landslides in Quebec Province (Chagnon, 1968).

The geology of the Champlain Sea region has been described by Gadd (1957, 1960), Karrow (1961), LaSalle (1966), LaRochelle et al (1970) and others. Radiocarbon dating, palaeontological studies and detailed field explorations have established the Champlain Sea as late glacial rather than postglacial. The marine period is known to have been influenced greatly by the inflow of fresh water creating brakish and even fresh water conditions (Gadd, 1960). The clay size content, plasticity and natural water content increase with increasing distance from the ocean. The clay is slightly overconsolidated and contains substantial amounts of illite and chlorite with lesser amounts of inert materials such as quartz and feldspar. It is frequently stratified with thin layers of silt and fine sand, like the Scandinavian clays. Further, the clay commonly exhibits a black mottling in fresh samples from below the water-table, caused by anaerobic bacteria; this mottling disappears on exposure to air. There seems to be no mottling in areas of high salt concentration (Crawford, 1961). Attention has been drawn to the banding due to fossils which could readily be mistaken for glacial varves (Gadd, 1960).

Crawford and others have reported that the engineering geological properties of the Leda Clay vary widely. The undrained shear strength varies from 30 to 150 kPa, and the sensitivity from 10 to 100, but exceptional values of up to 175 have been recorded (Mitchell & Markell, 1974). Also, w_n=40 to 80%, w_L=20 to 70% and w_P=17 to 30%. The salt content varies dramatically over short distances. The activity of the clay is relatively low in all regions.

Landslides of the flow type are very common in the region, particularly along the St Lawrence and Ottawa Rivers. The many landslides recorded so far in the various unstable areas are indicated in Fig. 1.8b.

It should be mentioned that geotechnical engineering in the modern sense post-dated the end of World War II in Canada (Legget & LaSalle, 1978). Before that time, only rough field investigation methods were used (wash borings, standard penetration tests, etc.) which were not very suitable for the soft, very sensitive Leda Clay. The quick clay problems were, of course, encountered early in practice (see above), but these were not tackled rationally from an engineering point of view. After about 1950, the way in which the soft clay problems were handled changed radically, and today Canada is in the forefront of knowledge in soft clay mechanics, especially in regard to the landslide problems which have given eastern Canada so much trouble for so long.

The *Mexico City Clay,* of volcanic origin, contains several sand layers. The water content is very high, normally between 200 and 300%, and practically independent of depth. A water content of 400% has been measured at a 100 m depth (Terzaghi, 1953). For a more detailed description of the Mexico City Clay, reference should be made to the report by Marsal & Mazari (1959).

The *Bangkok Clay,* described by Eide (1968), Moh et al (1969) and Holmberg (1977), is very uniform and is intersected by fine cracks. It has a high water content and a high plasticity index like the Gothenburg Clay. The sensitivity is generally low. The clay has a low permeability and is very compressible,

with large secondary consolidation. It is geologically normally consolidated but behaves as a slightly overconsolidated clay. Down to about 15 m depth, the shear strength of the clay varies between about 20 and 40 kPa. Below this, it is stiff with a shear strength of about 300 kPa and a water content of 25 to 30%.

Only brief mention has been made in this chapter of the role of the USSR in the history of soft clay, mainly because of linguistic problems. A report dealing with soft clay in the USSR was presented in 1965 at the All-Union Conference on the Problems of Construction on Saturated Weak Clay Soils. A recent book by Abelev (1973), translated into French by J.P. Magnan in 1977 and dealing with structures on soft saturated clayey soils, is also of interest in this connection (see also Ter-Stepanian, 1963; Tsytovich, 1976).

A bibliography on sensitive clays has fairly recently been published in Canada by Thurston (1976).

1.4 BASIC PROPERTIES OF SOFT CLAY

Many of the basic studies on physico-chemical properties, consistency, deformation properties, shear strength and permeability of soft clay are, of course, applicable to other soils. It is necessary, therefore, to review here also works which are relatively general.

1.4.1 Consistency and physico-chemical properties

The physico-chemical properties are of fundamental importance to the understanding of the behaviour of soft clay. Of particular significance have been the studies of water in its different forms and the forces acting between soil particles. Terzaghi (1925a) pointed out, for example, that the absorbed water is solid at the surface of a particle and that its density is far above normal; the density and the viscosity of the water decrease with increasing distance from the particle. Terzaghi recognized that this layer was not affected by a hydraulic gradient. Studies of the physico-chemical properties of clays have to a great extent been carried out by chemists, physicists and soil scientists, including those in agriculture, ceramics, etc (see Goldschmidt, 1926; Childs & Youngs, 1974). To keep this review within reasonable limits, only investigations from the first half of this century will be discussed.

Among the most important contributions from Scandinavia are those by the Swedish chemist and agricultural researcher Albert Atterberg. He started his basic investigations on the consistency of soft clays and their classification in 1896. He studied how the consistency of a soil changed with the water content and defined certain characteristic properties called *consistency limits,* i.e. the liquid, plastic and shrinkage limits. The results were first presented in 1910 and in a more complete form in several later papers (Atterberg, 1910, 1911, 1916, etc). Terzaghi (1925a) recognized very early the importance of

Atterberg's work, and the Atterberg consistency limits are used today all over the world. The Swedish Geotechnical Commission (1922) found, however, that Atterberg's method to determine the liquid limit was too subjective and recommended the fall-cone method instead. The so-called *fineness number,* or *fall-cone liquid limit* as it is called today, was considered to be less affected by human factors than the test proposed by Atterberg. The fall-cone liquid limit corresponds to the water content of the remoulded soil when the penetration of the standard cone, 60 g and 60°, is 10 mm (SGF Laboratory Committee, 1977). A. Casagrande (1932a) improved the method originally proposed by Atterberg in order to reduce the subjectiveness of the liquid limit test, and this is today called the *percussion liquid limit* method.

Atterberg's work was continued by his assistant, the geologist Simon Johansson, who presented in 1914 an article on the strength of soils at different water contents, and in 1924 he wrote a paper on the viscosity and elasticity of clays. Another Swedish contribution of interest from about the same time is that by Ekström (1927) who investigated the role of water in clays. Of great interest also are the studies by Brenner (Fig. 1.9) on the physical properties of soils (1931, 1946) and the shear strength of soils (1946). These investigations are not well known outside of Scandinavia. Brenner suggested that a critical hydraulic gradient existed in clay below which there is no flow of water, and Silfverberg (1949) proposed the existence of a *threshold value* and studied the electro-chemical treatment of clay and electro-osmosis in the laboratory.

Fig. 1.9. Thord Brenner of Finland (left) and Ivan Th. Rosenqvist of Norway.

The investigations of quick clays by the Norwegian geologist Rosenqvist (1946a) (Fig. 1.9) are well known. The formation of quick clay by the leaching of salt from marine clays (the leaching theory), originally suggested by the Norwegian geologist Gunnar Holmsen in the 1930s (G. Holmsen, 1930; Odenstad, 1951), is closely connected with Rosenqvist's name (Rosenqvist,

1946a, 1953, 1955). The importance of salt in the consistency of clays was recognized by the Norwegian engineer Grennes as early as 1928 (see also Goldschmidt, 1926). The properties of quick clay were later studied by many researchers. The Swedish chemist Söderblom (1969, 1974a) found that the leaching of salt is not the only process that causes quick clays, that various dispersants also play an important role, and that clay deposited in fresh or brackish water can be quick as well. Rosenqvist (1978) is in agreement with this.

Among Rosenqvist's other contributions, his classification system for sensitive clays should be mentioned (Rosenqvist, 1953); this system was an extension of a proposal by Skempton & Northey (1952). A classification system based on the energy required to remould a sample of quick clay has recently been suggested by Söderblom (1974b). Rosenqvist (1942) also started early investigations into clay petrography and the plasticity of clays (1946b) as well as investigating the clay-electrolyte-water system (1955).

Probably the first one to point out the importance of soil structure on the physical properties of soft clays was A. Casagrande (1932b) (see also Lambe, 1953; Lambe & Martin, 1953). The concept of thixotropy was investigated by Freundlich in 1928 and Ackermann in 1948, although the regain of shear strength after remoulding had been recognized much earlier by those involved in pile driving (e.g. Wendel, 1900). Terzaghi investigated in 1926 the mechanics of absorption and swelling of gels. Seiffert et al (1935), among others, also studied the problem of sorption of ions and water in clay. The rheology of soils, including deformation, flow, consistency, etc, was treated by Scott Blair (1944) and Reiner (1949).

The many outstanding fundamental contributions in the area of clay mineralogy by Grim and, among others, Bradley in the 1930s should not be forgotten, but these deserve a separate chapter.

1.4.2 Consolidation and settlement

1.4.2.1 The phenomenon of consolidation

The often large settlements of structures and embankments founded on soft clay could not be explained at the beginning of the century. It was observed by railway builders, for example, that embankments sank without any heaving of the ground around the embankments. At that time, it was generally believed in the Nordic countries that it was not possible to squeeze out water from clay and that settlements were caused by creep.

In 1918 Åke Virgin, a Swedish surveyor, found when checking the elevation of bench marks that some of them had settled considerably, especially where the soft clay was very thick. He suggested that these settlements were due to a reduction of the water content and the clay particles becoming closer together. He also suggested that the settlement rate decreased with time. The observations by Virgin prompted John Olsson, then secretary of the Swedish Geotechnical Commission, to construct a simple laboratory consolidation

device, a kind of oedometer, as illustrated in Fig. 1.10. Olsson found that water was squeezed out of the soft clay specimen when it was loaded and that the volume was reduced. This was about the same time that Terzaghi started his consolidation studies which culminated in his consolidation theory in 1923/1924.

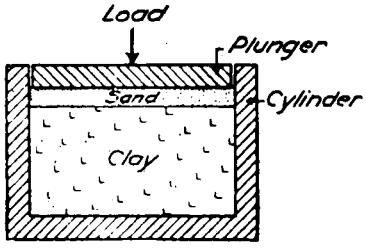

Fig. 1.10. Consolidation device produced in 1919 by John Olsson of the Swedish Geotechnical Commission.

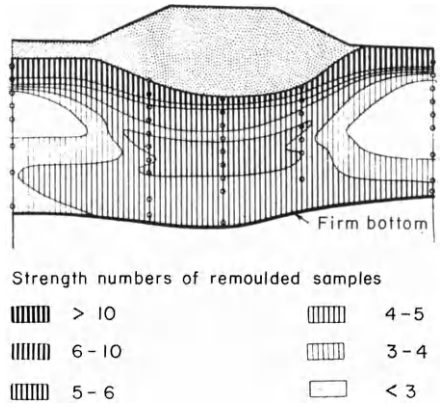

Fig. 1.11. Shear strengths (strength numbers) of remoulded samples below an embankment at Horsmossen, Sweden (Swedish Geotechnical Commission, 1922).

With the discoveries by Virgin and Olsson in mind, the Swedish Geotechnical Commission studied some old railway embankments that had settled. Samples were taken below and beside the embankments. It was found that the water content was the lowest and the shear strength the highest below the centre of a 1.5 m high embankment which had settled 1.7 m, as illustrated in Fig. 1.11. It was concluded that the settlements were caused by a reduction of the water content under the weight of the fill.

The discovery of the consolidation phenomenon in Sweden in connection with soft clay is an example of how the same theory can appear at different times and under different conditions and be unknown throughout the rest of the world (see below). In his excellent paper *Terzaghi's Discovery of Effective Stress,* Skempton (1960) mentioned that the first practicing engineer who was aware of the consolidation process seems to have been Telford in 1809 (Telford, 1830). He preloaded a 17 m thick bed of soft clay at foundation level of a sea lock on the Caledonian Canal by building an embankment and allowing it to settle for about nine months "for the purpose of squeezing out the water and consolidating the mud" before the construction of the lock. Skempton also mentioned that the English geologist Lyell (1871) found that water was pressed out of clay under load causing "compaction and solidification" of the material. W.S. Smith (1892) discovered for buildings founded on the Chicago clay that "slow progressive settlements result from squeezing out of water from the earth". Shankland (1896) agreed with Smith's views, and

he published some time-settlement curves for the Masonic Temple in Chicago covering a period of four and a half years. Later, Sorby (1908) tried to assess quantitatively the volume changes of clays due to loading.

The first to prove experimentally the consolidation process was probably Frontard in 1910 (Frontard, 1914). He investigated a specimen 50 mm thick and 350 mm in diameter in a consolidometer consisting of a metal container with a perforated base; the specimen was loaded through a piston. The sample was loaded in increments. The results were plotted on a graph relating water content and pressure. Frontard noticed that considerable time was required for complete consolidation and that five days was the minimum. Forchheimer (1914) treated this problem theoretically based on the assumption that the compression of a clay layer was equal to the volume of water expelled during the consolidation. He also derived an expression for the time required for a given settlement to take place. No further progress was made until Terzaghi presented his consolidation theory in 1923.

In connection with an investigation in Oslo into the cause of the settlements of buildings around the Holmenkollen subway, John Olsson was invited to give a talk about the work of the Swedish Geotechnical Commission (J. Olsson, 1923). He then demonstrated his "consolidometer" (Fig. 1.10) and attributed the settlements to consolidation of the clay. His explanation was much discussed and was not accepted by all (cf. Bjørlykke, 1914–1928). Many doubted that the explanation by Olsson could be valid for the Norwegian clays.

1.4.2.2 Terzaghi's consolidation theory

The concept of effective stress, σ', as expressed for saturated soils by the equation:

$$\sigma' = \sigma - u \tag{1.1}$$

where σ is the total stress and u is the pore water pressure, was first formulated by Terzaghi in 1923. This concept is simple in principle, but its fundamental importance for the consolidation and shear strength of soils was not immediately recognized by the geotechnical community in general.

Terzaghi presented in this paper the basic differential equations governing the consolidation process. With these equations it was possible for the first time to calculate from laboratory tests the consolidation rate. Terzaghi presented his consolidation theory also at the International Congress for Applied Mechanics in Delft, Holland in 1924 where it found wide acceptance. It should be mentioned, however, that everybody did not accept the new theory. In 1937, a colleague of Terzaghi in Vienna criticized it severely. He wrongly claimed that there was a mistake in the derivation of Terzaghi's basic equations (Tschebotarioff, 1976).

Terzaghi (1925a) pointed out the analogy between the consolidation of soils and heat transfer, in which the coefficient of permeability corresponds to the

coefficient of heat conductivity and the hydraulic gradient to the temperature gradient. Different boundary condition cases were investigated by Terzaghi & Fröhlich (1936) and by Fröhlich (1936).

The consolidation theory received much attention in the USSR, as described in a well documented paper by Tsytovich & Zaretsky (1969), and its practical application resulted in further development of the theory. The basic work commenced with Gersevanov (1931), when he presented the solution to a large number of practical problems; apparently, he was also the first to change the traditional formulation of the law of liquid filtration through a porous skeleton and to investigate the one-dimensional problem in detail. He drew attention to the substantial influence of the gas saturation of soil on the consolidation process, and he considered the soil swelling problem. Later, Bulichev (1948) and Florin (1948) considered in detail the influence of the presence of gas on soil consolidation. Others who pioneered work with the one-dimensional consolidation theory were Popov (1934), Gersevanoff (1936), Roza (1937), Korotkin (1938) and Tsytovich (1934, 1940).

Florin (1948) generalized the one-dimensional soil consolidation theory for the three-dimensional case using a soil model called "the basic model", which has been much used in the USSR; this model was developed independently of the similar model used by Biot (1941). The system of equations for three-dimensional soil consolidation was further developed by Frankel (1944). The need to account for volumetric forces in considering the consolidation theory was first voiced by Gersevanov (1931) and was further developed by Florin (1938), who worked with a "volumetric forces model". Gersevanov & Polshin (1948) later used this model when solving three-dimensional consolidation problems.

1.4.2.3 Compressibility

An apparatus for carrying out consolidation tests (oedometer) was constructed by Terzaghi in 1919 when he was working at the Robert College in Istanbul (Terzaghi, 1924). The consolidation of clays was further studied by Rendulic (1935a) using a triaxial apparatus constructed at the Technical University in Vienna when he worked with Terzaghi. Rendulic observed that the agreement between measured and calculated consolidation rates was better for triaxial than for oedometer tests. This was attributed to the hydrostatic state of stress in the triaxial tests.

A. Casagrande (1932b) observed from oedometer tests large differences in compressibility between undisturbed and disturbed soil samples. He pointed out that the coagulating effect of the salt in sea water is so strong that the clay particles, and even silt and small sand grains, form flocks and that these flocks usually remain in the position they had when they first touched the bottom, as shown in Fig. 1.12. He pointed out that larger deformations occur in the space between the silt particles than in the space between the clay particles;

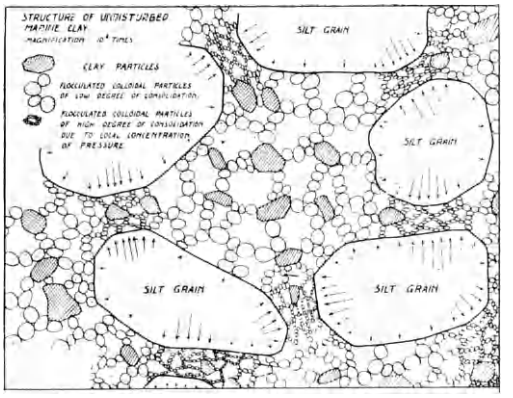

Fig. 1.12. Structure of clay (A. Casagrande, 1932b).

the clay between the silt particles will only carry a small fraction of the pressure and will remain very soft. Casagrande also observed that the consolidation curve for carefully taken samples had a sharp break at the preconsolidation pressure. For disturbed samples, the break occurred at a pressure which was less than the preconsolidation pressure.

An important contribution was also the graphical procedure proposed by Casagrande (1936) to evaluate the preconsolidation pressure from oedometer tests. This method is today used all over the world. Rutledge (1944) found a good correlation for the Chicago clay between the preconsolidation pressure determined by Casagrande's graphical method and the effective overburden pressure. Skempton (1948f), also, found for the Gosport clay a good agreement with Casagrande's method. Casagrande also proposed a graphical procedure to determine the point on the e—log t curve which corresponds to 100% consolidation. This point is used to calculate the coefficient of consolidation: in Taylor's method, the e—$\sqrt{t}$ relationship is used for the same purpose (Taylor, 1942).

The direct shear apparatus developed in 1936 by Kjellman (1950), and shown later in Fig. 1.14, was also intended for consolidation and permeability tests. To eliminate the friction between the rigid wall and the specimen, which occurs in the oedometer, the specimen in Kjellman's apparatus was confined laterally by a rubber hose and outside the hose by a series of rings. In a test where large compressions were foreseen, the space between the rings were made correspondingly large. In this case, the rubber hose bulged slightly between the rings causing some 'initial' consolidation and also some resistance to the consolidation.

Skempton (1944) studied the relationship between the compression index, C_c, and the liquid limit, w_L, and proposed an empirical relationship for normally consolidated clays with medium to low sensitivity as:

$$C_c = 0.007\ (w_L - 10\%) \tag{1.2}$$

Terzaghi & Peck (1948) pointed out, however, that the compressibility can be several times the value given by this expression if the liquid limit exceeds 100% or if the natural water content at a depth of 7 to 10 m is greater than the liquid limit.

The first to investigate with triaxial tests the relationship between void ratio and the applied principal stresses (σ_1 and σ_3) was Rendulic (1937). A study of Vienna Clay *(Wiener Tegel)* showed the stress ratio σ_1/σ_3 had an important effect on the void ratio and thus on the water content, as illustrated in Fig. 1.13.

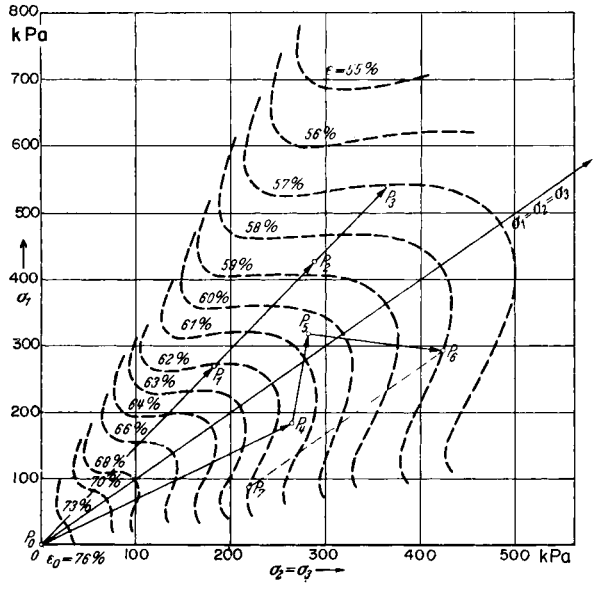

Fig. 1.13. Relationship between porosity and principal stresses (Rendulic, 1937).

Terzaghi pointed out in 1941 that the actual settlement for soft clay is often considerably smaller than that computed when the load increase is small. He indicated that a critical depth exists below which the slope of the real compression curve is insignificant and that the rigid bonds between the clay particles break when a clay is loaded beyond a certain limit. This is the first time, as far as the authors know, that the concept of a *quasi-consolidation* load was suggested.

Secondary consolidation has been investigated by many people, including Buisman (1936). He was the first to propose that the settlement caused by secondary consolidation increases with log t, where t is the time. The effect of temperature and other factors on the rate of secondary consolidation was pointed out by Gray (1936). Later, Haefeli & Schaad (1948) suggested that the settlement—log p relationship, including secondary consolidation, is

linear. This concept was later used by Bjerrum to explain the 'preloading' caused by secondary consolidation. Crawford (1961) discussed the differences between values of secondary compression obtained from pore pressure measurements in laboratory tests and those which exist in the field; he stressed the difficulty of determining the preconsolidaton pressure and suggested that accurate deformation properties of soils could only be obtained by means of combined laboratory and field tests.

1.4.2.4 Settlement

The initial settlement of a footing on soft clay is generally calculated from the theory of elasticity assuming a constant modulus of elasticity, E, and a constant Poisson's ratio, υ, of the soil. For a square rigid footing:

$$\delta_i = 0.95\,(1 - \upsilon^2)\,\frac{qB}{E} \tag{1.3}$$

where q is the applied load and B is the width. In the calculation, it is generally assumed that $\upsilon = 0.5$. Solutions are available for various shapes of loaded area and for different boundary conditions. Fadum (1948) evaluated the modulus of elasticity from unconfined compression tests on hand-carved samples from a test pit and from borehole samples. Load tests on 450 mm square plates, however, gave a modulus which was three to four times that of hand-carved samples from the test pit and 10 to 14 times that of the borehole samples.

It was not realized that settlements could be caused by a lowering of the ground water level until Terzaghi's investigations in the 1920s when the concept of effective stress was proposed. In a lecture to the Norwegian Geological Society in March 1922, Large (1924) reported that the lowering of the ground water level in connection with the Holmenkollen subway in Oslo had caused settlements of neighbouring buildings. He also reported that the seepage velocity in the clay was greater in the horizontal than in the vertical direction. Lidén (1934) reported that a lowering of the ground water level in Sweden had caused settlements of adjacent structures of up to 100 mm/year. He found that the settlement rate was reduced when pumping of ground water was stopped. Lidén warned that a lowering of the ground water-table can be caused by deep-lying sewer lines and by a decrease in infiltration when backyards and streets are paved. The rebound after the last glaciation also contributes to an apparent lowering of the ground water level. Brinkhorst (1936) reported from the construction of the locks at Vreeswijk in Holland that the dewatering of the excavation had caused settlements up to 1000 m from the excavation. A classical example where the lowering of the ground water level has caused large settlements is Mexico City, as described by Cuevas (1936). In 1948, parts of the city were settling at a rate of one millimetre a day; since measurements began in 1898, more than 6 m of subsidence has been recorded (Carrillo, 1948; Gonzalez, 1948).

Damage to buildings caused by a reduction in the water content of soil by seasonal changes by trees and shrubs or by the drying of the soil below by, for example, brick kilns or boiler houses, was reported by Cooling & Ward (1948). Terzaghi (1938b) gave an example of differential settlements of a building on Boston Clay measured between 1869 and 1909 of up to 600 mm: he pointed out that, without doubt, similar general and irregular settlements occur in every city built on beds of clay or silt.

A good example of observed long-term settlements of a building founded on soft clay is the Empress Hotel, Victoria, Canada (Crawford & Sutherland, 1971; Legget, 1973). The hotel, built in 1904, is founded on about 15 m of marine silty clay. Regular settlement observations have been made since 1912. It is one of the longest and most complete records known to exist. The primary and secondary consolidations have been about 1.22 m and 0.15 m respectively. A large quantity of soil was excavated under and around the structure as early as 1912 to reduce the settlements; this is described further in Section 1.9.2 below.

Another example of long-term settlement observations (since about 1920) is the *Jernbanetollsted* in Oslo, which is supported on 9 m long piles in an 80 m deep deposit of soft clay (Andersen & Clausen, 1975). The maximum observed settlement is now 0.7 m, while the final settlement is estimated as 1.1 m. In spite of the large settlements, no damage has been reported.

The test field at Väsby near Stockholm, where systematic settlement observations and other tests have been carried out since 1945 by the Swedish Geotechnical Institute (Chang, 1969, 1981), is also of interest.

1.4.3 Shear strength

1.4.3.1 Strength parameters

The first to express the shear strength, τ_f, of soils in terms of cohesion, c, and angle of internal friction, φ, was Coulomb in 1773 with the well-known equation:

$$\tau_f = c + \sigma_f \tan \varphi \tag{1.4}$$

where σ_f is the normal stress acting on the plane of failure.

Early tests to determine c and φ were made by Navier (1833), Leygue (1885) and Frontard (1914). It should be noted that the concept of pore pressure was not presented until the beginning of the 1920s by Terzaghi. The first pore pressure measurements inside a clay specimen were made in the spring of 1933 by Rendulic (Skempton, 1960).

The first systematic study on the shear strength of clays was that by Bell in 1912 (Bell, 1915). He found that the angle of internal friction was small for clays with a relatively low strength. For a clay with $c=32$ kPa, Bell reported that $\varphi=0.5°$. Terzaghi (1920, 1921, 1925a) investigated very early the nature of friction and the physical properties of clays. His test data

indicated that surface friction was proportional to the contact area between the particles. The observed apparent cohesion of a clay was attributed to the effect of capillary forces.

The shear strength of soils as expressed by the equation:

$$\tau_f' = c' + (\sigma_f - u_f) \tan \varphi' \tag{1.5}$$

where $(\sigma_f - u_f) = \sigma_f'$ is the effective stress on the failure plane, was first formulated by Terzaghi (1924, 1925b): "A hydrostatic pressure does not produce any static friction. Hence it is merely the remainder of the surcharge which counts. The friction resistance increases with decreasing hydrostatic pressure, and does not assume its normal value until the excess water has completely escaped from the layer of clay". In the beginning of the 1920s, Terzaghi also recognized the importance of drainage on the shear strength of clay. The concept of *drained strength* was proposed by A. Casagrande in 1930–32 (Skempton, 1960) and by Jürgensen (1934).

In 1927, Krey demonstrated for clays the relationship between the shear strengths under *undrained* and *drained* conditions. It was believed at that time that $\varphi_u > 0°$ for saturated clays under undrained conditions. Krey's work on shear strength was continued by his colleague Tiedemann (1937).

With the investigations carried out by Hvorslev in Vienna from 1934 to 1936 (Hvorslev, 1937), the understanding of the nature of the shear strength of clays advanced considerably. Hvorslev proposed that the shear strength could be expressed in terms of a *true cohesion,* c_e, and a *true angle of internal friction,* φ_e, as:

$$\tau_f' = c_e + \sigma_f' \tan \varphi_e \tag{1.6}$$

where σ_f' is the effective normal stress on the failure plain. The shear strength parameters c_e and φ_e were evaluated by Hvorslev by direct shear tests. It is also possible to determine φ_e from the slope of the failure surface in unconfined compression tests, as discussed by Terzaghi (1936c, 1938a) and Skempton (1948a). Hvorslev found that the true cohesion of clays was only a function of the water content and that the true cohesion for normally consolidated clays increased linearly with the consolidation pressure, σ_c'. Skempton (1948c) summarized the available data on the true cohesion and the true angle of internal friction, and he reported that the ratio c_e/σ_c' varied between 0 and 0.15.

Skempton (1948a) studied the relationship for clays between the true angle of internal friction, φ_e, and the porosity, n. He found from the slope of the failure plane in triaxial tests that φ_e decreased rapidly with increasing porosity to practically zero for bentonite. Endell & Hoffman (1936a, 1936b) suggested that shear strength is related to the mineralogical composition of the clay and the nature of the cations.

Up to about 1940, clay was generally considered as a cohesionless material in stability calculations, with an angle of internal friction equal to the angle

of repose. It was argued that the cohesion of clays gradually decreased with time and, therefore, could not be utilized fully in design. Evidence of this reduction was, for example, the stability of unsupported deep cuts in clay where the slope of the walls gradually decreased with time if the walls were left unsupported. Another fact which appeared to support this assumption was the increase in the shear strength with depth for normally consolidated clays; this was observed, for example, at pile load tests (T. Hultin, 1911). An angle of 21° was used, for instance, in stability calculations for Gothenburg Harbour.

Considerable uncertainty existed about the evaluation of the shear strength of soft clays in the 1930s. It is believed that the discussion that was going on in the technical journals in Sweden about the shear strength of soft clays is typical. Two groups of thought existed. The first group, with Kjellman (1936a) as its principal spokesman, insisted that the results from consolidated-drained tests should be used in stability calculations because of the increase in the shear strength that takes place when the water content is reduced. Kjellman emphasized that the shear stress in a direct shear test had to be applied in increments and that the clay after each load increment must have sufficient time to adjust its water content with respect to the applied load (consolidated-drained tests). Since the shear strength of a soil depends on the effective stress, the pore water pressure at all times had to be kept as close to zero as possible according to Kjellman, otherwise the effective pressure would be unknown. The second group, with John Olsson (1936/37) as spokesman, insisted that it is not possible in general to utilize the increase of shear strength that takes place with decreasing water content, because of the low permeability of the clay. John Olsson referred to settlement observations on buildings up to 50 years old constructed on soft organic soil which were still settling. These observations indicated, according to Olsson, that the water content of the soil was still decreasing, whereas Kjellman attributed these settlements to lateral creep in the soil without any change of the water content. Olsson suggested that *'$\varphi=0$' analysis* could be applied using the shear strength as determined by fall-cone tests (see Section 1.4.4.4).

1.4.3.2 $\varphi=0$ analysis

Clay was considered as a purely cohesive material ($\varphi=0$) by John Olsson as early as about 1915 when he evaluated the fall-cone test. Some years later, he correlated the fall-cone test with the cohesions determined from old slides on the basis of $\varphi=0$ analyses. Also, W. Fellenius (1918) (cf. Bjerrum & Flodin, 1960) was aware of the $\varphi=0$ method and presented it more specifically in 1926/27. This method was also used in Sweden by B. Fellenius (1935) to calculate the stability of anchored sheet pile walls.

Terzaghi (1932) reported an angle of internal friction of about 0.5° from undrained triaxial tests on a saturated clay. He showed that the shear strength

as determined by undrained triaxial tests was the same as that from unconfined compression tests. For drained conditions, the angle of internal friction was 24°. It was later confirmed by Jürgenson (1934) with triaxial, direct shear and squeeze tests, and by Rendulic (1937) with triaxial tests, that $\varphi=0$ was valid for clays under undrained conditions.

Terzaghi (1936c) was the first to explain why the apparent angle of internal friction of saturated clays under undrained conditions is equal to zero. He also showed that the inclination of the failure plane corresponds to the true angle of internal friction. The $\varphi=0$ concept requires, however, that the pore water is incompressible in comparison with the soil skeleton, as shown by Taylor (1944) and Hilf (1948), and that the effective area of contact between the soil particles is negligible. In this case, a change in the pore water pressure will not affect the effective stress in the soil. The significance of pore water pressures under undrained conditions was discussed by Skempton (1948a).

An additional step in the understanding of the nature of the shear strength of soft clays was the λ-theory proposed by Skempton (1948a,b,c). With the λ-theory, it was possible to explain the apparent contradiction between the $\varphi=0$ analysis and the observation that the undrained shear strength of normally consolidated clays increases approximately linearly with depth (see below). A further development was the introduction of the A and B coefficients suggested by Skempton in 1954.

The validity of the $\varphi=0$ analysis for soft clay was discussed by Skempton & Golder (1948), Odenstad (1948) and others. Skempton & Golder found that this analysis could be used in calculations for lateral earth pressures in uniform clays, for bearing capacity and for the stability of slopes. Jakobson & Odenstad (1940) and Jakobson (1948) used the $\varphi=0$ analysis in the design of embankments on soft clays and for the strengthening of embankments using loading berms (see Section 1.6.2 below). It was, thus, not until 1948 that it was generall accepted that $\varphi=0$ analysis could be used to analyse the stability of cuts and embankments in soft clays throughout the construction period.

1.4.3.3 s_u/σ'_{vo} ratio

B. Fellenius (1936) presented in a discussion to the First International Conference results which showed that the undrained shear strength, s_u, of a normally consolidated clay increases with depth, and, therefore, with the vertical effective stress, σ'_{vo}. The shear strength was evaluated by fall-cone tests and expressed by the so-called H_3-*number* (see below). The undrained shear strength just below the stiff dry surface crust was about 10 kPa ($H_3 \approx 40$) and increased to about 50 kPa ($H_3 \approx 360$) at a depth of 28 m. Similar results were later published by Carlsson (1948). It was not until 1948 when Skempton (1948c) analyzed a number of field records, that it was accepted that the undrained shear strength of a normally consolidated clay increases with increasing depth (Cadling & Odenstad, 1950; Skempton & Bishop, 1950).

It was recognized very early that the shear strength of soft clay increases with decreasing water content, and, therefore, with the consolidation pressure, σ_c'. The Swedish Geotechnical Commission (1922) reported that the undrained shear strength, as determined by the fall-cone test on remoulded samples, had more than doubled below an embankment due to consolidation of the soft clay under the weight of the embankment.

Hvorslev (1937) and Rutledge (1947) found that the undrained shear strength of clays is primarily a function of the water content, w, and that the w—log σ_c' and w—log s_u relationships are two parallel straight lines. The implication of this observation is that s_u/σ_c' (or the s_u/σ_{vo}' ratio) is constant. It is believed, however, that Jürgenson (1934) was the first to suggest that this ratio is constant and that it can be used to calculate the preconsolidation pressure.

Terzaghi (1947) pointed out that the s_u/σ_{vo}' ratio of a clay in the field can be considerably lower than the s_u/σ_{vo}' ratio evaluated from isotropically consolidated-undrained tests in the laboratory, since the lateral pressure in the field is lower than the isotropic consolidation pressure in the triaxial tests. It had been observed in several cases that the shear strength of soft normally consolidated clays does not increase with depth (Terzaghi, 1941; A. Casagrande, 1944).

As mentioned previously, Telford in 1809 was the first to utilize consolidation to "compact and solidificate" clay for practical purposes (Telford, 1830).

Biemond (1936) took advantage of the increase in shear strength during consolidation by constructing an embankment in stages which otherwise would have failed. Also, Middlebrooks (1936) described a case where the increase in the undrained shear strength that took place during construction was utilized. It was learned early at the Gothenburg Harbour that excavation and dredging had to be carried out in stages to reduce the risk of failure in the soft clay.

1.4.3.4 Reduction factors for measured shear strength

John Olsson (1937) and B. Fellenius (1938a) pointed out that the shear strength of some organic soils *(gyttja)* is overestimated by the fall-cone test and that a reduction coefficient of 0.6 to 0.7 is required for such soils. It is believed that this was the first time that a reduction in the undrained shear strength for organic soils with a high liquid limit or a high plasticity index was suggested. The Swedish Geotechnical Institute used, from the middle of the 1940s, a reduction of the shear strength measured by the fall-cone test; these measured strengths were reduced by 20% for organic clay and 40% for 'gyttja' soil (Jakobson, 1947). A 'history' of the methods for correcting the measured undrained shear strengths of soft clays has recently been presented by Helenelund (1977).

1.4.3.5 Residual strength

Residual strengths of clays were investigated early on by Hvorslev (1937) and Tiedemann (1937). The measured residual strengths varied from 20% to 60% of the peak strengths. Tiedemann observed that the residual strength of a normally consolidated clay is approximately equal to that of undisturbed overconsolidated samples of the same clay.

1.4.3.6 Sensitivity

The term *sensitivity* was defined by Terzaghi in 1944 as the ratio of the undisturbed and remoulded shear strengths as measured by unconfined compression tests. For the soft Scandinavian clays, sensitivity has been mainly used in terms of strengths from fall-cone tests. John Olsson presented to the Swedish Geotechnical Commission the concept of sensitivity in January 1916 (Bjerrum & Flodin, 1960), and he further investigated in 1917 the shear strengths of clays as affected by remoulding and disturbance from sampling, as part of the work for the Commission. It was, however, not until about 1923, when the first piston sampler for taking undisturbed samples of soft clay was developed, that sensitivity in its present sense could be formulated (J. Olsson, 1925). Among the railway geotechnical engineers in Scandinavia, the term *slide number*, H_3/H_1, (see Section 1.4.4.4) was used in the 1930s instead.

1.4.3.7 Anisotropy

Many investigators were aware early that shear strength can vary with the direction of the failure plane and with the direction of the applied stress. Rendulic (1938) reported, for example, that the shear strength with respect to the effective stresses in a soil is higher (7 to 9%) in axial extension than in axial compression tests. Casagrande & Carillo (1944) investigated analytically the shear strength of anisotropic materials. Brinch Hansen & Gibson (1949) utilized the λ-theory to evaluate for different stress conditions the variation in the shear strength along different planes for anisotropically consolidated clays.

1.4.3.8 Creep

It has been known for a long time that creep can affect the shear strength of soft clays. Terzaghi pointed out in 1931 that plastic deformations can start at a shear stress considerably lower than the shear strength determined by standard laboratory tests and that these deformations may continue for a long time. Terzaghi (1941) further suggested that the effective stress in a soil is partly carried by a viscous film of absorbed water which surrounds the soil particles and partly by direct bond. The effective stress in the clay can, according to Terzaghi, be divided into film bond and grain bond, each with different characteristics. A film bond stress is associated with viscous flow (creep), while a grain bond stress does not produce any significant deformations.

The effect of loading rate and creep on the undrained shear strength of clays has been investigated experimentally by Taylor (1944), Rutledge (1947), Geuze (1948) and Casagrande & Wilson (1949). Geuze (1948) studied creep in soft clays using direct shear tests and found that the shear deformations increased linearly with log t. Casagrande & Wilson (1949) reported that the undrained shear strength of some clays decreased 15 to 30% when the loading rate was reduced 10,000 times. At high loading rates, the shear strength was 1.5 to 2.0 times the static strength; the largest increase was observed for soils with a low initial shear strength.

Tsytovich & Zaretsky (1969) have pointed out that Puzirevsky (1934) recognised the occurrence of creep in soils. Other early work in this field in the USSR was that of Maslov (1936, 1949).

1.4.4 Laboratory methods for the evaluation of shear strength

1.4.4.1 Introduction

Many different methods were developed, particularly between 1913 and 1936, to determine shear strengths under different loading and drainage conditions both in the laboratory and in the field. Examples of laboratory methods are the direct shear test, torsion shear test, fall-cone test, laboratory vane test, unconfined compression test, squeeze test and triaxial test. By 1936, most leading soil laboratories were able to carry out direct shear, triaxial and consolidation tests (see *Proceedings of the First International Conference*, 1936, Vol. 2). The development of these methods is described briefly below.

1.4.4.2 Direct shear test

The first method which was used to determine the shear strength of soils was the direct shear test. A direct shear apparatus was constructed by Collin as early as 1846, in which square soil samples, 40×40 mm in cross-section, could be tested. Also Leygue (1885) constructed a direct shear apparatus which was mainly intended for cohesionless soils. The earliest shear box which was similar to modern ones was used by Goodrich (1904); the box consisted of two halves, each holding a 305 mm cube (Skempton, 1958). The first practical tests were made by Frontard in 1910 in connection with his consolidation experiments (Frontard, 1914); he used 100 mm high compacted clay samples (not fully saturated). Results from early direct shear tests on clays were also reported by Bell (1915); different clays were tested with his apparatus (quick tests) in about 30 minutes without drainage or consolidation. A direct shear test device was also developed by Krey (1926), which was later modified by A. Casagrande. In connection with his investigations on effective stress, Terzaghi constructed in 1921 a shear box for soft clay (Terzaghi, 1925b; Skempton, 1960) in which a vertical pressure was applied and maintained for about two days before a fairly rapid rate of application of the shear force was

used, the test conditions being approximately consolidated-undrained by today's nomenclature.

A new type of direct shear apparatus was developed in 1936 by Kjellman (1942, 1950), where a soil sample was subjected to a pure shear deformation. The cylindrical soil sample, 60 mm diameter, was surrounded by a series of aluminium rings and a rubber membrane so that the drainage conditions could be controlled, as illustrated in Fig. 1.14.

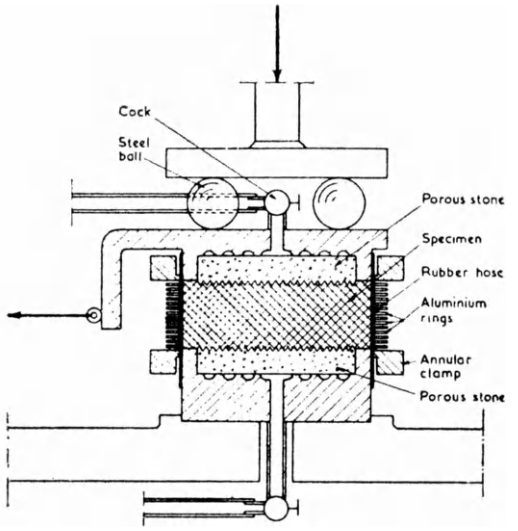

Fig. 1.14. Kjellman's direct shear apparatus of 1936.

1.4.4.3 Torsion shear test

A significant development in the study of residual strength was the torsion shear apparatus in which ring-shaped specimens could be tested (Cooling & Smith, 1935; Hvorslev, 1936a, 1937, 1939; Tiedemann, 1937). With this apparatus, it became possible to study the shear strength at large shear deformations. Hvorslev had reported as early as 1936 that the reduction in shear strength at large deformations could be as large as 30 to 60%.

1.4.4.4 Fall-cone test

The fall-cone test was developed by John Olsson in 1915 to measure the strength of soft clay (Bjerrum & Flodin, 1960). A steel cone, as shown in Fig. 1.15, was released when the point just touched the surface of a clay sample. The penetration in millimeters of the cone was taken as a measure of the strength or the consistency of the clay. The fall-cone test was inspired by the Brinell hardness test, where the indentation of a steel ball is measured,

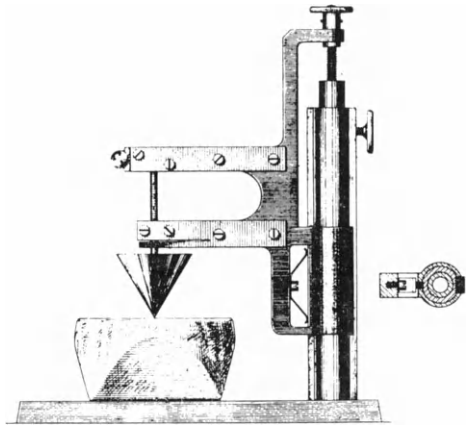

Fig. 1.15. Swedish fall-cone test device of 1915/1922.

a method widely used at that time to determine the yield strength of steel. Cones weighing between 10 and 1000 g with different apex angles were tested. Three different cones were finally chosen: 10 g and 60°, 60 g and 60°, 100 g and 20°. The 60 g, 60° cone was chosen as standard.

The strength was expressed by John Olsson by a so-called relative strength number, *H-number,* which corresponds to the weight in grams of an equivalent cone which gives a penetration of 10 mm. Conversion tables were prepared so that the results from the standard 60 g cone could be used. Three different H-numbers were used (H_1, H_2 and H_3). The H_1-value expresses the relative strength of remoulded clays, while the H_3-number is used for 'undisturbed' clays.

John Olsson correlated the H_3-value with the undrained shear strength of a soft clay. Samples from the site where Wendel in 1900 had investigated the bearing capacity of timber piles were tested (see Section 1.8.2). He also correlated results from the fall-cone test with the cohesions deduced from old landslides using $\varphi=0$ analyses. He found that an H_3-value of 40 corresponded to a shear strength of about 10 kPa, i.e. the penetration of a cone weighing 40 g will be 10 mm when the shear strength is 10 kPa.

The fall-cone test was also used by the Swedish Geotechnical Commission (1922) to determine the so-called *fineness number* of a soil, today called the *fall-cone liquid limit,* as mentioned previously.

Two different fall-cone apparatuses were developed by Terzaghi in 1923 (Terzaghi, 1927). The apex angle of the cone was 90°. His test data indicated that the shear strength of a soil was proportional to $1/h^2$, where h is the penetration of the cone. Terzaghi warned that the results from the test could be misleading unless the test is used to estimate the relative consistency of materials with similar characteristics.

Skaven Haug (1931) calibrated the Swedish fall-cone test against undrained shear strengths determined by direct shear tests. The apparatus shown in

Fig. 1.16 was used. For Norwegian clays, Skaven Haug (Fig. 1.17) found a unique relationship between the H_3-value and the undrained shear strength, as expressed by the relationship:

$$s_u = \frac{H_3}{(3.2+0.0073\ H_3)}\ \text{kPa} \tag{1.7}$$

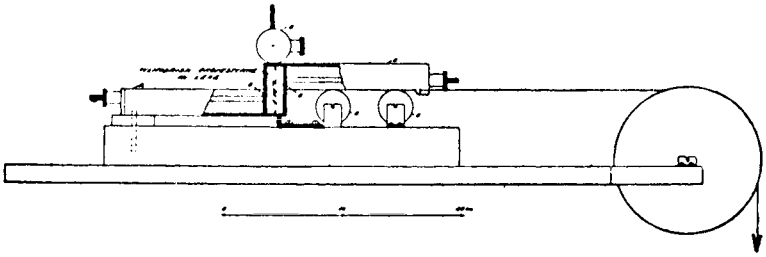

Fig. 1.16. Skaven Haug's shear box of 1931 for calibrating the fall-cone test.

Fig. 1.17. Sverre Skaven Haug of Norway.

T. Hultin (1937) similarly investigated the relationship between the H_3-number and the undrained shear strength for Gothenburg Clay, and he found that:

$$s_u = \frac{H_3}{(4.0+0.0055\ H_3)}\ \text{kPa} \tag{1.8}$$

A similar expression was obtained by Caldenius (1938). The average relationship:

$$s_u = \frac{H_3}{(3.6+0.0064\ H_3)}\ \text{kPa} \tag{1.9}$$

has been extensively used in Sweden to evaluate the shear strength of soft clays (Jakobson, 1947). Also, Bror Fellenius (1938a, 1938b) correlated H_3-values with undrained shear strengths determined by a punching shear apparatus on samples 43 mm diameter. The force required to punch a 25 mm hole in the 5 mm thick samples was measured. The relationship proposed by Fellenius corresponded closely to that determined by Skaven Haug (1931).

1.4.4.5 Unconfined compression test

One of the oldest methods to determine the shear strength of clays is the unconfined compression test. A simple unconfined compression test was demonstrated by John Olsson to the Swedish Geotechnical Commission in 1915 (Bjerrum & Flodin, 1960). Modern unconfined compression apparatuses were developed by Cooling & Smith (1935, 1936) and by Kjellman in the beginning of the 1940s (Kjellman, 1942). The apparatus by Cooling & Smith was provided with conical ends to eliminate the restraint offered by flat end plates. They found that the shear strength of 'plastic' clays was half the unconfined strength. The Kjellman apparatus (Fig. 1.18), in which four samples could be tested simultaneously, plotted the stress-strain relationships automatically. A portable apparatus was designed by Cooling & Golder in 1940.

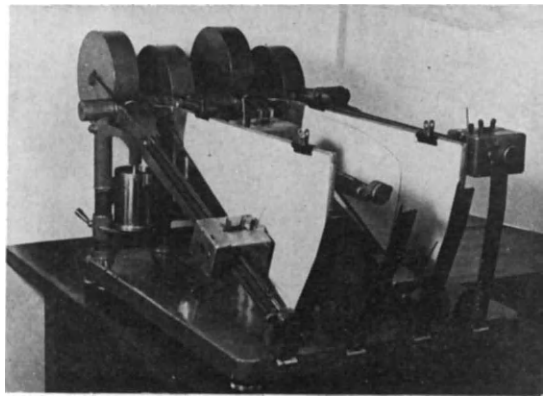

Fig. 1.18. Kjellman's unconfined compression test apparatus of 1940.

1.4.4.6 Squeeze test

Jürgenson (1934) developed the squeeze test to measure the shear strength and deformation properties of soft clay. A thin sample of clay is squeezed between two rigid plates until the clay is forced out through the two open sides. The normal pressure can be varied by simply closing one of the outlets. Jürgenson also proposed a method to analyze this test. He pointed out that the squeeze test gives an average shear strength of a clay, while the direct shear test and the unconfined compression test indicate the shear strength along the weakest plane in the soil. Warlam (1936) recommended that the

squeeze test should be used primarily for comparative studies of the strengths of different clays or the strength of the same clay at different water contents.

1.4.4.7 Triaxial test

Triaxial tests on rock were already being carried out at the turn of the century by Adams and Nicholson with marble samples which were confined by a metal jacket.

A primitive triaxial apparatus for soft clay was constructed by Westerberg in Sweden in 1921 (Krey, 1927; Bjerrum & Flodin, 1960) where a soil sample could be subjected simultaneously to a vertical and lateral pressure independently of one another. The *ratio* of the two pressures was found to be constant at failure for sand, while for clay the *difference* between the two pressures was constant. These tests indicated that sand behaved as a cohesionless material with a constant angle of internal friction, while clays behaved as a purely cohesive material with $\varphi=0$. It is believed that Westerberg is the first to have proved experimentally that a saturated clay under rapid loading behaves as a $\varphi=0$ material (Skempton, 1960).

The cell test, which is a type of triaxial test, was developed in Holland by Buisman about 1930 to determine shear strengths mainly of clays. It was pointed out by Bishop (1950) that the maximum strain in this test is not sufficient to cause failure of the sample and that the measured shear strength is lower than that from an undrained triaxial test.

A triaxial apparatus was constructed in Vienna by Terzaghi around 1930 (Skempton, 1960). This first triaxial apparatus for soil was further developed by Rendulic in 1933 when he was working together with Terzaghi on the shear strength of clays (Rendulic, 1935a). The pore pressure inside a clay specimen could now for the first time be measured. Rendulic carried out both drained and undrained tests, and he was able to demonstrate by direct observation the validity of the principle of effective stress (Skempton, 1960; see also Rendulic, 1936b). The first triaxial apparatus for soils in the United States was constructed at MIT by Casagrande about 1930 and was a modification of the triaxial apparatus developed by Terzaghi. Ter-Stepanian (1963) pointed out that Professor Davidenkov used triaxial tests in Russia at the beginning of the 1930s.

A true triaxial apparatus, intended mainly for sand, was developed by Kjellman (1936b) using cubical samples where the three principal stresses could be varied independently. The load was applied through a system of rods. The maximum deformation of the sample was small, and the sample could therefore not be loaded to failure.

1.4.4.8 Laboratory vane test

After the field vane borer (see Section 1.5.4) had been developed in the 1940s, laboratory vane equipment also appeared (e.g. Evans et al, 1948; Jakobson, 1954).

1.5 SITE INVESTIGATIONS

1.5.1 Early beginnings

We know from the Roman engineer Vitruvius (first century B.C.) and the Italian architect Palladio (1508–1580) that the Romans had a good knowledge of the behaviour of soils as related to road construction (Jumikis, 1967; Plommer, 1973). It is also known that Leonardo da Vinci and other universal genii of the same time worked as 'soil engineers'. The design of structures and other civil engineering works during medieval times was primarily based on experience and intuition. Hardly any information is available on how site investigations were carried out, but 'test pits', often in the form of wells, have been mentioned. Palladio indicated that certain species of grass would only grow where the ground was *firm,* and that firm ground is encountered "if the ground does not resound or tremble if something heavy is dropped. In order to ascertain this, one can observe whether some drum-skins placed on the ground vibrate and give off a weak sound or whether the water in a vessel placed on the ground gets into motion." (Atterbom & Branzell, 1928).

In old Scandinavian literature, one can find notes about the use of stakes or iron rods for exploratory purposes. One such example is in connection with investigations after the Sarpfossen landslide when, in 1722 and 1726, stakes 14 to 16 *alen* along (about 13 to 15 m) or rods 8 *alen* long (about 7 m) were used in the soft clay without reaching firm layers (Nygård, 1944). The need for making thorough soil explorations was stressed by Thomas Telford, who is regarded as the father of civil engineering in England, as early as 1804 in connection with the construction of the Caledonian Canal. He mentions that the "shore consisted of soft mud into which an iron rod could easily be thrust 55 feet" (Tomlinson, 1956). Up to the 1930s, the only form of subsoil exploration, however, was frequently that by a foreman in the bottom of an excavation "using no tool than the heel of his boot and the allowable pressure was selected on the spot, on the basis of this verdict" (Terzaghi, 1953). A short stick or cane was often used instead of the heel. Perhaps the soil investigations for the Tower of Pisa were done by this method, since a clay layer located below a surface layer of sand was apparently not known to the architect.

1.5.2 Investigation methods and equipment

1.5.2.1 General boring methods

Tools for exploration in soil and rock were first developed for well borings and for mining explorations. Primitive percussion drills were used in China even before Christ. Deep holes, one as deep as 1242 m in the 17th century, have been reported as being achieved with these methods (*Svenska Diamantbergborrningsaktiebolaget*, 1936). An ancient screw auger, as well as other types of screw and can samplers *(anciennes tarières),* have been described by Cambefort (1955). Boring equipment of British origin from about 1700 has

been mentioned by Whyte (1976), and this device was 'imported' to Sweden from England as early as 1725 and was called the *English Earth Auger* (Triewald, 1740). A Swedish auger similar to that mentioned by Cambefort (1955) has been described by Gadd (1773). Lippmann (1879) described a number of screw augers which could be used down to 100 m depth, and these were still used in Sweden in soft clay around the turn of this century (Karlsson, 1912; H. Olsson, 1915, Vol. 1).

An impressive collection of universal boring equipment from the 19th century was presented in the handbook *Der Grundbau* by Strukel (1895), in which the augers mentioned above are included; this collection is reproduced in Fig. 1.19. A similar handbook was also prepared by Kolderup (1894).

A boring manual was published by the Swedish Geotechnical Commission in 1917. In this early guide for subsoil exploration, recommendations were given about borings, the handling of samples, the classification of soils and the presentation of test data.

A significant advancement was the appearance of the admirable report by Hvorslev (1949) which gave a complete picture of all the equipment of that time. Associated geotechnical aspects, such as sample disturbance, were also discussed in the report.

1.5.2.2 Wash borings

Sediments in drilling wash water have been used for centuries as an indication of soil and rock conditions, especially in mining. Terzaghi (1953) made the comment about soil investigations based on an examination of the cuttings brought up with wash water that "Similar techniques were used by some of the sooth sayers in ancient Greece for the purpose of predicting future events and the results were equally reliable." The wash boring method is described in handbooks from the 19th century (e.g. Kolderup, 1894; Strukel, 1895). Strukel described how wash boring was a new and convenient method and that it could be used also in clay. He added that the wash boring method was first used by a German engineer, Jensen, but that it should be credited to a "boring engineer" with the name Fouvelle.

According to Terzaghi (1953), wash borings were commonly used in the USA in the 19th century. The method probably originated during the construction of the Erie Canal at or before the middle of the 19th century. Also, Terzaghi used the method and developed it further in 1930 by combining the method with static penetration tests (Terzaghi, 1953; Terzaghi & Peck, 1948). Wash borings were also used in Norway in connection with investigations of landslides (Reusch, 1901), and in Gothenburg by, for example, Wendel (1900). An open pipe (casing) with 50 mm inside diameter was used, while a smaller pipe with a point shaped as a chisel was lowered into the casing. Water was forced through holes in the point, and the soil removed by the water jets was collected. Little (1961) mentioned that wash borings were also carried out in Canada for subsoil exploration purposes in 1913. G. Holmsen

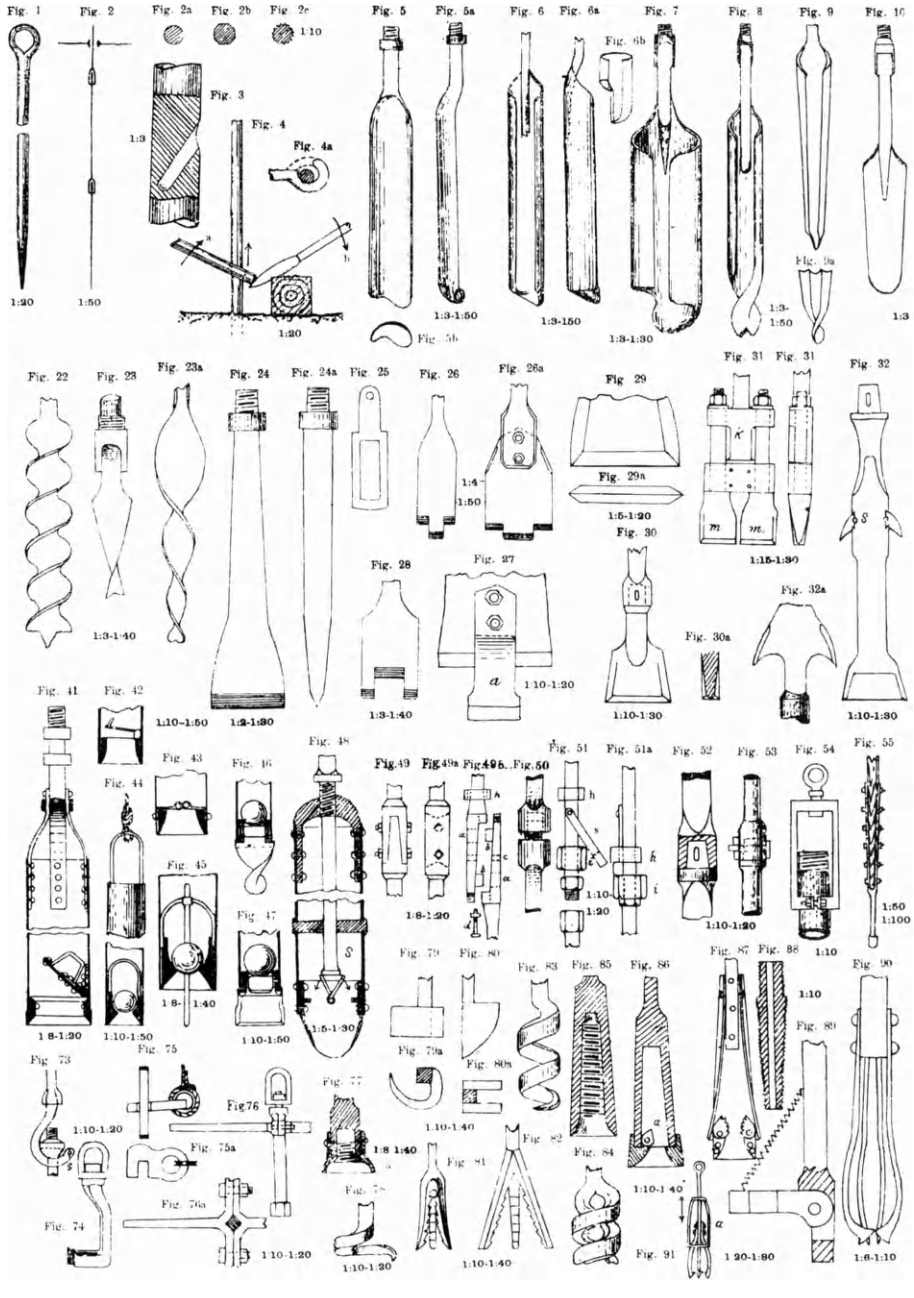

Fig. 1.19. Collection of boring equipment used during the 19th century (Strukel, 1895).

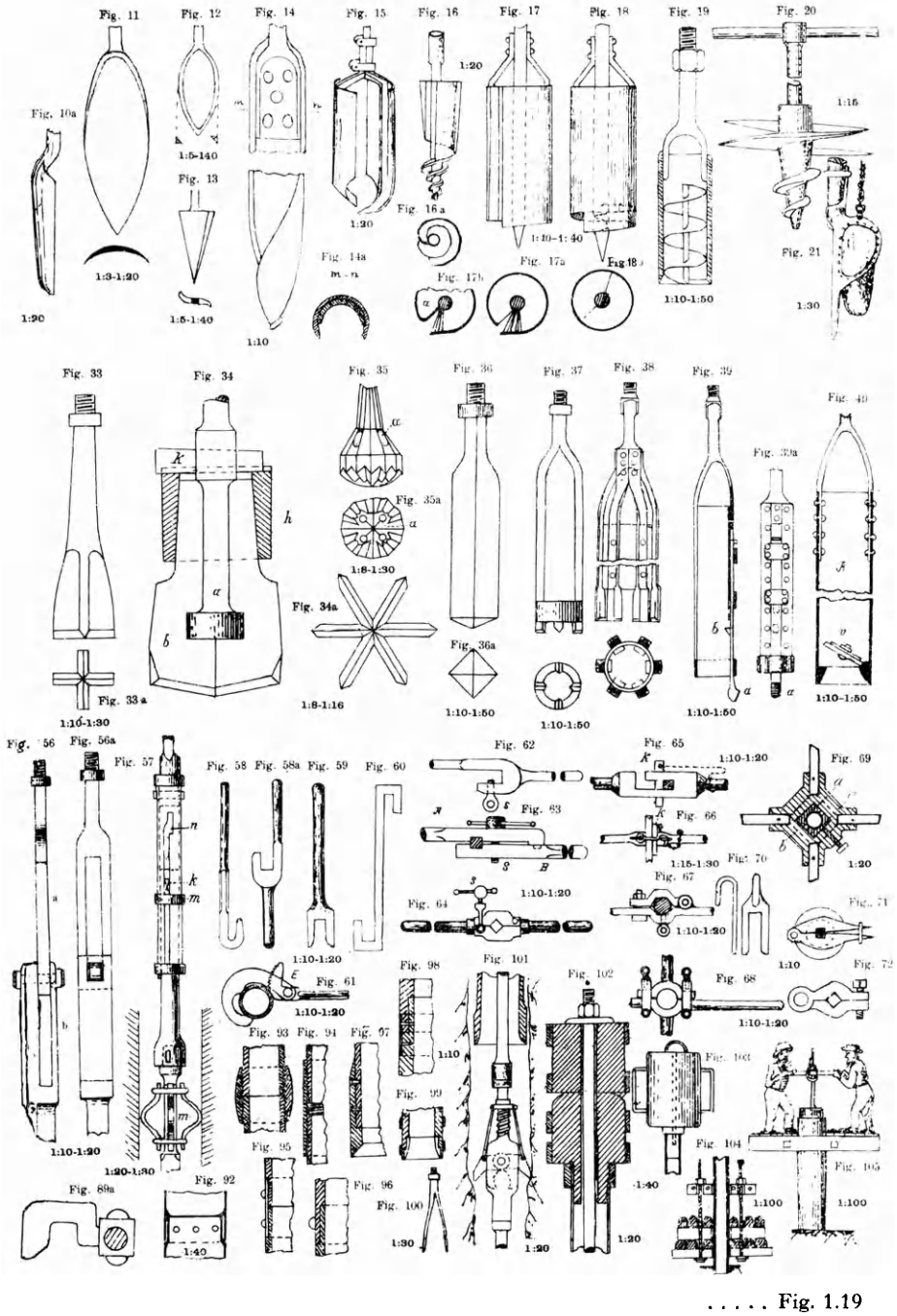
Fig. 11
Fig. 12
Fig. 14
Fig. 15
Fig. 16
Fig. 17
Fig. 18
Fig. 19
Fig. 20
Fig. 10a
1:5-140
Fig. 13
1:3-1:20
1:20
1:20
1:16
Fig. 16a
1:10-1:40
Fig. 14a
Fig. 17a
Fig. 18a
Fig. 21
Fig. 17b
1:10-1:50
1:30
1:90
1:5-1:40
1:10
Fig. 33
Fig. 34
Fig. 35
Fig. 36
Fig. 37
Fig. 38
Fig. 39
Fig. 40
Fig. 35a
Fig. 33a
1:8-1:30
Fig. 34a
Fig. 36a
1:10-1:50
1:10-1:50
1:10-1:50
1:10-1:50
1:10-1:50
Fig. 33 a
1:8-1:16
1:16-1:30
Fig. 56
Fig. 56a
Fig. 57
Fig. 58
Fig. 58a
Fig. 59
Fig. 60
Fig. 62
Fig. 65
1:10-1:20
Fig. 69
Fig. 63
Fig. 66
1:15-1:30
Fig. 70
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Fig. 67
Fig. 64
1:10-1:20
1:10-1:20
Fig. 71
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1:10
Fig. 72
Fig. 61
1:10-1:20
Fig. 98
Fig. 101
Fig. 102
Fig. 68
1:10-1:20
Fig. 93
Fig. 94
Fig. 97
1:10
Fig. 103
Fig. 99
Fig. 104
Fig. 105
Fig. 95
1:10-1:20
1:20-1:30
Fig. 92
Fig. 96
1:40
Fig. 100
1:100
1:100
Fig. 89a
1:40
1:30
1:20
1:20

. . . . . Fig. 1.19

(1930) indicated that wash borings have limitations (e.g. a stone can be mistaken as firm rock). He stated that an experienced boring foreman can form a rather good picture of the soil conditions by this method. In connection with investigations of the large landslide at Vaerdalen in Norway in 1893, wash borings were made to a 100 m depth (G. Holmsen, 1938).

Detailed instructions for wash boring in connection with sampling were issued by the Swedish Geotechnical Commission (1917) and by Skaven Haug (1940a), who suggested that wash borings were superior to other boring methods in clay even if the depth was as large as 80 m but that it was a problem to establish the quality of the clay.

1.5.3 Penetration testing

1.5.3.1. Swedish sounding methods

One of the main problems during the construction of the railways in Sweden, as well as in other Nordic countries, was, as mentioned previously, the soft soils, mainly peat and clay. In many cases, the dry surface crust was sufficient to carry the weight of the embankments. The strength of the dry crust and of the underlying soft clay and the thickness of the different soil strata were in early times simply investigated by forcing a stake into the soil. At the end of the 19th century, steel rods in one length of up to 10 m were used, although Senneset (1974) described a soil investigation in Norway in 1736 where a 12 m long sounding rod was used. The rods were either circular of diameter 15 mm to 30 mm or were about 28 mm square. Hexagonal and octagonal rods were also used. The rods, which were later spliced by external couplings, were pressed, rotated or hammered into the ground. The penetration resistance was often expressed in terms of the number of men required to push the sounding rod into the soil; it was then possible to classify roughly the soil as *soft* or *stiff*. At the end of the 19th century, Svensson (1899) proposed that the "stiffness" of the soil should be measured by a steel rod. He divided the soil into the grades 0, 1, 2, 3, etc. Grade 0 indicated that the steel rod sank under its own weight; at grade 1, one man was required to push the rod without rotation; at Grade 2, two men were required, etc. For very soft soils, half grades were used ($\frac{1}{2}$ and $1\frac{1}{2}$).

Wendel (1900), then head of harbour and river works in Gothenburg, pointed out the disadvantage of square sounding rods. When stones were encountered in the clay, the resistance was very uneven because of the outside couplings. He also criticised the method of expressing the penetration resistance in terms of the ability of the boring crew to push down the sounding rod. Wendel also described a "new" probe, which was then under construction in Gothenburg, similar to the one in the upper left corner of Fig. 1.19 (indicated as *Fig. 1*). This tool, called a *needle probe,* consisted of a number of short pipes which were spliced using inside couplings, and the outside surface

of the probe was thus smooth. It was intended to drive down the probe using a weight which was dropped a predetermined height, and to measure the penetration for each blow. It was believed that the penetration resistance would be a measure of the bearing capacity of the soil. It is not known if this probe was ever actually used in Gothenburg.

About ten years later, a discussion was going on among civil engineers (mainly railway builders) in Sweden about the "need to investigate the bearing power of ground on soft subsoil" (Sparr, 1912), because of several failures which had occurred in the southwestern parts of Sweden. The importance of carefully executed borings was emphasized, and it was suggested that an investigation of existing boring methods should be made. The following year, Dahlberg (1913), then head of railway construction in Sweden, discussed in a comprehensive and well-documented report the problems connected with the construction of railways on soft soil. He pointed out the limited knowledge about the stability and settlement of embankments. He presented a relationship, based on experience, between bearing capacity and the results from soundings made with 25 mm diameter rods: this is shown in Table 1.1. The bearing capacity could be higher, according to Dahlberg, if there was a stiff surface crust. This layer "can, as a rule, not be trusted" if the thickness is less than 4 m or the embankment is not low.

Table 1.1. Relationship between penetration resistance and bearing capacity (after Dahlberg, 1913).

Penetration resistance (number of men)	Maximum height of embankment that soil can carry (m)
½	0.5
½–1	0.5–1
1–2	1–2
2–3	2–5
3–4	5–15

In the manual by H. Olsson (1915) published by the Swedish State Railways, the whole railway engineering field was reviewed. The boring equipment which was used at that time was described, the simplest being a narrow steel rod or probe which was provided with a point (needle). The length of the rods was 3 m to 4 m, and they were spliced with exterior couplings. The lower rod could be provided with a 'can' or a pocket sampler. The more "advanced sounding device" shown in Fig. 1.19(7) could be used down to 20 or 30 m, below which thicker rods or tubes with strong couplings were required. In the same manual, another device with about the same appearance as the later Swedish Weight Sounding Device was described (Fig. 1.20). One metre long rods of 15 mm diameter were used. The bottom

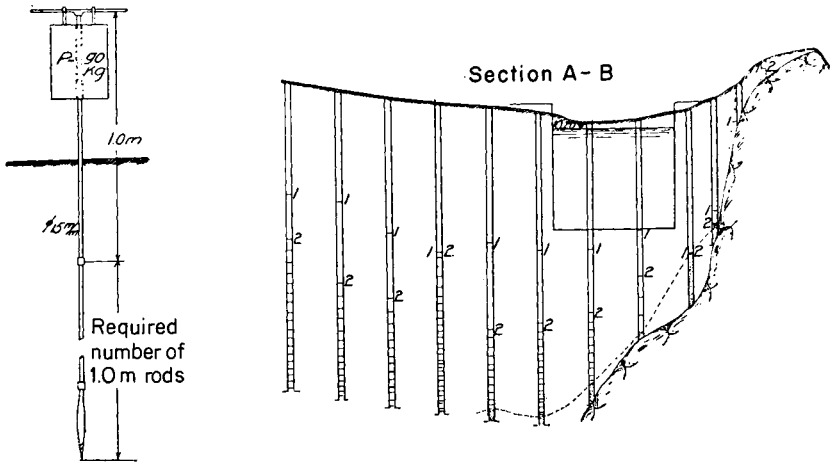


Fig. 1.20. Swedish sounding method from the beginning of the 20th century (H. Olsson, 1915).

rod was provided with a twisted point to facilitate the penetration. The type of sound (noise) in the rod when the point penetrated a layer of coarse grained soil was recorded; in this way, an experienced foreman could distinguish between sand and gravel. The penetration resistance was expressed as the number of men (one, two, etc) required to push the penetrometer into the soil. If the penetrometer could not be pushed down by hand, a 90 kg weight was added, the penetrometer was rotated, and the penetration every 25 turns was recorded. The penetration tests were normally supplemented by sampling.

The penetration method mentioned in the manual by H. Olsson was developed further by the Swedish Geotechnical Commission (1917, 1922). The thrust produced by one or two men was replaced by the number of weights (up to 100 kg) required to push down the penetrometer, as shown in Fig. 1.21. The penetration resistance when the rod sank under its own weight was also recorded. In this way, it was possible to classify the different strata with respect to consistency as, for example, a "25 kg clay" or a "75 kg clay". When the pentration resistance exceeded 100 kg, the rod was rotated and the penetration every 25 half-turns recorded.

The Swedish Weight Sounding Method, as the above penetration method has been called, is today mechanized. The rod diameter has been increased to 22 mm and the number of half-turns to drive the rod 200 mm is recorded. A Scandinavian standard for the method was adopted in 1974, and a European standard was approved in 1977 (see *Proceedings, European Symposium on Penetration Testing, ESOPT,* 1974, Vol. 1).

1.5.3.2 Static penetration tests

The first static penetration tests were probably carried out in Canada as early as 1872 by the civil engineer Sir Stanford Fleming (1876) during the

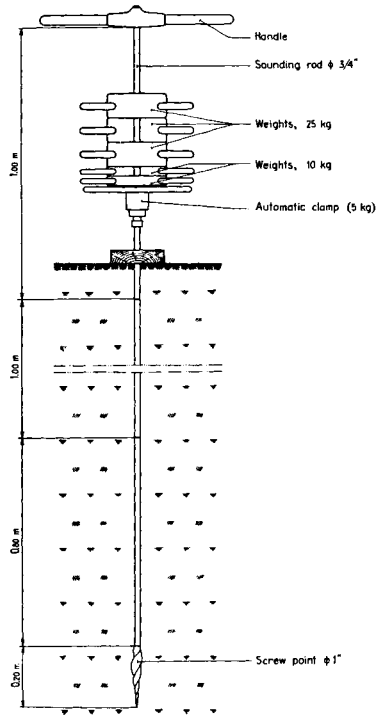


Fig. 1.21. Swedish Weight Sounding Method. (The photograph was taken about 1920 and is from the files of the Swedish State Railways).

construction of a bridge near Newcastle, New Brunswick. The purpose of the tests was to determine the load carrying capacities of the different soil strata. A 75 mm diameter steel rod with a blunt end was used, and the friction along the rod was eliminated by means of a 125 mm steel casing. The tests were carried out in water, and the rod was loaded above the water surface with weights. The loading time was varied. This technique was very advanced for its time when one considers that this sounding method was reinvented by Terzaghi about 1930 (Peckover & Legget, 1973).

According to Terzaghi (1930, 1953), continuous variations in penetration resistance were probably first recorded in 1929 in New York City. The cone was provided with small nozzles for jetting to facilitate the penetration and to reduce the friction along the rod. This type of cone is known today as a *wash point*.

In the early 1930s, Professor A.S. Keverling Buisman at the Technical University of Delft developed a method to determine the point resistance of piles using a cone which was pushed into the soil continuously. He derived formulae based on Prandtl's theoretical studies on metals which related the penetration resistance to the shear strength of the soil (Buisman, 1935;

Barentsen, 1936). It then became possible to predict the bearing capacity of piles.

The Dutch Cone Penetrometer was described for the first time by Barentsen (1936), who at that time was employed at the Rijkswaterstaat in Holland. This apparatus was mainly used to determine the soil conditions beneath road embankments. The 10 cm^2 cone with an apex angle of 60° was pushed down by a 15 mm rod which was protected by a tube of 19 mm inside diameter. The penetration rate was kept constant at 10 mm per second, and the resistance was read every 500 mm together with the friction along the tube. The shear strength was evaluated from the assumption that the point resistance was 11 times the shear strength of the soil. The maximum capacity of this first apparatus was limited; it could not be used in sand, and only soils with a *sounding firmness* less than 1 MPa (10 kg/cm^2) could be penetrated.

The Dutch Cone Penetrometer has gradually been improved, and it is used today all over the world and in all kinds of soils. The development up to 1950 has been described by Kantey (1951) (see also *ESOPT,* 1974).

A static cone penetrometer was developed in Sweden in 1948 by Kjellman in an effort to mechanize and improve the Swedish weight sounding methods (Swedish Geotechnical Institute, 1949). A cone of 25 mm or 80 mm diameter was pushed down with a maximum force of 10 kN using a rod of 19 mm diameter (now 22 mm). The rod was rotated at the same time it was pushed down, and the relationship between the rotation and the penetration was kept constant. The skin friction was calculated from the torque required to rotate the rod. The point resistance, which was recorded automatically as a function of depth, was determined by subtracting the skin friction from the total vertical resistance, as indicated in Fig. 1.22. This was done automatically.

Another later development in Sweden was the Nilcon Static Sounding Method. The apparatus is provided with a sleeve coupling just above the 10 cm^2 point. The skin friction resistance along the unprotected rod is determined by first pulling and then pushing down the rod about 50 mm; the

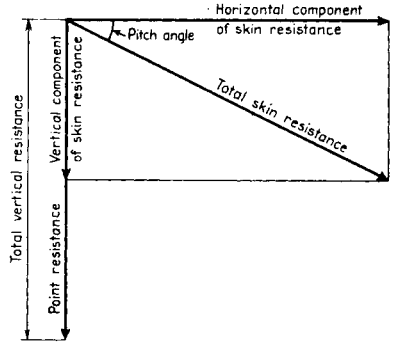


Fig. 1.22. Principle of Kjellman's static cone penetrometer of 1948.

point does not move because of the sleeve coupling. The latest version is based on the Kjellman concept of measuring the torque when the rod is rotated. The point resistance is recorded automatically (see *ESOPT,* 1974).

A third Swedish method is the Borros Static Sounding Method, where the point resistance is measured directly in-situ with strain gauges, and a fourth method is the Geotech Static Sounding Method developed in 1973, in which the penetration resistance is measured in-situ by a vibrating wire gauge. The signals are transferred to the surface by a transmitter at the penetrometer point (see *ESOPT,* 1974).

A large number of different static penetrometers have been developed since 1960 in Holland, Belgium, France, Norway, Germany, Italy, etc, as described by Sanglerat (1972) and *ESOPT* (1974). These penetrometers are mainly used in cohesionless soils.

1.5.3.3 Dynamic penetration tests

The most common soil exploration method is without doubt the Standard Penetration Test *(SPT),* originally proposed by Terzaghi in 1926 and further developed by A.H. Mohr (Terzaghi, 1953). In this method, which is a combination of penetration testing and sampling, the number of blows *(N-value)* is measured for a fixed penetration of the standard sampling spoon into the bottom of a borehole (Terzaghi & Peck, 1948). The *SPT* is mainly used in cohesionless soils and is hardly used at all in Scandinavia; it is of little use in soft clay. A comprehensive state-of-the-art report on the *SPT* method, with more than 350 references, was fairly recently presented by de Mello (1971). The reader is also referred to the summary prepared by Hvorslev (1948). The *SPT* procedure used at present in different countries varies very widely, and a European standard for the *SPT* test was proposed in 1977 to overcome this difficulty.

The Ram Penetration Method is mainly used in cohesionless soils. In soft clay, it is primarily used to predetermine the lengths of point bearing piles. The Swedish Ram Sounding Method was developed in the 1930s by the boring equipment firm Borros Co. Ltd. Different samplers are used in combination to obtain 'undisturbed' soil samples. The method has been standardized in Sweden (see *ESOPT,* 1974).

1.5.4 Vane tests

1.5.4.1 Early vane borers

The development of the vane borer made it possible for the first time to determine the in-situ shear strength and sensitivity of a soft clay. A surface vane borer was used in England as early as 1944 by the Army Operational Research Group to investigate the mobility of military vehicles (Smith, 1945; Evans, 1950) on the suggestion of the Soil Mechanics Section of the Building

Research Station. The vane was developed by Evans et al (1948). A laboratory vane apparatus was also constructed.

A vane with six blades was used in Canada by A.W. McLaughlin in 1941 during the construction of the Welland Canal. This vane was also used to study slides along the Beauharnois Canal in 1942 and in Sault St Marie in 1943, as reported by Lea & Benedict (1953). They mention that "In Canada considerable work has been concentrated on the vane's use for military purposes".

In an article in the Swedish technical journal *Väg- och Vattenbyggaren,* Bergfelt (1956), at that time head of the Design Section at Gothenburg Harbour, wrote "We have therefore designed a new vane borer which can be directly combined with wash borings. The saving in time is, of course, large for deep holes. In the design we used several details derived from experience with the small vane borer of the Harbour of 1939–41 and the well-known type of 1942 of the Swedish Geotechnical Institute." The existence of this "small vane borer" was a well kept secret, since it was not even mentioned by Cadling (1948) nor by Cadling & Odenstad (1950); the date of 1942 mentioned by Bergfelt is a printing error. Bergfelt (1977) indicates that the 1939–41 vane borer, "which probably had *circular* blades", was designed and constructed by the late Mr. Nils Engström, then head of the Design Section at Gothenburg Harbour. The prototype was probably lost in a fire in 1963. It is possible that drawings of the vane borer can be found in the files of the Gothenburg Harbour or in one of its workshops. In a letter to the Senior Author of this chapter, Bergfelt (1977) stated that he was now attempting to trace the vane. It will be interesting to see if additional information will become available on this early Swedish vane borer with circular blades.

A German patent (508 711) from 1929 on a vane borer is mentioned in the literature (Cadling & Odenstad, 1950). There is no information about its use in practice. It is also said that Carl Forssell, at that time Professor at the Royal Institute of Technology in Stockholm, who is known for his early investigations into the buckling of piles (Bjerrum & Flodin, 1960), demonstrated a vane at the Third International Congress for Applied Mechanics in Stockholm in 1936, although this is not mentioned in the *Proceedings* of the Congress. One of Professor Forssell's assistants at that time (Mr. Gregor Aminoff) indicated in October 1977 that a helical screw, which was screwed down to different depths inside a casing, was used instead. The helical part was loaded with weights to determine the soil resistance (see the so-called A-sounding Method in Section 1.5.5).

The very first vane borer, as far as can be determined, was designed by John Olsson, the secretary of the Geotechnical Commission. It was used in 1919 (not in 1928 as indicated in some references) during the construction of the Lidingö Bridge in Stockholm, built between 1917 and 1926 (Nordendahl, 1928). John Olsson was in charge of the geotechnical investigations. One of the problems at this site was buckling of the long piles needed to support the

bridge. The soil consisted of very soft clay with a thickness of up to 40 m. John Olsson was given the task to determine the coefficient of horizontal sub-grade reaction of the clay. For this purpose, he designed the vane borer shown in Fig. 1.23. It consisted in principle of two blades which were attached to a system of pipes and a torque wrench. In order to compensate for the friction, two tests were made at the same depth with two plates of different sizes. The widths of the plates were 300 mm and 700 mm, while the height was the same (500 mm) in both cases. Both the torque and the rotation were measured. From the difference in the two tests, the load—deformation relationship could be determined for a nearly horizontal movement of a 500 mm high plate with a width equal to the difference between the two plates. The two tests thus gave directly the coefficient of subgrade reaction in-situ under conditions which were closely related to those of a pile.

Olsson's tests were carried out during the winter from ice at depths of 3.4 and 5.4 m below the sea bottom. It was found that the rotation rate was important and that the displacements could be large if the load was applied

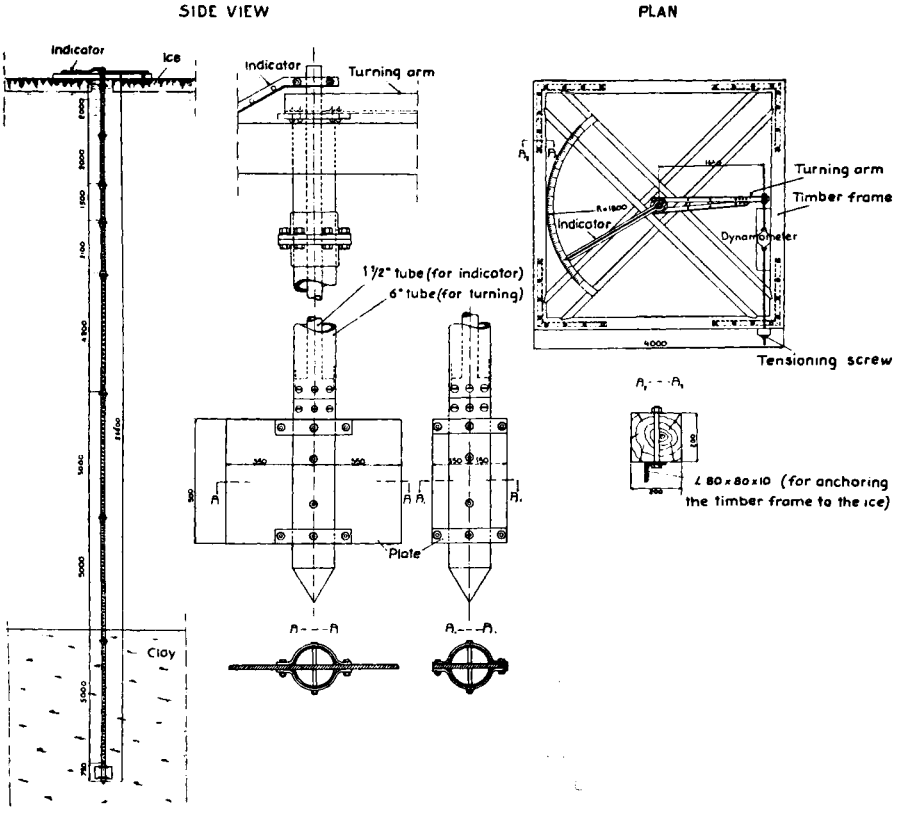


Fig. 1.23. The first vane borer, constructed by John Olsson in 1919.

very slowly (Nordendahl, 1928). The author of the report concludes: "The apparatus was found to be very well suited for its purpose and it is highly desirable that further investigations of the strength of the clay should be performed with the method now described or by a similar method."

1.5.4.2 The Cadling vane borer

The vane borer as it is used today was presented for the first time by Lyman Carlsson (Cadling) in 1948 at the Second International Conference in Rotterdam (Fig. 1.24). A report on a more advanced device was published two years later (Cadling & Odenstad, 1950). The original Cadling vane borer, which was designed for soft soil, was pushed into the soil without preboring. The rod was encased to eliminate friction, and the torque required to rotate the vane was measured at the ground surface by a separate instrument. From the torque and geometry of the vane, the shear strength of the soil could be calculated. The blades were made as thin as possible to reduce the disturbance when the vane was pushed into the soil. The vane was initially unprotected. Vane borings in Horten, Norway, in 1948, with the Senior Author as field engineer, indicated, however, that it was necessary to protect the vane because

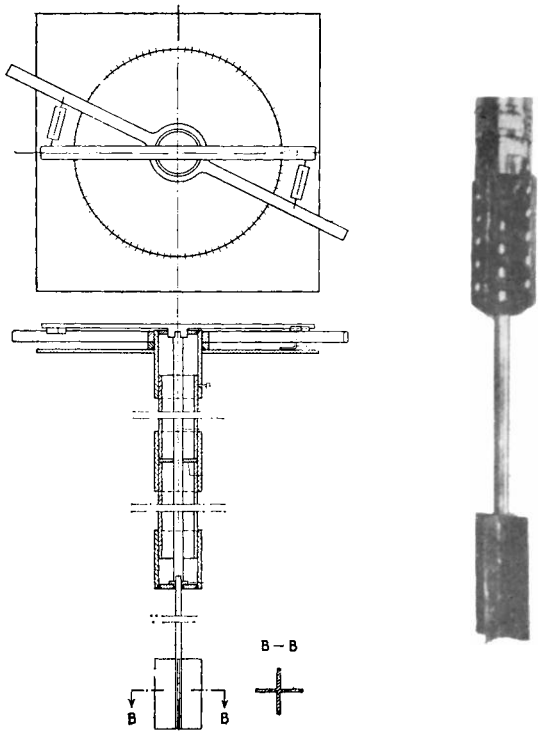


Fig. 1.24. Cadling's original vane borer of 1948, and (right) the protective sheath.

of stones in the clay (Brinch Hansen, 1950). A protective sheath was added, therefore, and has been included since then.

Investigations by Cadling during the development of the vane borer indicated that the vane should preferably have four blades to reduce the disturbance and to give a relatively uniform stress distribution along the rupture surface around the vane. The rotation rate should be about 0.1° per second to correspond to the loading rate for unconfined compression tests. Cadling proposed that the height of the vane should be twice the width and that the shear strength of the remoulded soil, and thus the sensitivity, should be determined after at least 20 revolutions of the vane. The standard Cadling vane had a height of 130 mm and a diameter of 65 mm. A vane for very soft clay (H=160 mm, D=80 mm) and one for stiff clay (H=110 mm, D=55 mm) were also used.

The modified 1950 vane borer was provided with a recorder which indicated the relationship between torque and rotation of the vane. Later models for routine tests were also provided with an indicator which showed the maximum applied torque.

1.5.4.3 Later vane borers

The Cadling vane borer was immediately accepted by the geotechnical world. Vane borers based on the same principle were built by, amongst others, Bishop and Skempton and used by Skempton (1948d) to investigate a thick deposit of soft silty clay near Grangemouth, England. The tests were carried out in cased holes. The shear strength as determined by unconfined compression tests was almost constant with depth, while the vane tests indicated a definite increase in the shear strength with depth below about 15 m. The maximum depth of the tests was about 33 m.

Brinch Hansen (1950) also recognized the possibilities of the vane borer and used it in the autumn of 1948 at Horten, Norway in a soft quick clay ($S_t \approx 100$), as mentioned above. The measured shear strength was essentially constant down to about 7 m, below which it increased linearly with depth; the s_u/σ'_{vo} ratio was 0.18. The shear strengths from fall-cone and undrained shear tests below 15 m were substantially lower than the vane values.

Vane borers were constructed in USA by Vey & Schlesinger (1949) and in Canada by Lea in 1950 (Lea & Benedict, 1953). Two symposia on vane testing were held in USA in 1956 and 1965 (*ASTM,* 1957, 1966). Of special interest from the first meeting are the results on the sensitive Leda Clay in Canada. It was concluded, for instance, that the best application of the vane borer is in soft uniform clays with a shear strength less than 100 kPa (Eden, 1966) and that vane tests should be supplemented by sampling. A simple six-bladed pocket vane, later known as the *Torvane,* was described at the second symposium (*ASTM,* 1966). In early investigations of Leda Clay, the in-situ vane strengths exceeded those given by laboratory unconfined compression tests, the uncon-

fined values being as low as one-third to one-half of the vane values (Hurtibise & Rochette, 1956; Eden & Hamilton, 1957).

Terzaghi (1953) pointed out that the vane borer, like other refined soil exploration tools, can only be used under certain more or less defined conditions.

Another Swedish vane borer, the *Nilcon* type, was designed by Nilsson (1958). It has no casing or sheath for the protection of the vane blades, and the blades are relatively thick and bevelled.

1.5.5 Other in-situ methods

One of the first apparatuses, besides the vane borer, which could be used to determine the shear strength of soft soil in place was the *in-situ machine,* later called the *Iskymeter,* developed by Kjellman (1943) (Hvorslev, 1949). The apparatus consisted in principle of two blades *(resistor)* which could be folded and forced down into the soil to the desired depth using a 19 mm sounding rod. The rod was then slowly pulled using a wire, whereby the arms of the resistor unfolded, as shown in Fig. 1.25. The pull-out resistance was recorded at the ground as a function of depth. The area of the open resistor was either 100 or 200 cm^2. With this method, even very thin layers of sand in a soft soil clay could be detected. It was, however, necessary to calibrate the apparatus at each test site. The method was especially useful to determine the location of the slip surface after a landslide had occurred, as in the Göta River Valley in Sweden during the stability investigations in the 1950s and 1960s (see Section 1.7.2.3). The Iskymeter can be regarded as a sounding device.

Another in-situ method which was developed in Sweden particularly to measure the shear strength of soft clay is the so-called A-sounding Method

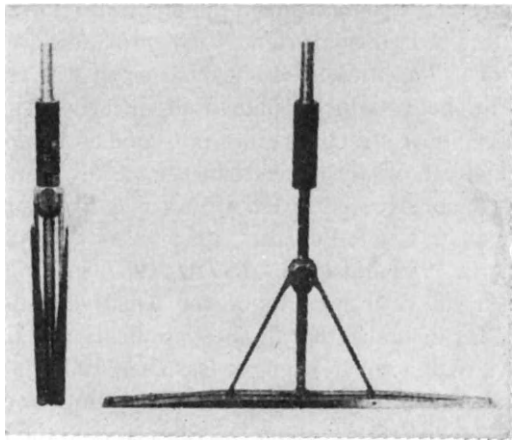


Fig. 1.25. The resistor of the Kjellman Iskymeter in (left) folded and (right) unfolded position.

(Sökjer, 1961). The shear strength was determined with a helical auger with wide, thin flanges by slowly pulling the helical screw a certain distance and by measuring the required force (see the method by Forssell above). The machine could also be used for mechanized weight soundings and sampling. The clay which was trapped between the flanges was disturbed surprisingly little and could be used for routine laboratory tests. For unknown reasons, this universal machine has not been extensively used in Sweden or elsewhere.

1.5.6 Sampling

Many different types of soil sampler and auger were used during the 19th century, as can be seen in Fig. 1.19. Some of them originated during medieval times. The samples were of the 'can' type or were open longitudinally. Many were formed as, or were provided with, screws. The samples obtained were disturbed.

The side intake sampler shown in Fig. 1.26 was described by the Swedish Geotechnical Commission (1922). The sampler, which has a longitudinal slot, is filled with soil when rotated. It has been used extensively in both peat and soft clay in northern Europe since about 1860 (Haglund, 1909; Fries & Hafsten, 1965). The sampler was probably invented by the Norwegian biologist Asbjörnsen (1868) and was called the *Asbjörnsen Peat Sampler* or the *Norwegian Chamber Sampler*. Other names are *Blytt's Sampler* (in Germany), *Krüger's Sampler, Fredriksson's Sampler* and *Hiller's Sampler*. Today it is perhaps best known as the *Peat Sampler* and is mainly used by geologists and soil engineers for inspection purposes because of its low weight.

The first piston-type sampler was invented by Kellgren (1894) and was used for shallow sampling, primarily in peat but also in soft clay. It consisted of a

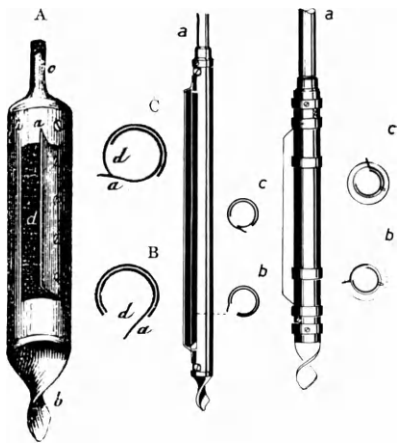


Fig. 1.26. The Asbjörnsen soil sampler of 1868.

1.5 m long steel tube with 30 mm inside diameter. The inner rod could be fixed while the tube was screwed down. Only disturbed samples could be obtained because of the high length to diameter ratio.

When the Swedish Geotechnical Commission started its work in 1914, it was not possible to obtain undisturbed samples of soft soil, and one of the tasks of the Commission, therefore, was to develop a sampler for this purpose. Many types were tried, including a square sampler with a shutter (Caldenius, 1925). It was not until 1923 (J. Olsson, 1925), however, that the first modern sampler with a stationary piston was ready. It was designed by John Olsson with the assistance of Caldenius (1937), who had earlier worked for the Geotechnical Commission as a geologist. The 900 mm long sampler, with an inside diameter of 44.5 mm and a total sample length of 640 mm, is shown in Fig. 1.27. The sampler was pushed down to the desired depth without

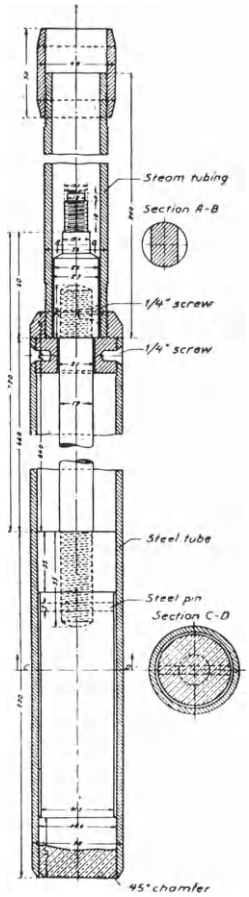


Fig. 1.27. The first piston sampler, designed by J. Olsson in 1923.

preboring. The sampling head was then forced down while the piston was locked in position. The sample was kept in the sampler during withdrawal by the vacuum caused by the airtight piston. This sampler was used extensively in Scandinavia and served as the prototype for later piston samplers. After the sampler had been developed, it became possible to investigate systematically the stability of slopes based on the undrained shear strength of the soil as determined by the fall-cone test ($\varphi=0$ analysis).

Samplers in the Nordic countries are generally pushed down manually in soft clay. In other regions, sampling is commonly done in predrilled holes. However, it is generally not possible to obtain truly 'undisturbed' soil samples when a sampler is pushed down, as indicated by Casagrande (1932b) who compared samples taken with open tubes (tube samples) and hand-carved samples (block samples). (See also Terzaghi, 1941; Mohr, 1943; Rutledge, 1944; Terzaghi & Peck, 1948; Fadum, 1948; Hvorslev, 1948, 1949; Kallstenius, 1963.)

Caldenius (1938) studied the disturbance of the ends of samples obtained with John Olsson's piston sampler and other samplers, as well as the disturbance caused by the transportation of the samples. It was found that the new piston sampler improved considerably the quality of the samples. The handling of the samples taken with this early sampler, however, left much to be desired by today's standards. The long samples were forced out of the sampler in the field like a long sausage and cut into pieces, as a rule 80 mm in length. The specimens were placed in glass jars and sent to the laboratory. The disturbance, especially of quick clays, caused by the handling must have been great.

The handling operation was improved and the disturbance was reduced with the new sampler which was constructed for the subsurface investigations of the San Francisco to Oakland Bridge (Beatty, 1932). A thin 50 mm outside diameter brass tube was placed in the sampler so that the tube with its sample could easily be removed. The sample tube was then sealed and sent to the laboratory. There, the sample tube could be cut longitudinally and the sample removed undisturbed.

The idea of a brass tube was adopted by the Gothenburg Harbour (Petterson, 1933). A modified piston sampler was constructed using a 600 mm long and 49 mm inner diameter brass tube. The sample was cut into three 200 mm long pieces which were sealed and sent to the laboratory. Later, both the Gothenburg Harbour Sampler (42.5 mm sample diameter) and the Swedish State Railways Sampler (of John Olsson) were provided with several short brass tubes which were placed in line in the sampler head. These samplers were used extensively in the Nordic countries.

A new 60 mm piston sampler was designed in 1937 by Kjellman (1942), then head of the Geotechnical Department at the Swedish Road Board (the predecessor of the Swedish Geotechnical Institute). The sampler was relatively

short, and only one 170 mm long sample could be obtained. It was used mainly for routine investigations. Experience showed, however, that the disturbance of the samples could be appreciable under unfavourable conditions (the area ratio was 0.9).

A comprehensive study of sample disturbance was initiated in Sweden in the 1950s by Jakobson (1954) at the Swedish Geotechnical Institute. Inspired by the report by Hvorslev (1949), he investigated seven different samplers. The Swedish Foil Sampler (Fig. 1.28), which had been developed by Kjellman at the end of the 1940s (Kjellman, 1948b; Hvorslev, 1949; Kjellman et al, 1950), was also included in the study. With this new type of piston sampler, undisturbed cores of soft soils of practically unlimited length could be obtained. For example, a 90 m long core was taken for the Tingstad Tunnel in Gothenburg (see Fig. 1.7 and Brink, 1967). The friction between the soil and the inside of the sampler tube was eliminated by means of foils which were rolled off during the sampling process (Fig. 1.28), and an inside clearance was used to allow for some swelling of the sample core. Short specimens could be cut from the core extracted and trimmed to a suitable diameter for testing in the laboratory. A 42 mm diameter foil sampler was later constructed, primarily for geological studies, particularly of varved clays. The sampler could also be used for frictional soils.

A special joint committee was appointed in 1957 by the Swedish Geotechnical Society and the Swedish Geotechnical Institute to develop a Swedish standard piston sampler (Kallstenius, 1958). The samplers which existed at that time in the Scandiavian countries were studied, including a 54 mm piston sampler developed by the Norwegian Geotechnical Institute (Vold, 1956). A 50 mm Swedish Standard Piston Sampler was adopted in 1961, and this standardized sampler is used today (1981) in almost all investigations in Sweden and also, to some extent, in the other Nordic countries and in Canada. The sampler is provided with three 170 mm long tubes, as shown in Fig. 1.29 (Swedish Geotechnical Institute, 1961). Kallstenius (1963) compared the results for different samplers, and he also investigated the influence of storage and transportation of the soil samples.

Large diameter soil samplers have been developed. One such sampler, with 101 mm inside diameter, was constructed by the Norwegian Geotechnical Institute in 1941. Also, a hydraulically operated piston sampler for 120 mm diameter samples is still in use (Osterberg, 1952).

1.5.7. Pore pressure measurements

The first attempts to measure pore water pressures in clay were probably made in connection with the construction of earth dams (Fellenius & Johansson, 1933). Terzaghi (1953) reported that he measured the pore pressure in "a remarkably homogeneous deposit of pre-compressed clay" in 1943; pore pressure gauges were installed at different depths to observe the progress of the consolidation.

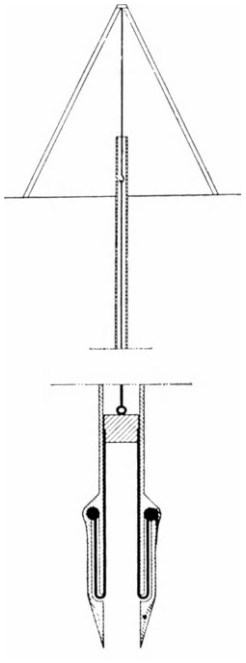


Fig. 1.28. Principle of the Swedish Foil Piston Sampler.

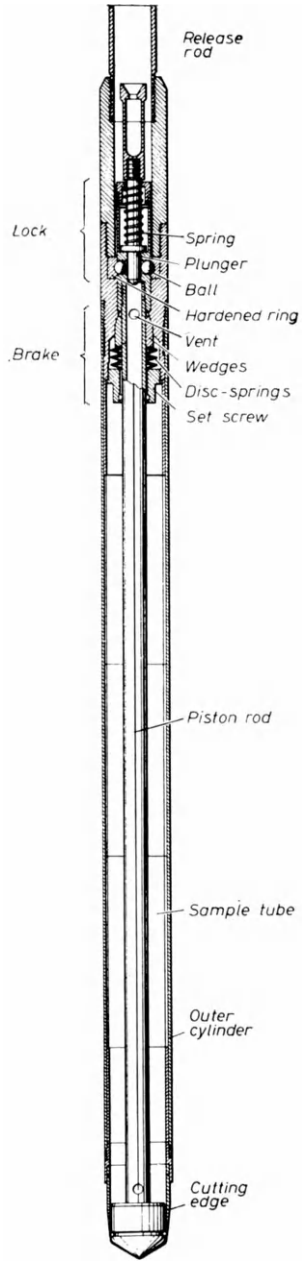


Fig. 1.29. Swedish Standard Piston Sampler of 1961 (sample diameter, 50 mm).

Hvorslev (1948, 1949, 1951) has pointed out that the time lag of piezometers in clay can be considerable. A piezometer for clay with relatively short time-lag was described by A. Casagrande (1946) which consisted of a 13 mm tube connected to a porous ceramic point. Electric pore-pressure devices, so-called *Goldbeck cells,* were used early in the field by the US Bureau of Reclamation (Walker & Daehn, 1948). Other hydraulic pore pressure devices were reported by Cuperus & de Nie (1948).

The need for pore pressure gauges to study the in-situ consolidation process in Sweden was pointed out by Terzaghi in 1946 when he acted as a consultant for the runways of a large airfield in Sweden to be constructed on a 12 m deep deposit of very soft clay at Väsby, located about 30 km north of Stockholm (Chang, 1969). Several different pore pressure gauges were therefore developed at the Swedish Geotechnical Institute (SGI) between 1948 and 1950 by Kallstenius & Wallgren (1956). The first gauge, shown in Fig. 1.30, had a 38 mm tip of porous stone which was connected to a Bourdon gauge. This pore pressure gauge has gradually been improved.

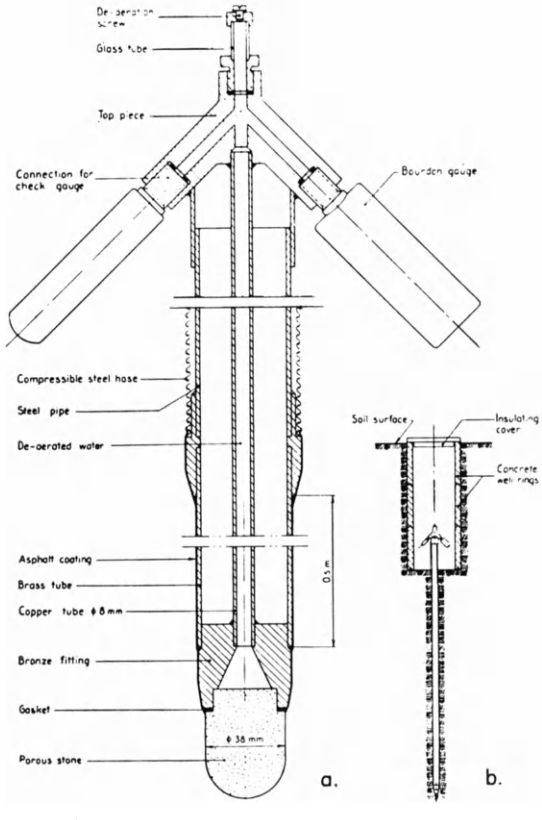


Fig. 1.30. Swedish Geotechnical Institute pore pressure meter of 1948.

1.5.8 Settlement gauges

Systematic measurements of settlements were relatively rare before 1920, and the settlements of structures were generally determined using conventional surveying techniques. The Swedish Geotechnical Commission (1922) developed a precise settlement gauge in which settlements were measured relative to a benchmark founded in firm soil or rock, as illustrated in Fig. 1.31. This settlement gauge has been improved so that it can now be used in landslide warning systems for railways. Slide warning systems are today also used in the Quebec area and in Japan.

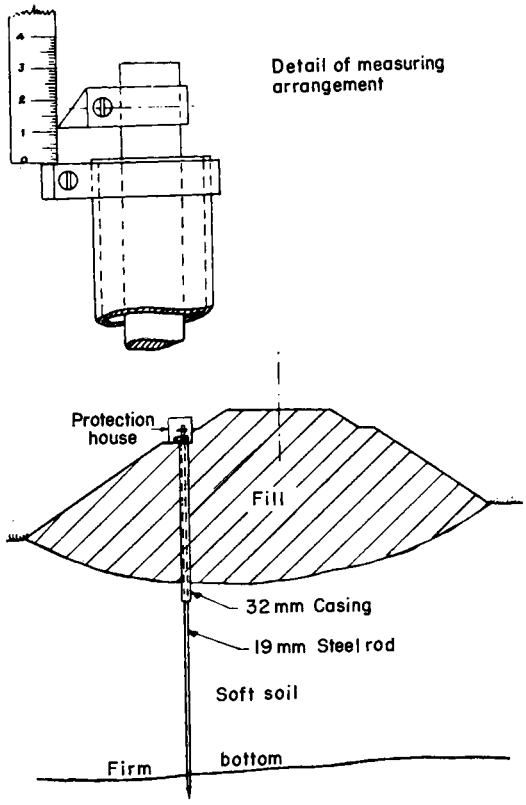


Fig. 1.31. Settlement measurement device used as part of a slide warning system (Swedish Geotechnical Commission, 1922).

1.6 BEARING CAPACITY

1.6.1 Footings

Attempts were made at the beginning of this century to calculate theoretically the bearing capacity of spread footings on cohesive soils. Bell (1915)

calculated the bearing capacity of clay by assuming that failure takes place along two plane surfaces. The ultimate bearing capacity is then:

$$q_u = 4\, s_u + \gamma H \tag{1.10}$$

where γ is the total unit weight of the soil and H is the depth of embedment.

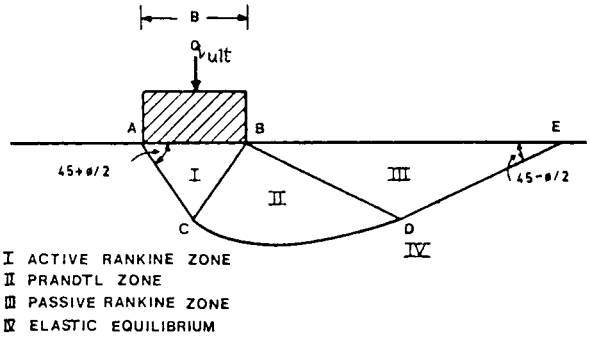


Fig. 1.32. Bearing capacity of an ideal plastic material as postulated by Prandtl (1920).

Prandtl (1920) investigated the bearing capacity of an ideal plastic material under plane strain conditions. He assumed that a wedge of the material below the loaded area moves downwards, and that this wedge deforms the material laterally on either side, as illustrated in Fig. 1.32. In this case:

$$q_u = (2 + \pi)\, s_u = 5.14 \tag{1.11}$$

The ultimate strength of a circular rigid footing at the surface of a plastic material was calculated by Hencky (1923), who obtained:

$$q_u = 5.64\, s_u \tag{1.12}$$

Buisman (1932) extended Prandtl's solution to include the effect of an overburden pressure, q, to give:

$$q_u = 5.14\, s_u + q \tag{1.13}$$

W. Fellenius (1929) calculated the ultimate bearing capacity of a surface strip footing by assuming a circular failure surface, to give:

$$q_u = 5.52\, s_u \tag{1.14}$$

This method was later extended by Odenstad (1941) to a non-uniform load distribution. Bjurström (1944) used a spherical failure surface to analyze the ultimate bearing capacity, considering edge effects, and derived:

$$q_u = 6.33\, s_u \tag{1.15}$$

which is 15% higher than for a circular failure surface as calculated by Fellenius. Ishlinskii (1944) investigated the bearing capacity of a circular footing with a smooth base, and obtained:

$$q_u = 5.68\ s_u \tag{1.16}$$

which is practically the same as that obtained by Hencky (1923).

Meyerhof (1951) calculated the bearing capacity of a deep buried footing as:

$$q_u = 9.3\ s_u \tag{1.17}$$

The ultimate bearing capacity factor is here about 60% higher than that calculated by Hencky (1923) for a surface load.

Theoretical solutions are available today which consider such factors as the slope of the ground surface, the inclination and roughness of the base, the compressibility of the soil, the layer sequence and scale effects. The effect of the modulus of elasticity of the material, for example, was considered by Gibson (1950) using a solution proposed by R.F. Bishop et al (1945); the variation in the bearing capacity was found to be small ($\pm 10\%$).

An early analysis by Skempton (1942) of a footing on soft clay that had failed indicated a bearing capacity factor of 6.7, which agrees closely with that calculated theoretically. Model tests on clay carried out by Golder (1941) gave a bearing capacity factor of 5.1 for a long narrow footing.

1.6.2 Embankments and berms

Loading berms, or pressure berms, were already being used as counter weights in Sweden in the 19th century to increase the stability of embankments. Their effect was initially evaluated from experience (Insulander, 1869), but in 1939 Bjurström presented a method to analyze the stabilizing effects of such berms using a cylindrical failure surface. The influence of different parameters was investigated by Jakobson & Odenstad (1940) and Jakobson (1948) (see Fig. 1.33). In an excellent, comprehensive report by Helenelund (1953) with the title *Stability and Failure of the Subsoil, with Special Reference to Railway Embankments in Finland,* design diagrams were presented.

A dry weathered surface crust can increase considerably the bearing capacity of embankments. The old road and railway builders utilized fully the dry crust, with the potential risk of ground failure. For the Geotechnical Commission, the dry crust was a reality, but its effect could not be evaluated rationally until after the Commission had completed its work. Caldenius (1925) discussed in a report the relationship between soil conditions and the bearing capacity of embankments. He divided the dry crust into three different parts with respect to its shear strength and used in his analysis primarily the results from weight soundings.

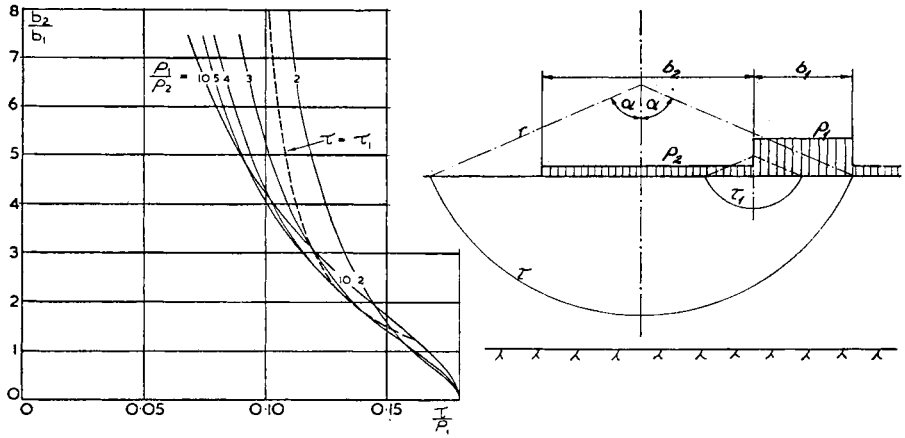


Fig. 1.33. Design of loading berms (Jakobson, 1948).

Helenelund (1953) also considered the role of the dry crust on the bearing capacity, and he recommended that only half the real thickness of the crust should be used in the calculations. He also found that a thin surface layer of peat can contribute appreciably to the bearing capacity of low embankments (see Section 1.9.3).

Piles have been used in Sweden and elsewhere to improve the stability of embankments on soft soils. Asplund (1939) used 1300 poles of only 50 mm diameter to stabilize a railway embankment. The stability was analyzed using a cylindrical failure surface.

1.7 STABILITY OF SLOPES

1.7.1 Calculation methods

Coulomb (1773) was the first to calculate, in his work on lateral earth pressure, the critical height, H_c, of deep vertical cuts in a material with both a cohesion, c, and an angle of internal friction, φ, using a plane slip surface. He derived:

$$H_c = \frac{4c}{\gamma} \frac{\cos \varphi}{1\text{-}\sin \varphi} \tag{1.18}$$

Français (1820) later derived the critical height of a slope inclined at an angle β to the horizontal, as:

$$H_c = \frac{4c}{\gamma} \frac{\cos \varphi \sin \beta}{1\text{-}\cos (\beta - \varphi)} \tag{1.19}$$

It was not possible, however, to use these results in practice for cohesive soils since methods did not exist at that time to determine c and φ experimentally.

For cohesionless soils, the angle of internal friction was estimated from the angle of repose.

Collin (1846) observed from actual slides in clay that the shape of the slip surface was almost cycloidal and that the slip surfaces were polished and striated, and he presented an approximate solution to calculate the stability of a slope using a cycloidal slip surface. Collin's remarkable work was practically forgotten until 1956 when it was translated into English by W.R. Schriever.

Thanks to the Hultin-Petterson slip circle method, landslides and the stability of slopes could, after the failure of the Stigberg Quay in Gothenburg in 1916, be analyzed rationally (S. Hultin, 1916; Petterson, 1916). Hultin based his computations on the assumption that the clay could be treated as a cohesionless material with an angle of internal friction of 9°40′ (see Section 1.9.3 and Petterson, 1955). For the computation of the stability of a quay wall, W. Fellenius used as early as 1910 a method with a slip surface consisting of three straight lines (see Fig. 1.52 later).

The Norwegian engineer Hellan (1917) became interested in the slip circle method and suggested that a cohesion instead of a friction angle should be used in the calculations. An investigation of the Stigberg Quay failure using a plane failure surface indicated an average shear strength of the soft clay of 12 kPa. W. Fellenius presented in 1918 an analysis of the failure at Stigberg using a circular slip surface, where the soil was regarded as a purely cohesive material. To the knowledge of the Authors, this was the first time that a $\varphi=0$ analysis was used in combination with a circular failure surface. John Olsson of the Geotechnical Commission recognized the importance of the analysis presented by Fellenius where the clay was regarded as a purely cohesive material ($\varphi=0$ analysis), and he analyzed a number of old landslides by this method. The calculated shear strength of the soil was correlated with results from fall-cone tests, and good agreement was obtained (Caldenius, 1946; Odenstad, 1947).

W. Fellenius (1918, 1926, 1927, 1929) extended the slip circle method proposed by Hultin and Petterson to soils with both a cohesion and an angle of internal fricton. This method has been known since about 1930 as the *Swedish Slip Circle Method.* He also introduced the concept of safety factors as they are used today. The method of slices was first proposed by Fellenius (1927) and was developed further by Krey (1936). It was assumed that the forces on each side of a slice balanced each other. The Fellenius method of slices was used early on by Cooling & Smith (1935), amongst others.

A modification of the Swedish slip circle method was presented by May in 1936 (May & Brahtz, 1936) in which the graphical construction was simplified, and the effect of hydrostatic pressure was considered. Bjurström (1944) later used a spherical failure surface to take into account end effects.

W. Fellenius (1927) and Taylor (1937) expressed the stability of clay slopes in terms of a stability factor, $N=s_u/\gamma H$. This stability number was found to

depend only on the slope angle and on the depth of the clay layer; for instance, $N=0.18$ for slopes less than 53° where the depth of the clay layer is large. Taylor discussed the effects of such factors as submergence, rapid drawdown and steady seepage. Taylor also described the Friction Circle Method and the Logarithmic Spiral Method as proposed by Rendulic (1935b) to analyze the stability of slopes.

The stability of a slope in soft clay was analyzed in 1936 (Terzaghi, 1936b) from the relationship $\tau_f = c' + \sigma'_f \tan \varphi'$. (According to Terzaghi, φ' varied between 20° and 30°). Comparisons of available test data indicated that it was not justifiable to assume, according to Terzaghi, a shear resistance appreciably greater than c'. It was not until 1948, when Skempton showed that the undrained shear strength of a normally consolidated clay increases with depth, that the $\varphi=0$ analysis was generally accepted. According to Terzaghi & Peck (1948):

> "For computing the factor of safety of slopes on natural clay strata with respect to sliding, the plasticity method is recommended. It is based on the observation that the average shearing resistance along surfaces of sliding in soft natural clay strata is roughly equal to one-half the average unconfined compressive strength of fairly undisturbed samples of the clay taken close to these surfaces".

An analysis by Skempton & Golder (1948) of a failure in soft clay at Huntspill, England, gave a factor of safety of about 0.9 with respect to the undrained shear strength as determined by unconfined compression tests. Skempton (1945) had earlier analyzed a slip in soft clay at the Eau Brink cut and found that the calculated factor of safety was 1.02 with respect to the undrained shear strength, and that the same factor of safety was obtained from a $c'-\varphi'$ analysis.

1.7.2 Landslides

1.7.2.1 General history of landslides

Landslides can occur in practically all types of soil provided that certain characteristic soil, hydraulic and stability conditions prevail. Slides can be divided roughly into rock slides and earth slides, and, with respect to the velocity of the mass movement, into creep and landslides. In his paper *Mechanism of Landslides,* Terzaghi (1950a) listed the processes which lead to landslides. He discussed the different agents which cause landslides (such as water), the events and processes which bring each agent into action (e.g. erosion), the mode of action of each agent (e.g. increase in height of slope, high-frequency vibrations, chemical weathering, rapid change in water-table), as well as such factors as the increase in shearing stresses and decrease in cohesion in the slope. He also mentioned in general terms slides in extra sensitive clays such as those in Scandinavia and Canada, and he further referred to the publications of Ladd (1934) and Seaton (1938).

An example of a large, disastrous landslide, perhaps the largest that has ever occurred, is that in the Shansun Province of China in 1920, described by Close & McCormick (1922). The soil consists in this seismic region of loess (silty and clayey soils). More than 100,000 persons were killed, and an area of about 160 × 480 km, i.e. almost the size of Ireland, was affected. Ten large towns and many smaller towns and villages were swept away, buried or severely damaged. The whole landscape was changed.

The *Historical Catalogue of Slide Phenomena* compiled by Milan Špurek (1972) is an excellent source of information on landslides from a statistical point of view. No less than 649 rock slides and 2205 earth slides are listed from the year 328 to 1970. The work concentrates on slides in Europe; of the 1984 earth slides listed for Europe, Norway with its 'favourable' soil and slope conditions tops the list with 106 slides, while Sweden has 47 slides. None is indicated for Finland, and for Canada only 12 are given: the low figures for these two countries results to a large extent from the fact that there were then few published reports of old landslides in these countries. (Since 1972, however, excellent documents have been published on Canadian landslides; see below). In some cases, the size of the slide area, number of victims, cause of the slide, damage to houses, etc, are indicated in Špurek's work. It is reported that 108,208 persons were killed in landslides between 1345 and 1970, most of which were victims of the Shansun slide.

There are few other indications given in the literature on the disastrous consequences of large landslides. Ladd (1934) mentioned that, within the three years up to 1934, more than 3,000 deaths and very great material losses were caused by landslides. Since the spring of 1931, landslides had led to at least thirteen railway disasters; four of them were in "foreign" countries and nine in the United States; 227 people were killed and 31 were injured.

Bjerrum (1971) stated that, in the period 1871–1940, 17 persons were killed on average every year in Norway due to landslides. At the large slide in Gaudalen Valley south of Trondheim in 1345, several hundred persons were killed and many farms were destroyed. In Canada, the total loss of lives due to landslides exceeds 100 and more than 400 km^2 of farmland have been lost (Mitchell & Markell, 1974).

In Scandinavia as well as in Canada, with their soft, often quick, clays, landslides occur suddenly without warning and last generally not more than 3 to 10 minutes. The only landslides which can occur without warning, according to Terzaghi (1950), are those caused by earthquakes and spontaneous liquefaction. All others are preceded by a gradual decrease in the ratio between the average shearing resistance and the average shearing stress along the potential surface of sliding; this, in turn, involves a progressive deformation of the soil located above the potential surface of sliding and a downward movement of all points located on the surface of the wedge. Terzaghi contended that, if a landslide comes as a surprise to the eyewitnesses,

it would be more accurate to say that the observers failed to detect the phenomenon which preceded the slide. If it is remembered that not all eyewitnesses are soil engineers or geologists, Terzaghi is both right and wrong in this respect since, from interviews with eyewitnesses of many recent large landslides in soft clay, it has been learned that small movements or vibrations in the slide area were observed sometimes several days in advance. The Vaerdalen slide, described in Section 1.7.2.2, is a good example of this.

Old place names in Scandinavia like *Earth Fall*, *Land Fall* and *Clay Fall* indicate that landslides are common in this part of the world. They were once regarded as an act of God and as punishment for a sinful life. Landslides were even considered as part of theological science (!) (Bjerrum, 1971), and valuable reports on old landslides have been found in books of Scandinavian churches. It is of interest that Legget & LaSalle (1978) reported the existence of the place name *Les Eboulements* (Landfalls) in the Champlain Sea area of Canada.

Landslides in soft clay are often triggered by erosion, by vibrations from blasting, pile driving and traffic or by sudden changes in pore water pressure. G. Holmsen & P. Holmsen (1946) and Ackerman (1948b) reported that landslides during World War II were also caused by bombing. Long-term changes in the soil properties can also contribute to failures. In Scandinavia as well as in Canada, reductions in stability are also caused by isostatic uplift. As mentioned previously, seismic activity can also cause sudden landslides, called "earthquake shocks" by Terzaghi (1950a), but such causes are happily rare in the sensitive glacial/postglacial soft clay regions. Some slides in Sweden from the 18th century are documented to have been initiated by earthquakes (Sundborg & Norrman, 1963), as are some in Norway (Wenner, 1951).

It is frequently observed in areas with quick clay that the shapes of the slides are 'bottle-necked' with a narrow mouth at the point where the slide was initiated. One example of this is the landslide at Ullensaker in 1953 (Bjerrum & Rosenqvist, 1956) shown in Fig. 1.34, where the remoulded quick clay flowed rapidly through the mouth of the slide, and pieces of the weathered dry crust floated on the clay slurry. The bottle-neck shape of flow slides in quick clays is also common in Canada (LaRochelle et al, 1970).

At the turn of the century, it was believed by many that the ground underneath a solid dry crust consisted of great basins of liquid-clay suspension. Many old slides in quick clay were attributed to "wet clay" caused by rain water which penetrated the soil (Rosenqvist, 1953). Terzaghi (1950a) did not believe in these theories and did not accept "the lubrication action" of water. It is known, however, that many landslides occurred after heavy rainstorms in the autumn and in the spring when the snow melted (Reusch, 1901; Wenner, 1951; Helenelund, 1958; Jørstad, 1968). The fact that the ground water pressure also increased because of the autumn rains was not fully understood by early investigators.

To prevent landslides, Terzaghi (1950a) suggested drainage and the reduction of excess pore water pressure by vacuum, electro-osmosis, etc.

Fig. 1.34. Typical 'bottle-neck' slide at Ullensaker, Norway, in 1953.

Other possible preventive measures are the reduction of the slope angle and the construction of artificial barriers (such as heavy retaining walls or piles) across the potential path of the moving soil. On this subject, Terzaghi referred to the writings of Ladd (1935) and Seaton (1938).

As can be seen from the preceding sections, it was not until the 1920s that people began to analyze the stability of slopes and landslides more rationally, thanks to a better understanding of the shear strength of soils and to the existence of the Swedish Slip Circle Method. Up to that time, geologists and engineers could only study landslides qualitatively from a morphological point of view to establish the fact that a landslide had occurred; the work of the Swedish Bohus Commission, described in Section 1.7.2.3 below, was of this kind. More recent landslides in Scandinavia (at Tuve near Gothenburg in 1977, and Rissa near Trondheim in 1978) and Canada (at St Jean Vianney in the St Lawrence Valley in 1971) clearly show, however, that the landslide problem is far from solved and that much work remains to be done.

1.7.2.2 Landslides in Norway

There are many descriptions available of old landslides in Norway, especially by the geologist Gunnar Holmsen, who was engaged on this issue from about 1925 to 1955. A systematic investigation into landslides was started in Norway in 1953 by the Norwegian Geotechnical Institute. Its first Director, Laurits Bjerrum (Fig. 1.35), had a special interest in soft clays, and a large number of papers on shear strength and the stability of slopes bear his name.

The two regions in Norway with quick clay which are especially affected by landslides are Romerike (Fig. 1.36), located south-east of Oslo (Bjerrum, 1971), and Tröndelag in the Trondheim area. A 'classical' slide area is also the town of Drammen. As early as 1725, a map was compiled of the many

Fig. 1.35. Laurits Bjerrum of Norway.

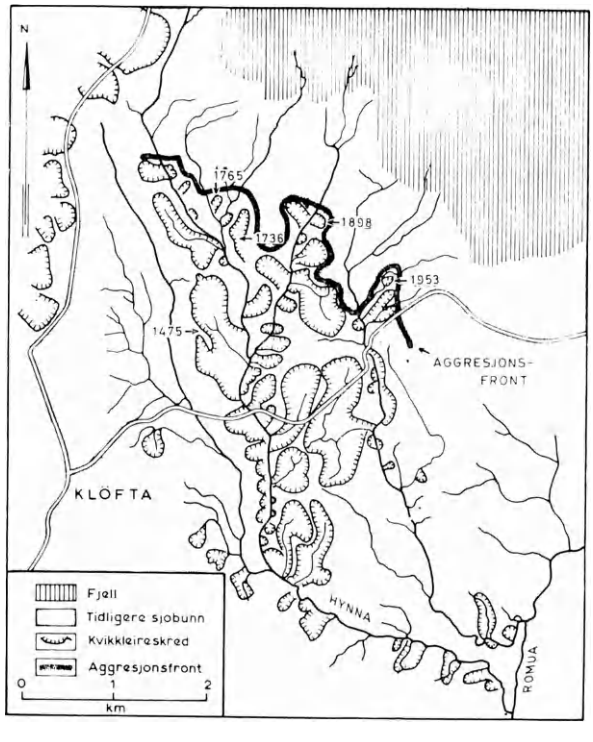


Fig. 1.36. Slide areas in Romerike, Norway (Bjerrum, 1971).

small and large landslides which had occurred in Romerike (Jørstad, 1981). A number of landslides were described by G. Holmsen (1929, 1934, 1946a, 1946b). Also, the contributions by Helland (1896), Friis (1898), Reusch (1901), F. Vogt (1943) and Løken et al (1970) should be mentioned, as well as the many papers by Bjerrum (e.g. 1955, 1971), Aas (1970), Bjerrum & Rosenqvist (1956), Jørstad (e.g. 1970) and others. Landslides in quick clays were also described by P. Holmsen (1953), and a large number of case records of slips in clay in and around Oslo were presented earlier by Haukelid (1930).

A slide in 1320 in Romerike was the oldest slide in Norway that had been dated (Løken et al, 1970), prior to that reported by Eggestad (1978) (see below). The oldest slide mentioned in the literature is that in the Icelandic *Skalholt Annals* (Helland, 1894). This landslide, which occurred in the Gaudalen south of Trondheim in 1345, is probably the largest ever recorded in Norway; 250 persons were killed "in addition to travellers and the poor" (P. Holmsen, 1953). A slide in the River Glomma at Sarpen, the *Sarpfossen Slide,* in 1702, when 14 persons and 200 animals died and more than one million cubic metres of clay flowed away, was recorded in a poem by a contemporary poet. The height of the slide scar was up to 34 m, and the average thickness of the slipped mass was 10 to 15 m. The greatest length and width of the slide were 380 m and 340 m respectively (Nygård, 1944).

Another disastrous landslide in Romerike was that at the Tesen Waterfall *(Tesen-fallet)* in 1795 when the River Vorma was blocked completely for 111 days. A trench was excavated through the slide mass to make the site conditions normal again (Jørstad, 1981).

G. Holmsen (1927) described in an interesting report the slides along the Drammen River at Bragarnes in 1923–24. A slide from 1765 is also mentioned in this report. Seventeen slides were reported to have occurred in Drammen between 1749 and 1935 by Špurek (1972). As mentioned previously, a special committee was appointed in 1838 to investigate the cause of the slides and to prevent further failures. A permanent Royal Commission was appointed in 1864 which was also concerned with the problems in Drammen.

Fascines and crushed rock were used successfully at Bragarnes in 1812, probably for the first time in Norway, as erosion protection and to strengthen the river banks. Relief wells were utilized for the first time in 1930 to reduce high artesian pressures.

The best-known landslide in Norway is that at Vaerdalen near Trondheim which occurred in 1893. This landslide has been described by Reusch (1901), amongst others, then Director of the Geological Survey of Norway, by Friis (1898) and P. Holmsen (1953). The slide, which was very dramatic, lasted about 30 minutes. Clay flowed down the valley like water "faster than any horseman can ride". 112 persons lost their lives and 16 farms were destroyed. The slide covered a 2.9 km^2 area and the displaced mass was about 55 million m^3. It was reported that horses and cattle were restless just before the slide and that a smell of sulphur was noticed. The same smell has also been observed

at landslides in Sweden (Järnefors, 1957) and in Finland (Helenelund, 1958). There were rumours that something 'mysterious' had been going on in Vaerdalen. A year before the slide, a peasant had put down a crowbar in one of his fields, and when he returned after a while the crowbar had sunk. This was a sign to him, and he sold his farm, a wise decision as it transpired.

Reusch (1901) was fully aware of the central role played at Vaerdalen by the quick clay. He found that quick clays occurred in layers or lenses (see Sections 1.3 & 1.4.1). A photograph of a painting of the Vaerdalen slide is shown in Fig. 1.37. There were many dramatic experiences during the slide, and people floated in the clay slurry or seated on the roofs of moving houses. Some people outside the slide area put on their skis, so that they were able to walk on the slurry to look for buried people, and a few persons were saved by these 'skiers', who had to move around the whole time in order not to sink with their skis into the 'soup'.

Fig. 1.37. The slide at Vaerdalen in 1893 (from a painting by W. Anker).

A very recent paper by Eggestad (1978) about a prehistoric landslide has added much to the understanding of slides in quick clay. The slide occurred in the Grorud Valley, 5 to 10 km from Oslo. The mass that moved was estimated to be 30 to 40 million m^3, which is one of the largest slides that has ever occurred in Norway. Samples taken in the area during recent times have frequently shown twigs and other plant fragments in the marine clay deposits. In 1970, a large trunk of spruce was observed in a 2.5 m deep excavation in soft blue clay, and it was assumed that the trunk had been buried in the clay by a landslide. A sample of the trunk was dated in Oslo by the C^{14} method and found to be $8{,}310 \pm 110$ years old. In 1973, a well-preserved birch trunk

was found 500 m from the spruce trunk and at the same depth. A few metres from this trunk, an organic layer was found which had an inclination of about 45°. The tree was dated, this time in Stockholm, to be 8,330±120 years old. A third (undated) hazel trunk containing a quantity of nutshells was found in the neighbourhood at a 2 m depth in a 10 m clay layer. A fourth find of a birch trunk, about 4 km from the site of the other finds and at a 2 to 3 m depth, was dated as 8,265±135 years old. It was obvious to Eggestad that all these finds were part of the debris from a landslide, and the explanation of the interesting fact that all of the finds had been made at very small depths is probably that the trees had floated in the liquefied clay, which has extremely good preservative properties.

A spectacular landslide in quick clay occurred on 29th April 1978 at Rissa, just north of Trondheim (Gregersen, 1980). The slide, which was initiated by a small excavation in a farmyard, progressed rapidly to cover an area of 33 ha, which is the largest clay slide in Norway this century. More than 5 million m^3 of clay virtually flowed out of the area in about five minutes, taking seven farms and five houses with it. A remarkable amateur movie film made of the slide showed that the debris moved at between 30 and 40 km/hr.

1.7.2.3 Landslides in Sweden

Relatively few landslides before 1900 have been documented in Sweden. An exception is the landslide at Intagan in 1648 in the Göta River Valley (Järnefors, 1957). Nathorst (1890) described some relatively small slides in the eastern parts of Sweden, and he mentioned that the Swedish scientist Urban Hjärne wrote in 1702 and 1706 about some old Swedish landslides, including the one at Sarpen (Glomma) in Norway in 1702 (see above). Wenner (1951) collected data about old Swedish landslides and studied especially the climatic conditions at the time of each slide.

Landslides in Sweden have occurred mainly in the south-western parts, near Gothenburg, where deep soft clay deposits of glacial and postglacial origin are common. The sensitivity of these clays is often high (up to 200 or more). Hundreds of old landslide scars are indications of the low stability of the banks along the Göta River, which flows from Lake Vänern to Gothenburg (Frödin, 1919). Slides have also occurred in the north-eastern parts of Sweden, along the Bothnian coast and in river valleys where soft silty organic sulphide clays (Swedish *svartmocka*) are found (such areas are also located in western Finland). Some slides have also occurred in the region of Lake Mälaren.

A 3 to 4 km^2 scar of a prehistoric slide, and probably the largest landslide every recorded in Sweden, was found by Johansson et al (1943) during geological mapping of the area around Lidköping near Lake Vänern. This area is located close to the Rivers Lidan and Göta and near the site where the later landslide at Sköttorp occurred in 1946 (Odenstad, 1951). A clay sensitivity of more than 200 was found at Sköttorp, the highest observed in Sweden up to that time. The slide, which was called the Mariedal Slide *(Mariedalsskredet)* and

was discovered by Johansson et al (1943), has not been dated but is believed to be very old. The slide must have been very dramatic and violent, as indicated by signs in the terrain and by the presence of quick clay at the site.

The large landslide at Intagan (the Swedish word *intagan* means *intake* due to a slide) in the Göta River Valley in 1648 occurred when the river was the border between Norway and Sweden. The failed mass moved across the river to the Swedish side. The Norwegians claimed compensation for the lost land, and they were invited by the Swedes to come over to take it back (Jakobson, 1952)! This slide has fairly recently been studied in some detail by Järnefors (1957). It was verified that the salt content of the clay was low, indicating the extent of the leaching process.

A large scar, 1000×300 m, in the Göta River Valley at a place called *Jordfallet* (Earth Fall) indicates that a huge slide, probably older than that at Intagan, has occurred at this place. The age of the slide has been put at about 1150 by Järnefors by the C^{14} dating method (Sundborg & Norrman, 1963).

The landslide at Bondeström in 1759, and probably some other slides in the Göta River Valley at about the same time, were caused by earthquakes, as documented by Sundborg & Norrman (1963) and others.

Parts of the Göta River Valley were investigated systematically by the geologist Frödin in 1919. He classified the slides as erosion slides, cup-like slides, surface slides, bottle-neck slides and deep slides. This classification was influenced by the system being developed by the Swedish Geotechnical Commission (see below).

As mentioned previously, numerous landslides and ground failures occurred in soft clay and organic soil during the construction of the main railways at the beginning of the 20th century, especially in the western part of Sweden along the so-called *Bohus Line*. A special committee, the Bohus Committee, was appointed in 1908, to study these landslides, but the report of the Committee in 1909 did not contain any recommendations about how to prevent future slides (Westergård, 1910, and Section 1.2.2 above).

The Geotechnical Commission of the Swedish State Railways was appointed officially on 29th December, 1913 after a failure of a railway embankment on 14th June, 1913 at Lake Aspen northeast of Gothenburg. The tasks of the Commission were the same as those of the Bohus Committee, viz. to evaluate the risk of additional slides and to give recommendations to prevent future slides and ground failures. The Commission worked between 1914 and 1920, i.e. mainly during World War I when the boundaries between the countries in Europe were more or less closed. Its *Final Report*, which was published in 1922 (Swedish Geotechnical Commission, 1922), was regarded by Terzaghi (1925a) as one of the milestones in modern geotechnical engineering.

The Commission consisted of two geologists (G. De Geer, Chairman until 1918, and L. von Post) and two civil engineers (W. Fellenius, Chairman from 1919, and F. Blidberg, Technical Director at Gothenburg Harbour). John

Olsson, who was the Secretary, had previously worked on the Bohus Line and had studied, amongst other things, the displacement of soft soil by blasting. Two additional geologists, among them C. Caldenius who became one of the first engineering geologists in Sweden, contributed considerably to the work of the Commission. Others were associated with the Commission, amongst whom Virgin was the first in Sweden to indicate that settlements can be caused by consolidation (see Section 1.4.2).

More than 300 sites were investigated and 20,000 samples were extracted and tested by the Commission. Slides were classified with respect to their cause and type as follows:

(1) *Causes of landslides*

A. Static-dynamic factors (external stability conditions):
   (a) Increase in active load.
   (b) Decrease in counter load.

B. Consistency factors (internal stability conditions):
   (a) Reduction in friction (valid for non-cohesive soils).
   (b) Reduction in cohesion (also so-called *saturation-slides,* Swedish *uppblötningsskred,* are mentioned. This term was possibly used to describe 'flow slides' in quick clay (see Section 1.7.2.5).

(2) *Types of landslide*

A. Simple ground movements.

B. Composite ground movements:
   (a) Progressive slides.
   (b) Retrogressive slides.

Progressive and retrogressive slides were discussed in detail in the *Final Report* of the Geotechnical Commission. Multiple rotational slides were described as follows:

> "It is only for the first slide that the cause of the slide is sufficient to initiate a slide. When the first slide has occurred, the next slide is released as a consequence of this, and so on".

Kjellman (1954b) modified somewhat this classification on the basis of some later Swedish landslides.

The slides investigated by the Commission were analysed by the slip circle method proposed by T. Hultin (1916) and Petterson (1916) and later modified by W. Fellenius (1918, 1927, 1929). This method of analysis, the Swedish Slip Circle Method, contributed greatly to the success of the Commission. One of the slides which was investigated by the Commission was the Getå slide *(Vita Sikudden)* in 1918, near the town of Norrköping. A railway embankment constructed on soft clay failed just as a train passed the area. In this railway accident, the most severe ever in Sweden, 41 persons were killed and many were injured. A slide warning system was installed after the

railway line had been restored. After this tragic landslide had occurred, the safety regulations of the Swedish State Railways were made more stringent and resulted in additional investigations in zones deemed to be questionable.

Of special interest in connection with old landslides and the work of the Swedish Geotechnical Commission is the 500 m long railway embankment along Lake Aspen near Gothenburg which forms part of the main Stockholm-Alingsås-Gothenburg railway line. In 1912 to 1916, the line was changed from a single track, founded on fascines and piles, to a double track, and the new line was moved some 40 m out into a lake with a water depth of up to 4 m. The upper 3 to 4 m thick soil layer consisted of a very soft postglacial deposit of muddy *(gyttjig)* silt, below which there was about 2 m of stiff clay resting on 1 to 2 m of very soft glacial clay, below which was sand resting on bedrock. The thickness of the soft bottom clay layer increased towards the centre of the lake. The embankment for the new double track consisted mainly of rock fill, and to make certain that this had sunk to a safe depth, blasting together with heavy test loads were used, and these gave no significant settlements. Inclined penetration tests from outside the rock fill area were also conducted beneath the embankment. In 1920, new penetration tests were conducted, and careful sampling of the stiff clay layer between the two very soft layers was carried out. From this investigation, it was confirmed that the stiff layer was an old dry crust upon which was a 0.1 m thick humus layer with branches and roots. Root-fibres were also found in the dry crust clay. Since humus soil cannot be formed below water, it was clear that this layer, 8 m below the water level, was an old top soil. These findings confirmed the Commission's suspicions that the dry crust had come to its position as a result of an ancient slide. The presence of shells from marine mollusks in the silt layer indicated a fjord from the Western Sea in this part of Sweden at the time of its deposition. In consequence of the land upheaval which had occurred since its deposition, it was assumed by the Commission that the slide took place some 4,000 years ago.

The more recent large landslides in Sweden at Sköttorp in 1946 (Odenstad, 1951), at Surte in 1950 (Fig. 1.38) (Jakobson, 1952; Caldenius & Lundström, 1956; Kerr, 1963), at Guntorp in 1954 (B. Fellenius, 1955) and at Göta in 1957 (Odenstad, 1958) contributed considerably to the knowledge of the factors affecting the stability of slopes in soft clay, especially the role of the pore water pressure (see also Kjellman, 1954 a-d). At Surte, an excess pore water pressure of 70 kPa was observed. It appears, however, that vibrations from pile driving initiated this slide. The gentle slope of the terrain where the slide started is remarkable, but it is not unusual for slides to occur in quick clay slopes of only a few degrees; the Aasrum slide of 1942 is reported by Rosenqvist (1978) to have progressed on a slope of only $\frac{1}{4}$ degree.

A committee was appointed in 1952 after the slide at Surte to investigate the stability of the whole Göta River Valley. The investigation was carried out by the Swedish Geotechnical Institute (SGI) and by the Geological Survey of Sweden (Statens Offentliga Utredningar, 1962). The evaluation of the risk

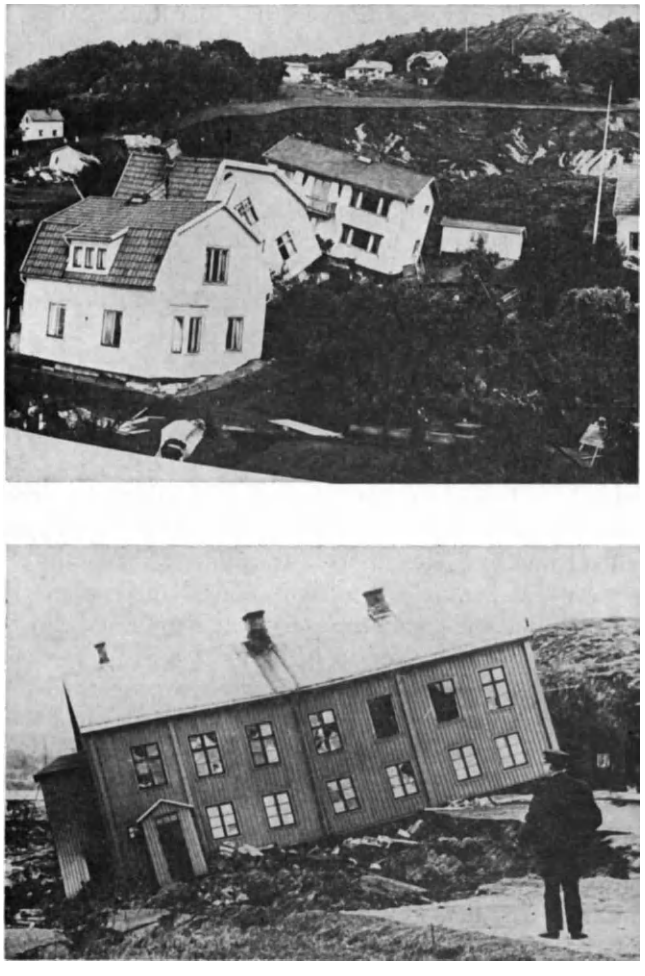


Fig. 1.38. The Surte slide of 1950 (above), and the slide in Sannegård Harbour, Gothenburg in 1911 (photograph from files of W. Fellenius).

of erosion of the river bank in the future was also included in the investigations, as reported by Sundborg & Norrman (1963), and published in a separate report. This report also has a chapter of historical interest on old landslides along the river. Some parts of the valley were found to be unstable, and it was recommended that further erosion of the river banks should be prevented in certain parts and that the slopes should be unloaded by excavation where the factor of safety was less than 1.3. No serious slides have occurred since then. The valley is inspected every year, and the pore pressures and movements of the slopes are checked in sensitive areas. It is not permitted to construct any new structures along the river without the prior approval of SGI.

In connection with the investigations in the Göta River Valley, Söderblom (1969, 1974a) studied quick clay and its role in the development of landslides. He found that phosphates from leaking sewers and septic tanks can reduce the shear strength and thus contribute to slides (see Section 1.4.1 above).

A photo from a slide in the Sannegård Harbour, Gothenburg in 1911 is shown in Fig. 1.38 (see Section 1.9.3 below).

A disastrous landslide occurred on 30th November 1977 at Tuve in the Gothenburg region, close to the site of the Surte landslide of 1950. The Tuve landslide, in which nine people were killed, was the largest this century in Sweden in terms of loss of life and property damage. The volume of the slide was 3 to 4 million m^3, and 27 ha of land were affected.

1.7.2.4 Landslides in Finland

Large landslides are not as common in Finland as in Norway, Sweden or Canada, since areas with extra-sensitive clays (quick clays) are relatively rare. Quick clay areas have been found in the south-western and southern parts of Finland where several landslides have occurred. These regions were investigated in a general way by Helenelund in the 1950s. The factor of safety was in many cases very low, in agreement with Canadian findings (Helenelund, 1959a,b). He found that the shear strength of the remoulded clay even several years after a slide could be as low as 10 to 30 kPa.

The slides at Uskela, Halikko and Nousis were investigated by Helenelund (1958, 1959a, 1959b). These slides were triggered by erosion at the toes of the slopes, which in some cases were as high as 20 m. The main reason for the slides was, of course, the low initial strength of the clay. The size of each slide was found to be governed by the sensitivity of the clay, which in general was less than 60; in a single case the sensitivity exceeded 100. Also, high artesian pressures were recorded. The rate of isostatic land upheaval is in Finland about the same as in Sweden. This upheaval has reduced the stability of slopes and contributed to a change in the soil properties by, for example, leaching and weathering. 'Bottle-neck' slides are rare.

The best known slides in the Uskala region are those along the Salo and Halikko Rivers. The first known slide occurred in 1580 in Yttelä. Many other slides between 1683 and 1853 have been documented. An area exceeding 100,000 m^2 (420×330 m) was affected by a slide in 1827 at Veitakkala, which is probably one of the largest ever to have occurred in Finland.

A recent paper by Niini (1977) reviewed landslides in Finland from an engineering geology viewpoint.

1.7.2.5 Landslides in Canada

During the last few decades in Canada, there has been an increasing interest in landslides, especially those concerning so-called *earth flows* or *flow slides* in quick clay. These slides occur mainly in the Champlain Clay deposits along the St Lawrence and Ottawa Rivers (Fig. 1.8) (Gadd, 1957, 1960; Hurtubise

et al, 1957; Crawford, 1961, 1968; Karrow, 1961; LaRochelle et al, 1970; Mitchell & Markell, 1974). Recent studies in Canada indicate that the many landslides are a result of the unfavourable ground water conditions; a high downward gradient at the top of a slope and an upward gradient at the toe results in artesian pressure, which causes leaching of salt and a subsequent decrease in shear strength, i.e. in the formation of quick clay. The unfavourable flow of the ground water is largely because of the steep dip in the bedrock which forms an abrupt and deep valley (LaRochelle et al, 1970).

The oldest account of a slide in the Champlain area is that of 1840 at Maskinonge, described in 1842 by Sir William Logan (1842a), the first Director of the Geological Survey of Canada. A sketch had been made earlier by Logan (1842b) of a retrogressive slide (flow slide) which occurred on the Nation River near Ottawa; this sketch (Fig. 1.39) was found much later by Legget (1979). One of the largest landslides in Canada in soft clay occurred at St Alban in 1894, when about 20 million m³ of soft clay were displaced (LaFlamme, 1894). Another Canadian flow slide that was described early is

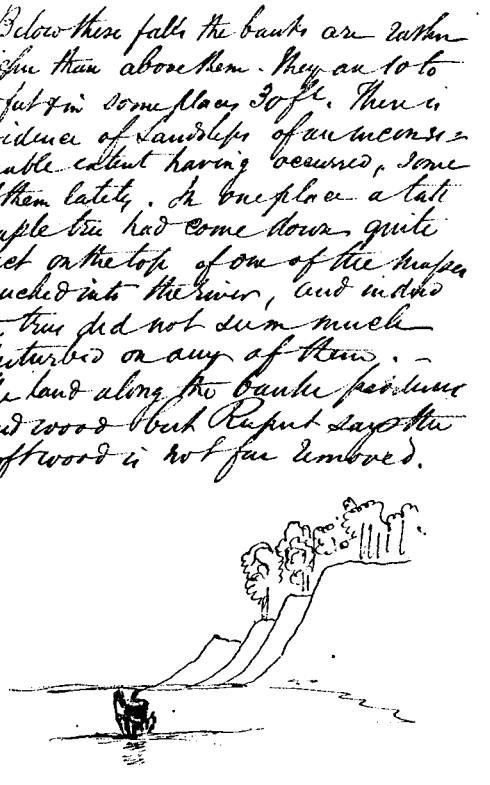


Fig. 1.39. Page from Logan's 1842 field notebook showing the Nation River landslip (Legget, 1979).

that at St Thuribe in Portneuf County, Quebec, in 1898 (Dawson, 1899) which affected an area of about 600×1000 m. An old, disastrous slide in the Champlain area, which claimed 33 victims, was that in 1908 at Notre Dame de la Salette, which removed 2.4 ha of land (Legget, 1973). Legget (1979) mentioned a railway landslide near Trois Pistoles, Quebec, which occurred in 1870.

With reference to the classic earthflow near St Thuribe in 1898, which was later re-investigated by Peck et al (1951), Dawson (1899) described the flow slide phenomenon in quick clays in the following words:

> "It appears probable that in this particular instance the silty-clay, surcharged with water, stood in a condition of unstable equilibrium, retaining its solidity merely by virtue of its unbroken molecular texture, and that at the moment in which it became subject to internal movement this texture gave way and it lapsed into a nearly liquid mass, the particles re-arranging themselves with some freedom in the water previously locked up in its pores."

The largest landslide known to have occurred in the Champlain Sea area took place some 400 to 500 years ago in St Jean Vianney on the north shore of the Saguenay River (see Fig. 1.8). It involved about 21 km^2 and an estimated volume of about 200 million m^3 (LaSalle & Chagnon, 1968). St Jean Vianney was also hit by a disastrous landslide on 4th May 1971, which removed an area of 26.8 ha and 6.9 million m^3; this slide took place within a crater of the earlier, much larger slide. At least 40 homes were carried away and 31 persons lost their lives (Tavenas et al, 1971; Legget, 1973). An eye-witness reported that the slide back-scarp was retrogressing at a rate of about 3 km/hr.

Recent investigations by Legget & LaSalle (1978) date the early St Jean Vianney slide fairly accurately. Before 1941, Legget had been involved in rather extensive investigations for the Shipshaw Power Project on the north shore of the Saguenay River (Legget, 1945; Legget & LaSalle, 1978), during which an almost horizontal peat bed had been found in the Leda Clay. This puzzled Legget very much and gave rise to much discussion among the experts. The high water content of the clay (above the liquid limit) was also confusing and showed, as was not fully understood in Canada at that time, dangerous quick conditions. Some vague suggestions were put forward that the peat appearance in the clay might have been the result of an ancient large landslide. This explanation, however, was dismissed since the area did not show apparent marks of any disaster (no aerial photographs were available at that time). It was in 1965 that LaSalle, during mapping of the area, first concluded that the area had suffered a huge landslide long ago. Two samples of wood from the buried podzol were collected at that time, and the C^{14} method gave the ages as from 560±90 years and 420±90 years. In 1969, a typical sample of the more solid material of the peat bed was obtained and dated as 330 to 140 years old. The cause of the slide was most probably a great earthquake which occurred in the region in 1663. This seems to be definitely confirmed by Marie

de l'Incarnation, who in a letter to her son dated 20th August 1663 gave a vivid description of the event, which completely changed the landscape (Legget & LaSalle, 1978; Marshall, 1967).

Eden (1967) and Crawford & Eden (1967) reported an old landslide at Green Creek near Ottawa City. An 8.5 m deep trench for a sewer was excavated through parts of the apron of the slide and provided an opportunity to examine the former soil profile. A fragment of a tree branch was recovered and dated by the C^{14} method. It was found that the slide had occurred more than 1100 years ago, i.e. about 850 A.D. The original soil profile indicated that very wet soil conditions existed at the time of the slide.

In 1955, a landslide occurred at the Village of Nicolet, when part of the village was destroyed and three lives were lost (Crawford & Eden, 1963). The damage was estimated at $5 million. The calculated safety factor was about 0.6 by the $\varphi=0$ method, and the authors pointed out that other methods of calculation are needed to study landslides of this type (see also Helenelund, 1959). The area of the Nicolet slide was heavily charged with water (Kerr, 1963), a sewer having ruptured a few months before the slide occurred. A similar slide, involving a broken sewage line, occurred at Jordbro near Stockholm in Sweden in 1972 (Söderblom, 1974b).

An extremely valuable analysis of 41 landslides in very sensitive soils (37 in Canada, two in Sweden and two in Norway) has been made by Mitchell & Markell (1974); some of these slides are very old. Dates, places, rivers involved, sizes of areas affected, volumes of masses moved, numbers of victims and references are given in tables, together with soil types, index properties, sensitivities and undrained shear strengths. In another table, the landslide characteristics are given in terms of slope height, depth of failure, height of back-scarp, and stability number, $N=\gamma H/s_u$. It is mentioned in the paper that more than 750 reasonably large landslides (>0.4 ha) have occurred in the Quebec and Ontario regions, and that several thousand small landslides have been identified from aerial photographs. The total area affected by the landslides approaches 400 km^2, and more than 400 lives have been lost in 50 large flow slides.

A compilation of landslides in the Champlain area was made by Chagnon (1968), as shown in Fig. 1.8. Here, the distribution of landslides is indicated as well as the numbers of slides in the specific areas. A map showing major landslides in the St Lawrence and Ottawa Valleys was published earlier by Hurtubise et al (1957).

Klugman & Chung (1976) recently studied the stability of slopes within the Municipality of Ottawa-Carleton, and maps were prepared showing potentially unstable areas. The classifications used by the authors are based on calculated factors of safety, with soil parameters from previous studies used in the analyses. Areas with a calculated factor of safety less than 0.8 were classified as "unsuitable for construction". A review and discussion on the subject of mechanisms and morphology of retrogressive landslides in sensitive sediments in general was published fairly recently by Carson (1977).

Not all landslides in Leda Clay are of the flow type. Crawford (1961) gave an example of a 25 m high bank which failed along a nearly perfect circular arc a few months after the cutting of the slope (Pryer & Woods, 1959). A few metres from this place, a typical flow-type failure occurred when the roof of a rock tunnel failed, allowing about 45,000 m^3 of liquefied clay to flow through the hole (Monaghan, 1954).

1.8 PILES

1.8.1 Pile types and piling equipment

Timber piles have been used extensively for several thousand years to support buildings in soft soils (Leimdörfer, 1974.) The piles were originally relatively short and the driving equipment was primitive. Gradually the length increased and the driving technique improved (Chellis, 1951, where primitive pile driving and early ram drivers are illustrated).

A very old Swedish pile driver used in Stockholm in the beginning of the 16th century is shown later in Fig. 1.49 of Section 1.9.1. Another Swedish pile driver of old origin is shown in Fig. 1.40. It was developed by Christopher Polhem (1661–1751), who is regarded as one of the universal genii of Sweden. This pile driver was used for lock Slussen between Lake Mälar and the Baltic in Stockholm about 1749 (König, 1752). The pile driver was an improvement over older, similar machines. One improvement was the tread-wheel which was driven from the outside and could be made relatively small. The other improvement was the hammer, which was automatically released. With this new machine, the number of men could be reduced from 10 or 12 to 4 or 5.

A pile driver where ropes were used to lift the hammer manually was described by Christopher Polhem's son, Gabriel Polhem (1744). The hammer could be automatically locked and released. Eliander (1753) improved Christopher Polhem's machine during the construction of the Royal Palace in Stockholm in such a way that the mast could be inclined to drive batter piles.

Ropes were generally used to lift the light rams of early pile drivers, which were up to 300 kg in weight. At the end of the 19th century, steam pile drivers appeared with up to 800 kg rams. At the beginning of the 20th century, electrically driven machines were in use, and the weight of the ram was increased to up to 1200 kg. In the 1930s, pile drivers with rams up to 3000 kg were common. About 1950, pile drivers were provided with caterpillar treads to improve their mobility (Bjerking, 1974).

It was something of a building revolution when concrete appeared as a construction material in the latter half of the 19th century. As a material for piles, it was regarded with some doubt at first, but it was very soon accepted in the Scandinavian countries because of its high bearing capacity and its

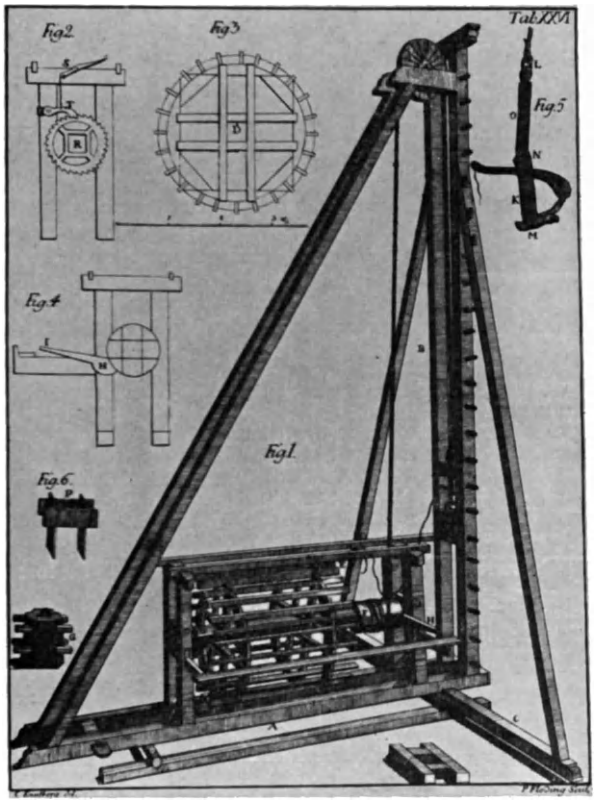


Fig. 1.40. Christopher Polhem's pile driver of 1749 (König, 1752). A model of this is in the Technical Museum, Stockholm.

resistance to marine borers and decay. The first steel piles were driven at the beginning of the present century. Screw piles of wood, steel or concrete were also used (Fig. 1.41); the method used for the installation of screw piles for the Hisingen Bridge in Gothenburg in 1915/16 is shown in Fig. 1.41 (Rogstad, 1977). In the United States, the Raymond step-tapered pile and the Simplex pile appeared around the turn of the century. Hand-excavated caissons (*Chicago* and *Gow* caissons) were common in Chicago and New York at about the same time (Upson, 1923).

In areas with soft clay, it was natural that piling was the main foundation method, especially in Gothenburg, which with its deep soft clay deposits became something of a 'pile town'. Timber piles of good quality could easily be obtained. The allowable load on timber piles in Sweden at the turn of the century was 50 to 80 kN depending on the pile length (10 to 12 m), and these piles were either point bearing or friction piles.

Driven precast concrete piles are today the dominant pile type in the Scandinavian countries. Cast-in-place piles were earlier quite common

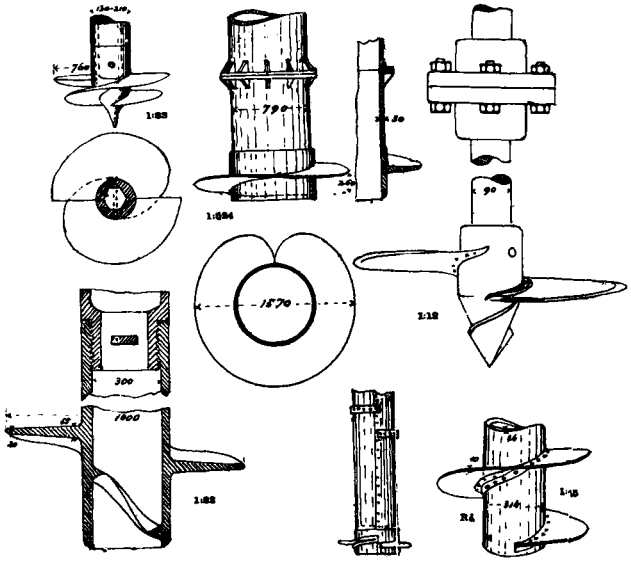
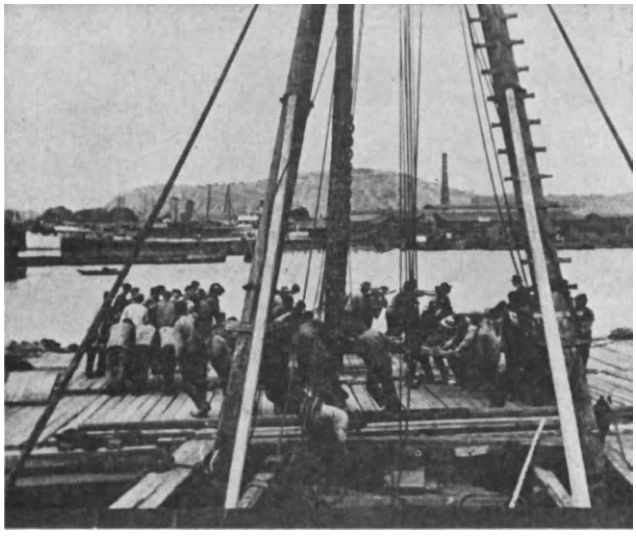


Fig. 1.41. Many types of screw pile existed (Strukel, 1895) . . . .



. . but it could be difficult to screw them down, even in the soft Gothenburg Clay (from the files of Gothenburg Harbour).

(Karlsson, 1912), but they are only rarely used at present. The cross-section of early precast piles was square, triangular or circular, and these were generally driven by 1.5 to 3 t drop hammers with a height of fall of 1 to 2 m or more; a cushion of sawdust was often used to soften the blow (Granberg, 1904). The first precast piles in Stockholm were driven in 1901. The piles, 300 × 300 mm square, were cast vertically as columns. They were driven by a 1.65 t hammer with a height of fall of 0.6 m. The allowable load was 200 kN. Concrete piles provided with steel points were used in Sweden for the first time in 1940 (Petterson, 1941).

The first composite piles, with a concrete top section and a timber bottom section, were driven in Gothenburg about 1935 (Petterson, 1937). Steel H-piles with a point of hardened steel were used in Norway as early as 1936 (Skaven Haug, 1940b). Pile splices have developed from primitive ones for timber piles to sophisticated, relatively stiff splices for concrete piles. Old splices for timber piles are shown in Fig. 1.42.

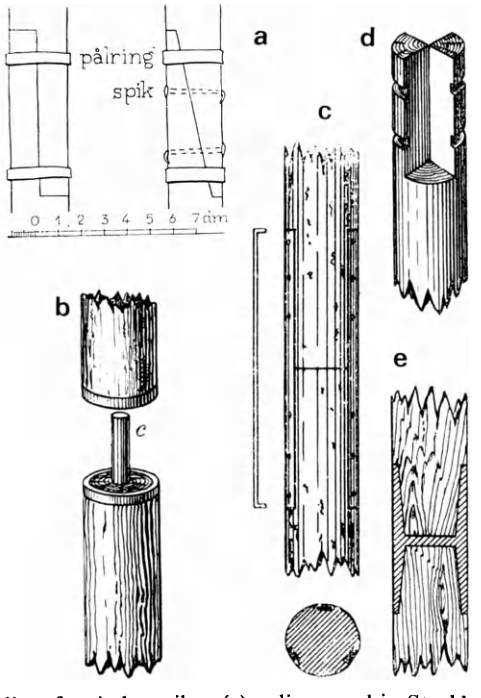


Fig. 1.42. Splices for timber piles: (a) splices used in Stockholm in 1608 (Hansson, 1956), (b) to (e) splices from the 19th century (Kolderup, 1894).

Steel reinforcing bars were used by John Olsson as piles in Sweden about 1915 (B. Fellenius, 1969). Olsson also carried out load tests in 1926 on a 9 m long reinforcing bar of 19 mm diameter in soft clay (Skaven Haug, 1940b) and found that it would support a load of 30 kN without buckling. Reinforcing

bars have since been frequently used in Sweden, mainly by the Swedish State Railways.

The disturbance of soil around a pile caused by driving has been studied by many people. Housel & Burkey (1948) reported that clay close to the ground surface was completely remoulded by pile driving to a distance of at least 80 mm from the pile. Deeper in the ground, the soil was not disturbed or was only partially disturbed. The investigation by Cummings et al (1948) on the effects of driving piles into soft clay should also be mentioned.

For some details of piles and piling equipment from the 19th century, reference should be made to the writings of Broch (1848), Kolderup (1894), Strukel (1895) and Willmann (1906).

1.8.2 Bearing capacity

Christopher Polhem, who was connected, among other things, with the construction of lock Slussen and the Royal Palace in Stockholm about 1740, discussed the bearing resistance of piles. In a book edited by H. Sandblad (1947) which dealt with Polhem's posthumous works from the first half of the 18th century, piling techniques were treated relatively extensively. Polhem wrote (roughly translated from old Swedish) that "one must know three things, namely the weight of the ram, how high to hoist it during the test blow and how much the pile sinks under the blow". Polheim maintained, without obtaining a theoretical relationship, that if one could relate those three factors, then one could get a measure of the bearing capacity of the pile and calculate the number of piles needed. He added that the 'ram test' became more reliable with increasing weight of rammer.

By the turn of the century, a large number of different pile formulae were available in Sweden (Kreüger, 1915). In the United States, the Engineering News Formula (Wellington, 1893) and, later, the Hiley formula (1930) were commonly used. However, Terzaghi (1925a, 1929a) pointed out that the driving resistance of a pile in clay is not directly related to its bearing capacity. The same conclusion was drawn by Petterson (1926a) and Skaven Haug (1940a, 1940b), and pile driving formulae are not used today to predict the bearing capacity of piles in soft clay.

The buckling of piles in soft clay was investigated very early in Sweden. Forssell (1918, 1926) calculated the buckling strength of an initially straight pile assuming that the lateral resistance of the surrounding soil could be replaced by a series of linearly elastic springs *(Winkler foundation)*. He derived the well-known relationship:

$$Q_u = 2\sqrt{(K_h.I_p.E_p)} \tag{1.20}$$

where K_h is the coefficient of horizontal subgrade reaction of the soil and $I_p\,E_p$ is the stiffness of the pile in bending. Forssell (1918) drew the conclusion

that buckling is only a problem in very soft soil. H. Granholm (1929) later extended the work of Forssell. Glick (1948) was the first to investigate the bearing capacity of an initially bent pile, and he found that even a small initial eccentricity reduced appreciably the bearing capacity. Glick also discussed the evaluation of the coefficient of horizontal subgrade reaction. Cummings (1938) also studied the stability of piles against buckling.

Wendel (Fig. 1.43) published in 1900 a remarkable report on the behaviour of 'floating' piles in soft clay. He had investigated the bearing capacity of a series of timber piles in both compression and tension; his apparatus is shown in Fig. 1.44. Both the length and the shape of the piles were varied. Wendel concluded from these experiments that the ultimate bearing capacity is mainly a factor of the adhesion along the sides of a pile, that the point resistance is small and can be neglected, and that the bearing capacity is approximately equal to the pull-out resistance. He also pointed out that, for maximum efficiency, a floating pile should be long and slender. Results from early load tests on timber and concrete piles were also reported by A. Granholm (1905), and these helped to establish at the beginning of this century that the cohesion governed the bearing capacity of piles in clay rather than the angle of internal friction.

Fig. 1.43. E. Wendel of Gothenburg.

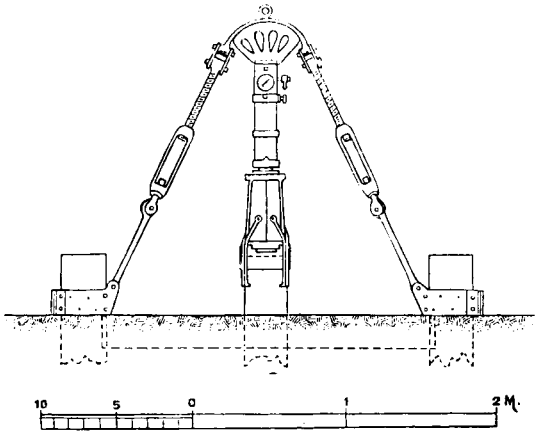


Fig. 1.44. Wendel's pile loading test device of 1900.

Wendel also investigated the increase in bearing capacity with time, and he found that the bearing capacity of the piles increased rapidly after the driving and reached a maximum after two to three weeks. He also studied the effect of loading rate, and measured that the bearing capacity decreased with increasing time of loading, the long-term strength being about 80% of the one-day strength. The lateral resistance was also investigated.

Wendel (1900) examined the heave of soft soil and the lateral soil displacement caused by pile driving. He compared the ground surface heave resulting from piling to an ocean wave which moves in the same direction as the sequence of the driving, and he recommended that piles should be driven randomly in plan. He concluded that the pile spacing, commonly 0.6 m at that time, should be increased to 0.8 m. A. Casagrande (1947) observed during pile driving for the John Hancock Building in Boston that the heave corresponded only to about 70% of the total pile volume and that the difference was probably due to the tendency of the clay to consolidate as a result of the disturbance caused by the driving. Surface heave due to pile driving was also investigated by Arfwidson (1940), who developed a method whereby the surface heave was reduced by precoring with a 12 m long open-ended steel pile of 250 mm diameter which was driven into the ground. When the pipe was withdrawn, compressed air was forced into the soil to relieve any vacuum at the bottom of the pipe. Compressed air was also used after the withdrawal to force out the clay plug trapped in the pipe.

Forssell (1918) found from an analysis of Wendel's results that pile adhesion is first mobilized at the top of a pile and that the shearing resistance along the pile at failure is constant and is independent of the overburden pressure. The same conclusion was drawn by B. Fellenius (1938b) for steel pipe piles. Forssell proposed a method to determine the axial stress distribution in a reinforced concrete pile by measuring the axial shortening. He recommended that a pile group in soft clay should be fan-shaped, in that the exterior piles should be inclined outwards with respect to the piles at the centre of the pile group.

About 6000 floating piles were used for the foundation of the Post Office Building in Gothenburg, which was constructed in 1918–1923 (Petterson, 1926a). Seven piles were tested at different times after the driving. The load tests indicated that the piles should not be tested until three weeks after the driving. After an additional two to three weeks, the bearing capacity had increased further by about 15%. The allowable load on the piles was calculated from the 'adhesion' determined from the load tests, neglecting the point resistance. A factor of safety of 4 was used, and the adhesion was reduced by 25% to allow for the disturbance caused by the driving of adjacent piles.

The bearing capacity of two floating piles, one in quick clay and one in 'common' clay, was investigated by John Olsson at the beginning of the 1920s (B. Fellenius, 1974). The pile in quick clay almost sunk under the weight of the hammer, while a large number of blows was required to drive the second pile in the clay with low sensitivity. The ultimate bearing capacity of the two piles, however, was about the same.

T. Hultin (1928) found from an analysis of load tests on timber piles in soft clay that the shearing resistance increased almost linearly with depth.

Lidén (1934) pointed out the harmful effects caused by short compaction piles driven into the dry surface crust above a deep layer of soft clay. Such

short timber piles were commonly used around the turn of the century in Stockholm. The purpose of the piles was to 'compact' the surface layer in order to increase the bearing capacity and reduce the settlement, but the remoulding of the soil caused by the driving *reduced* the shear strength of the underlying soil and *increased* the settlement.

B. Fellenius (1938b) investigated the possibility of increasing the bearing capacity of floating piles in soft clay by increasing the roughness of the surface. As mentioned above, it was believed at that time that very soft clay behaved as a more or less cohesionless material and that the roughness of the surface of the piles affected the bearing capacity. An angle of internal friction of 21° was frequently used in stability calculations. The test piles, which were provided with 'bands' of shotcrete, had a bearing capacity which was only 60% of that calculated from the undrained shear strength of the clay. This low bearing capacity was attributed to the remoulding of the clay by the bands during driving.

Fellenius also studied the ultimate bearing capacity in soft clay of up to 40 m long pipe piles of 51 and 76 mm diameter. Some of the piles were provided with an end shoe such that the end bearing resistance could be measured separately. The test results indicated that the point resistance was negligible and the skin friction resistance was considerably lower than that of a timber pile of the same shape, in agreement with the results reported earlier by Wendel (1900). Fellenius also found that the pipe piles driven to rock could be loaded up to the yield strength without buckling.

The bearing capacity of floating timber piles was investigated in Norway by Skaven Haug (1940a). He reported that their bearing capacity in quick clay in many cases was so low immediately after driving that it was necessary to hold down the piles for a few minutes to keep them from rising. It should be mentioned that this heave phenomenon was recognized very early; Kolderup (1894), for instance, indicated that special cleats should be used to keep piles in place (Fig. 1.45). Skaven Haug recommended that a timber pile in quick clay should not be tested until 30 days after driving. In order to improve the

Fig. 1.45. Cleats to prevent timber piles from heaving in quick clay (Kolderup, 1894).

bearing capacity of floating piles, they were also sometimes provided with wooden laths which were nailed on the piles to increase the surface area (Bjerking, 1974).

The ultimate bearing capacity of steel, concrete and timber piles in soft clay was investigated by Bjerrum et al (1948). Test data indicated that the skin friction resistance was somewhat higher for timber and concrete piles than for steel piles. Converse (1948) reported that the skin resistance of cast-in-place piles in soft clay corresponds to the undrained shear strength as evaluated by direct shear tests.

The practice in 1948 was to calculate the ultimate bearing capacity of single piles in soft clay from the undrained shear strength of the clay and the perimeter area of the pile. Reliable information could only be obtained from loading and pulling tests in the field (Terzaghi & Peck, 1948). When the spacing of the piles was small, the ultimate bearing capacity of a pile group was calculated as the sum of the skin friction resistance along the perimeter area of the pile group and the end-bearing of the bottom area of the group.

One of the first to investigate the load distribution in pile groups was Gullander (1902). He and T. Hultin (1911) investigated two-dimensional pile groups with randomly oriented piles (Gullander, 1911, 1912). The lateral resistance of piles was considered by Ekvall (1911).

The settlement of a pile group depends to a large extent on the stress increase and on the local soil conditions. Terzaghi & Peck (1948) illustrated the differences between the stress distributions below a raft and a pile group in the manner shown in Fig. 1.46. They pointed out that piles in soft clay are beneficial only when the pile length is at least equal to the width of the foundation. If the length is not larger than the width of the structure, the reduction in the settlement by the piles will be small.

Settlements during and after construction of pile-supported buildings had already been measured at the beginning of this century and much valuable

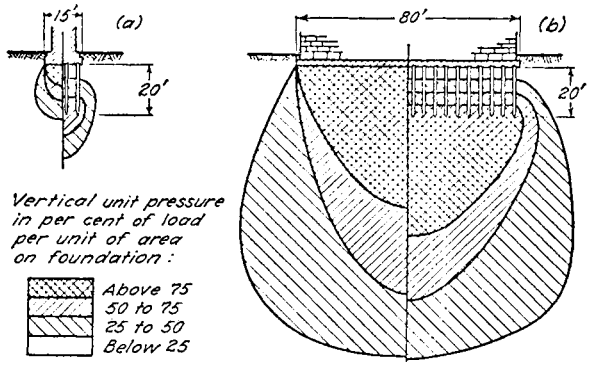


Fig. 1.46. Vertical pressures in soil beneath friction pile foundation for (a) small and (b) large width of foundation compared to pile length (Terzaghi & Peck, 1948).

experience was gained. Petterson (1926a) reported that the settlements of a building constructed on floating piles were the smallest where the pile driving started, while the largest settlements occurred where the last piles were driven, in agreement with the observations reported earlier by Wendel (1900). A. Casagrande (1932b) pointed out that pile driving in sensitive clays can increase the settlements because of remoulding of the soil during driving. Terzaghi & Peck (1948) suggested that the time-dependent settlements of a pile group can be calculated by assuming that the load is transferred to the surrounding soil at the lower third point.

Large settlements (300 mm) caused by negative skin friction were reported by Florentin & l'Hériteau (1948), amongst others. The increase in the pile load and the settlement caused by negative skin friction in soft clay due to a lowering of the ground water-table was pointed out early by Franx (1936). Franx also discussed the effect of negative skin friction on the load that can be allowed on such piles.

The effects of negative skin friction were investigated by Plomp & van Mierlo (1948) in Holland using instrumented piles. Terzaghi & Peck (1948) suggested that the maximum load increase caused by negative skin friction corresponds to the product of the perimeter area of the pile group and the average undrained shear strength of the clay. They further suggested that in order to reduce the negative skin friction, the spacing of the piles should be as small as possible. The most common method to reduce negative skin friction is to coat piles with bitumen, and this method was used as early as the 1940s (Swedish Geotechnical Institute, 1949).

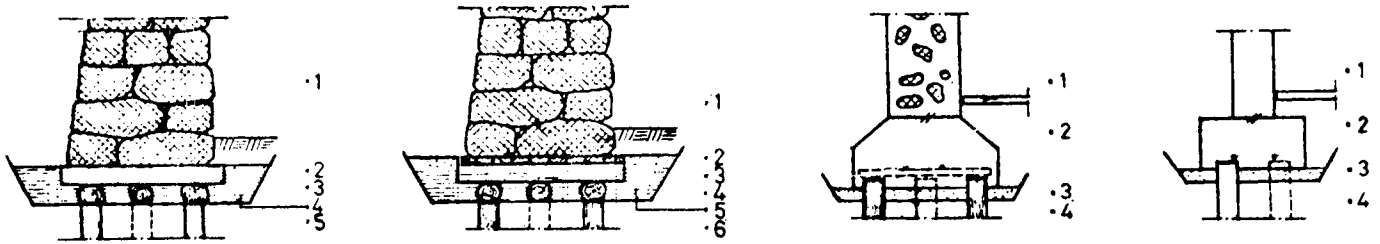
Field tests were performed by L. Casagrande (1937) to increase the bearing capacity of friction piles in clay by means of electro-chemical treatment.

The lateral displacement of pile-supported bridge abutments on soft clay caused by the weight of fill behind the abutments was described by Nord (1936) and by Franx & Boonstra (1948). When the weight of the fill exceeded the bearing capacity of the underlying soft clay, the soil was displaced laterally under the abutments.

1.9 CONSTRUCTION METHODS

1.9.1 Building foundations

From the very beginning of man's building activity, the most common foundation method in soft soil has been piling. This was especially so when cathedrals and other heavy buildings were erected. Recent restoration works in Oslo on old churches which had settled revealed timber piles which were very closely driven. The buildings in question are Hallward's Church (probably from the first half of the 12th century), Olaf's Church (from about 1250), and Maria Church from 1300. At Olaf's Church, it was found that two timber grillages were placed on the piles and bedded in sand to form a 'timber



(a) *before 1900*

1. Stone wall with displaced joints. Dry masonry wall (or with mortar).
2. Logs, transverse.
3. Logs, longitudinal.
4. Compacted clay.
5. Timber piles to firm ground or great depth.

(b) *1880–1930*

1. Stone wall with displaced joints. Dry masonry wall (or with mortar).
2. Planks, 50 to 75 mm, placed diagonally.
3. Planks, 75 to 100 m, placed diagonally.
4. Logs, longitudinal.
5. Compacted clay.
6. Timber piles to firm ground.

(c) *1910–1950*

1. Concrete (with cobbles before 1940).
2. Concrete with reinforcement of rails, etc.
3. Gravel compacted with crowbar.
4. Timber piles to firm ground.

(d) *after 1930*

1. Concrete.
2. Concrete with two longitudinal bars.
3. Blinding of gravel or concrete.
4. Concrete piles to firm ground.

Fig. 1.47. Development of foundations on very soft soil in Sweden (Bjerking, 1974).

raft' over a layer of compacted clay. The piles inspected at Olaf's Church were found to be in perfect condition after 650 years (Jørstad, 1981).

The foundations of structures built in Sweden before 1900 generally consisted of a ditch which was filled with stones and boulders. Later, dry-stone or masonry walls were used which were placed on a layer of sand or gravel. On very soft soils, the walls were placed on a timber grillage on timber piles protected by a layer of compacted clay, as shown in Fig. 1.47. Sometimes, the wall would also be placed in an excavation to prevent any lateral displacements, as indicated in Fig. 1.48 (Kolderup, 1894).

Timber grillages were generally used to support two- or three-storey wooden buildings, mainly to reduce the differential settlements, since this type of foundation is relatively stiff (Fig. 1.48b). The grillages were initially constructed of logs which were carefully jointed (Bjerking, 1974), but later, wooden planks or beams were used. The construction of grillages and other foundations used in the latter half of the 19th century has been described by Kolderup (1894) (see also the German design manual by Hagen, 1841, which was commonly used at that time). Timber grillages, alone and in combination with piles, were described by König as early as 1752.

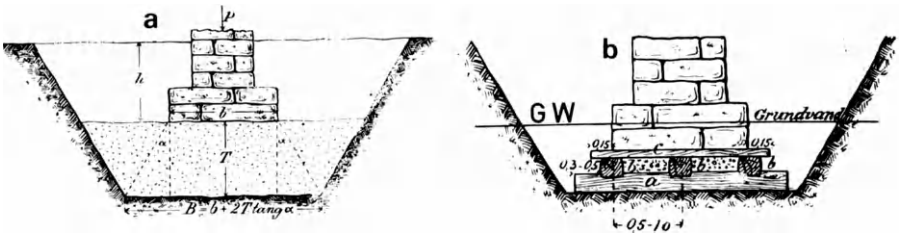


Fig. 1.48. Early foundation methods for soft soil (Kolderup, 1894).

It was realized early that wood had to be kept permanently below the ground water level. In the Swedish manual by Stål (1854) it is mentioned that wood above the ground water level could be preserved with charcoal ash or by charring the timber over an open fire. Frequently, the logs were embedded in clay.

Footings and walls of concrete were introduced at the turn of this century, and they were commonly used in Sweden about 1910 (Fig. 1.47). Thicknesses were at first relatively large, 0.5 to 1.0 m, but these were later reduced to 0.3 to 0.5 m. Grillages and concrete walls in soft clay were normally supported on timber piles or (sometimes) on piers which extended to firm bottoms. The lengths of early piles were only 4 to 7 m during the 19th century, and the spacing was 0.6 to 0.7 m. Three rows were generally required below load bearing walls. From a settlement point of view, such piles were not very efficient in locations with deep deposits of soft clay (e.g. Gothenburg). Between 1890 and 1930, the length of piles increased to 10 to 12 m, and they

were placed in two, three or four rows below load bearing walls. In the 1940s, 18 to 20 m long piles were common and were used as both friction and point bearing piles; they were often spliced when the depth of soft clay was large (Fig. 1.42), with a long bottom and a relatively short top section. Stål (1854) pointed out that in soft clay, adjacent piles could 'heave' during driving. It was recommended that timber piles should be driven with the root ends downwards. Special cleats were also used, as described above (Fig. 1.45).

The Old Town of Stockholm, founded about 1250, offers many foundation problems, as described by Kerisel (1975) in his Rankine lecture, because of isostatic uplift. Timber piles have rotted and structures have settled because of the gradually lowered ground water level. It has been suggested that the Old Town, where the soil consists of soft clay and fill, has not only settled but has also moved laterally, because piles which were driven in 1350 and 1650 are no longer vertical; this question, however, is still (1981) under discussion. The saving of this old, historically interesting part of Stockholm will be a very difficult task; it is a 'Venice of the North'.

The pile driver shown in Fig. 1.49 was used in the 16th century in Stockholm (Hansson, 1956) to drive piles from the ice during the winter. This pile driver was later placed on a barge so it could be used also in open water. Piles were driven for moorings as well as for fortifications in the harbours. Olaus Magnus (1555), who is the author of Fig. 1.49, wrote that "Stockholm, the capital of the Kingdom of Sweden, is resting on such piles". Figure 1.50, which is based on a drawing by the architect Arthur Löwe (1978) of the Real Estate Department of Stockholm, illustrates the problem of the Old Town of Stockholm in a nutshell. It shows the settlements of a street and a basement between 1650

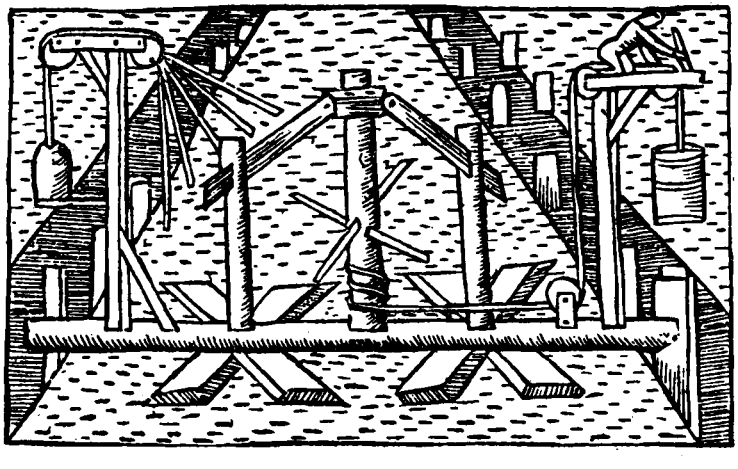


Fig. 1.49. 'Double acting' pile driver used in the Old Town of Stockholm in the 16th century, originally described by Olaus Magnus in 1555 in his large work *History of the Nordic Peoples* (Hansson, 1956).

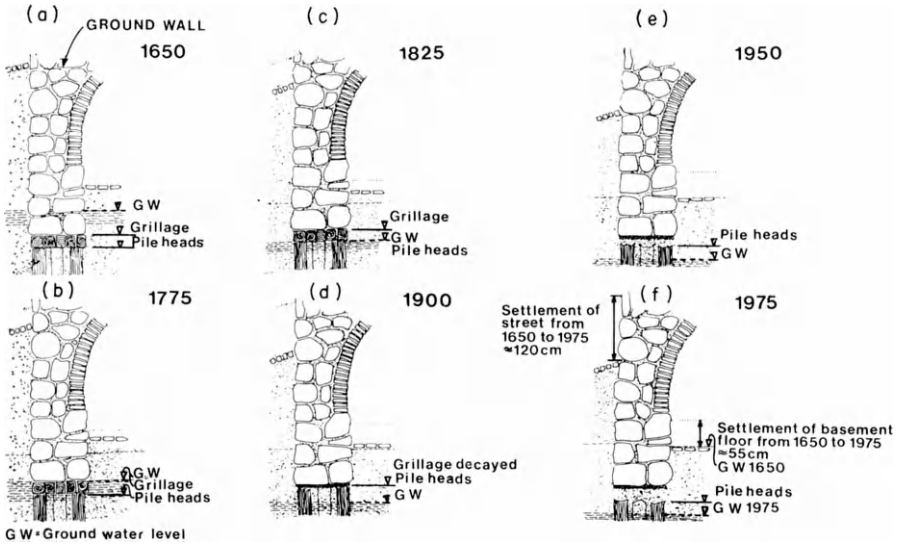


Fig. 1.50. Examples of settlements in the Old Town of Stockholm from 1650 to 1975 (Löwe, 1978).

and 1975, and illustrates how the timber grillage and the heads of the timber piles have decayed as the ground water level has declined because of isostatic uplift. A few of the houses which have settled have been underpinned in recent times, but the costs are very high, and this method cannot be used for all houses that have been affected. New and cheaper methods, therefore, are urgently required.

The underpinning of the two eastern wings of the Royal Palace in Stockholm (Fig. 1.51) should also be mentioned. The two wings at the edge of an esker were founded on short timber piles in the middle of the 18th century. Below the wings, there is a layer of old timber grillages, piles, etc which was left from the various stages of older construction. Beneath this troublesome foundation material, there are lenses or layers of soft clay of varying thickness, with a maximum of about 2 m below the southern wing and about 3.5 m below the northern wing. The clay is mixed with sand and gravel, and pieces of wood are also found. Below is sand, gravel and boulders, which rest on till and rock at a depth varying between about 11 m and 30 m. The timber piles, which were not driven to a firm bottom, have gradually rotted as the ground water level has lowered. Consolidation settlements have also contributed to the differential settlements, which are manifest in the tilting of the wings outwards towards the east. An attempt was made about 1920 to stop the settlements. A concrete slab more than 1 m thick was cast below the two wings in order to distribute the load more evenly. The added weight made the situation worse, however,

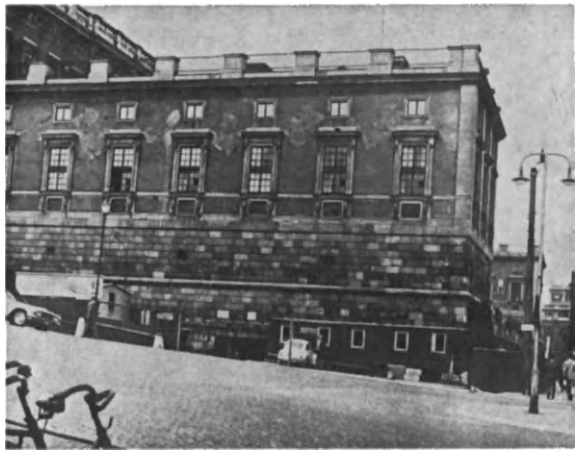


Fig. 1.51. Underpinned wings of the Royal Palace in Stockholm, photographed from the south.

and the settlement rate increased. Measurements in the 1950s showed that the wings settled 2.5 mm per year. In 1963, the wings were underpinned with concrete piers which extended to bedrock (Engineering News Record, 1965). The same method, developed by Nils Engström, had been used successfully for the first time a couple of years earlier to save a neighbouring building in the Old Town, as reported by Lindqvist (1961).

1.9.2 Floating foundations

The principle of floating foundations is frequently used to reduce settlements, especially in soft clays. The origin of the method has been discussed by Golder (1965, 1975) and Skempton (1971), and Terzaghi (1944). A. Casagrande & Fadum (1944) and Little (1961) have also shown interest in the early history of the method.

Golder (1975) described the method as: "A foundation for a building in which the weight of the building is approximately equal to the full weight (including water) of the soil removed from the site of the building". The method is perhaps best known under the name *compensated foundation*, but also the term *buoyant foundation* is used. The method has incorrectly been credited to the famous British civil engineer John Rennie in connection with the construction of the Albion Mill on the bank of the River Thames in London in about 1780, whereas it was probably Samuel Wyatt, the designer of the building, who first proposed the idea. The method was not utilized fully at Albion Mill, since the weight of excavated earth corresponded to about 50% of the total weight of the building.

The principle of floating foundations was known in Scandinavia in the 19th century and was described in the handbook by Hagen (1870). The method was more or less forgotten everywhere until it was 'rediscovered' at the beginning of the present century. It was used, for example, in Canada for the Empress Hotel in Victoria, British Columbia, which was built in 1908. In 1914, 1.4 m of soil was excavated below one end of this building and 3.8 m was removed just outside the building to reduce the alarmingly high settlement rate (Crawford & Sutherland, 1971).

The first 'modern' use of the principle of floating foundations was in the Ohio Bell Telephone Company Building in Cleveland, Ohio in 1925 (Freeman, 1944; Golder, 1975). This building is founded on stiff grey clay of great depth, and the credit for the foundation belongs to D.J. Moran, whose firm also designed the Alban Telephone Building in New York in 1929 where the method was also used. The soil below the latter structure consists of alternating layers of highly plastic, very sensitive, varved clay, silty clay, silt and very fine sand (Casagrande & Fadum, 1944). The method has in addition been applied in Scotland in soft estuarine clays where a so-called 'egg-box' foundation was used, in Nigeria and India in the 1940s, and in Guayana in 1956–57 (Golder, 1975).

In Scandinavia, the floating foundation method has been used in Norway in soft marine clays for five relatively high buildings in the Oslo and Tröndelag regions (Drammen and Fredrikstad), as described by Bjerrum & Eide (1966), who pointed out that it is important that the heave due to the excavation of the soil should be as small as possible to reduce the disturbance of the clay during construction and to minimize the resulting settlements.

Probably the world's best known example of the floating foundation method is the 140 m high Latin American Tower in Mexico City (Zeevaert, 1957). Timber piles were driven through the soft clay down to a bearing stratum at a 33.5 m depth to reduce the heave during excavation and to carry part of the weight of the building. The ground water level was also controlled to reduce the heave. Four years after the construction, the maximum settlement was only about 50 mm. The successful construction of this building in such difficult clay with its very high water content must be regarded as a great achievement. A real test of the building and its foundation was the severe earthquake of 1959.

1.9.3 Harbours

The need to improve and enlarge existing harbours and to build new ones became apparent at the beginning of the industrial revolution during the first half of the 19th century. To build deep harbours where soft clay layers existed proved to be a difficult task. Several failures occurred, for example, in Stockholm, Gothenburg and Trondheim before suitable design and construction methods were developed. The best-known example involving such

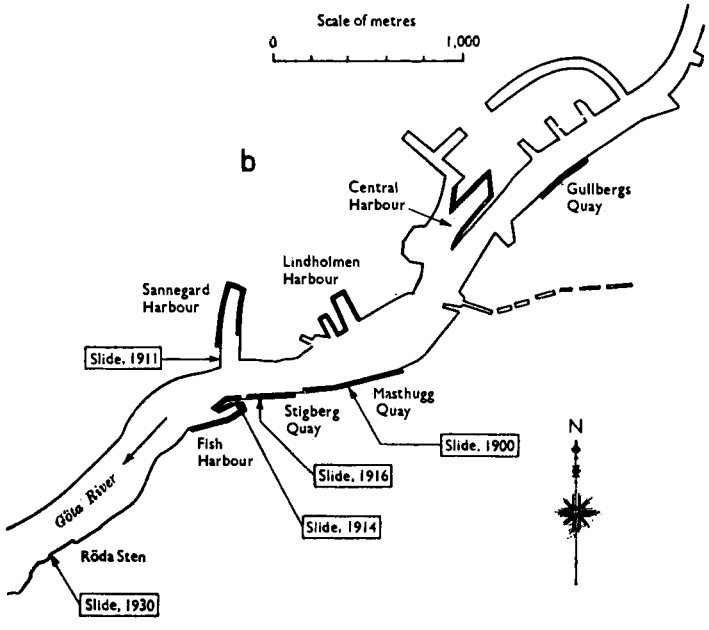


Fig. 1.52. Plan of Gothenburg Harbour indicating the main slides (Petterson, 1955).

developments is probably Gothenburg Harbour, illustrated in Fig. 1.52 (Petterson, 1935, 1955).

The first 'modern' quay in Gothenburg, with a water depth of 6 m, was the Masthugg Quay built in 1888 (see Fig. 1.53a). Caissons with walls of natural stone supported on timber piles were constructed, the purpose of the caissons being to reduce the weight of the quay. Fascines 25 m wide behind the caissons reduced the lateral pressure on the caissons. A slide which occurred in 1900, however, indicated that the fill behind the wall was too heavy. A pile-supported platform which carried the weight of the fill was then constructed (Fig. 1.53b), and a gravel berm was used as a counterweight in front of the wall. This design was later improved, and the fascines were omitted, as were the heavy caissons (Fig. 1.53c). Loading berms were also used at the Sannegard Quay (Fig. 1.53d) constructed in 1914, as well as for the extension of the Stigberg Quay (Fig. 1.53e), where the well-known slide of 1916 occurred.

Wolmar Fellenius, as the first head of the new Construction Department of Gothenburg Harbour, was responsible for the design of Gothenburg's new quays in 1905. He proposed the method shown in Fig. 1.54 to compute the stability of quay walls. Failure was assumed to take place along a surface composed of three straight lines. This method of analysis was presented by Fellenius in his inaugural lecture in 1910 as Professor of Hydraulics at the Royal Institute of Technology in Stockholm. At the site of a small slide at Fiskhamnspiren in Gothenburg in 1914, K.E. Petterson, who was Engineer for

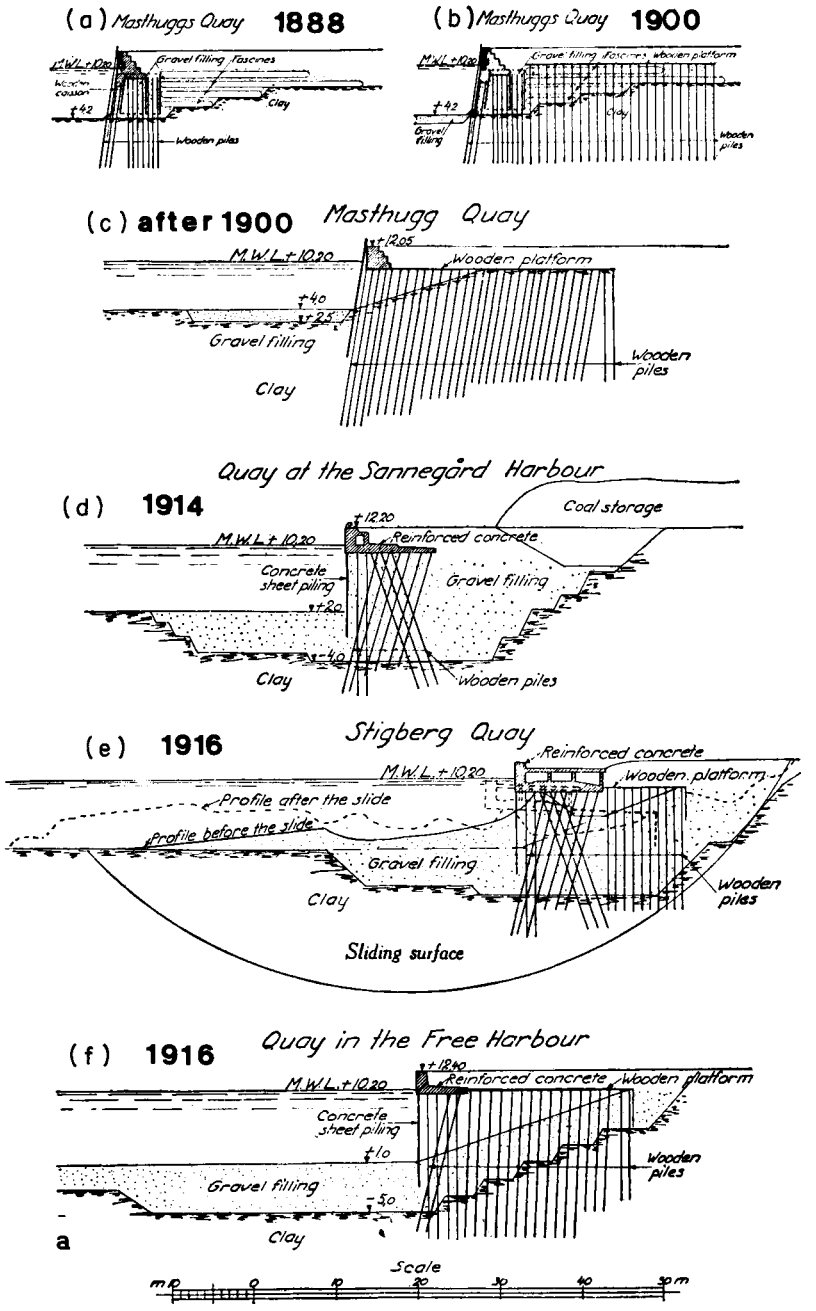


Fig. 1.53. Development of quay constructions in Gothenburg (Petterson 1935).

the harbour, had already observed from borings a rupture surface that was almost circular. This was confirmed at the Stigberg slide. S. Hultin, head of the Design Section, was given the task to develop an analysis method based on this observation. The resulting slip circle method, shown in Fig. 1.55 (Petterson, 1916; S. Hultin, 1916), was later modified by W. Fellenius (1918, 1919, 1926, 1927, 1929), as described in Section 1.7.1, to take into account both cohesion and friction.

A Commission was appointed in connection with the Stigberg slide, consisting of six persons, among them W. Fellenius, K.E. Petterson and the two Professors H.A. Ysselstein of Holland and Max Möller of Germany. A report was completed a few months after the slide (Göteborgs Hamnstyrelse, 1916). The Commission carried out a full-scale load test at the Central (Free) Harbour in Gothenburg which was then under construction. A gravel fill was used to investigate the stability of the structure. Some movements and cracks were observed, and the Commission concluded that the limit of the strength of the clay had been reached. A stability calculation using the slip circle method gave an angle of internal friction of 13°, compared with about 10° for the Stigberg Quay. The clay was considered as a cohesionless material.

The Stigberg slide showed that the weight of the fill in front of the quay was not adequate. The design was modified, therefore, when the quay was repaired, as illustrated in Fig. 1.53f. This modified design was also used for the Central Harbour, which was completed a few years later. The weight of the gravel fill behind the quay wall was reduced by sloping the surface, and friction piles were driven through the fill to increase the stability. Even this relatively light wall was found to be too heavy, the settlements being excessive.

A modified type of quay wall was used for the Stora Bommen Quay in 1927 and for the Gasverk Quay in 1928 in Gothenburg (not shown in Fig. 1.52). Composite piles with steel or concrete top sections were used to prevent decay of the piles at the water-line. The piles were spliced using special joints just below the mud-line. A further development was the platform of reinforced concrete which was used for the Gullberg Quay in the 1930s.

The Lindholmen Quay, constructed during 1935 to 1940 in Gothenburg in a water depth of 9.2 m, was entirely supported on composite piles. The fill behind the wall was carried by 18 to 22 m long battered timber piles. A gravel fill was placed as counterweight in front of the quay to increase the stability. A more recent example is the new Gullberg Quay built in 1958 to 1959, where spliced concrete piles 46 to 54 m long were used.

In soft clays where the depth to firm bottom is fairly small, as in Stockholm, steel sheet piles have been used since about 1930 (Petterson, 1935) in combination with timber piles.

Those interested in the detailed development of quay construction in soft clays are referred to the *Proceedings of the International Congresses on Navigation.*

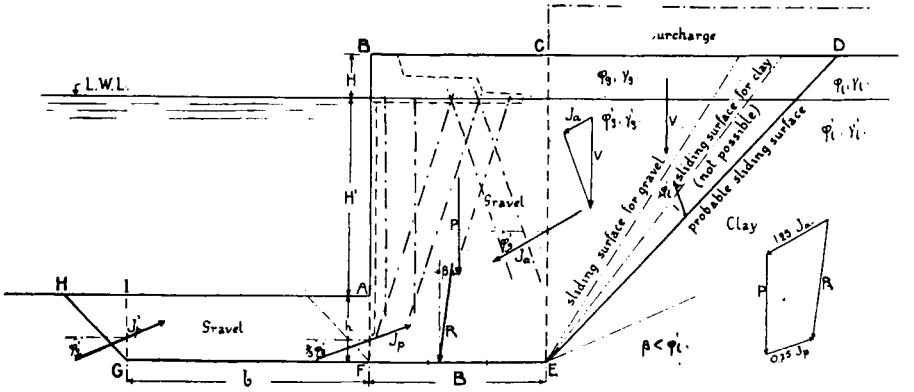


Fig. 1.54. Method for computing the stability of quay walls as proposed by W. Fellenius (1910).

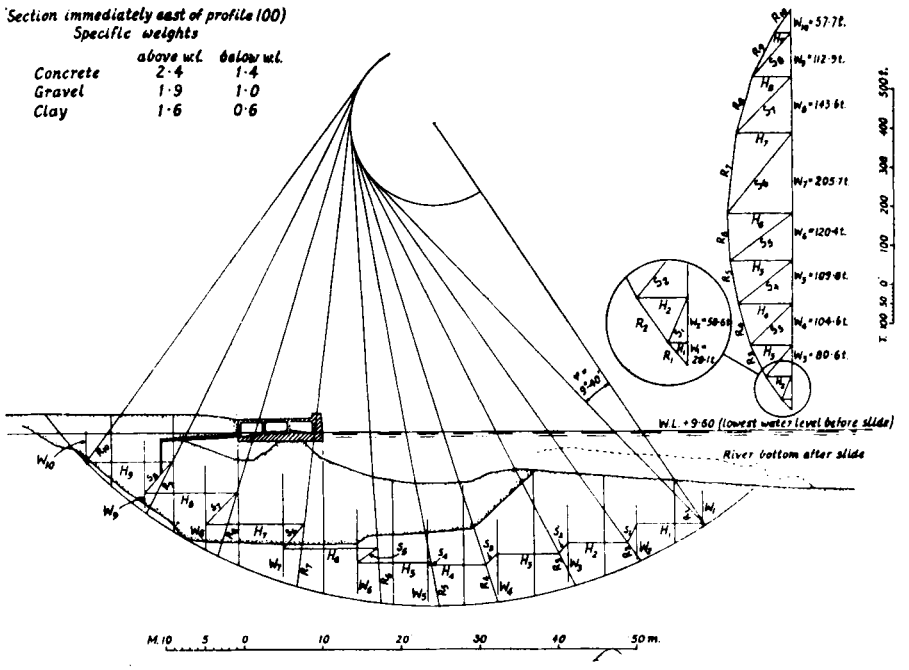


Fig. 1.55. Slip circle method developed in 1916 by S. Hultin and K.E. Petterson to analyze the Stigberg slide.

1.9.4 Tunnels

Tunnelling is one of the oldest activities of man. The book by Sandström (1963) covered 6,000 years of tunnelling, an indication of the length of history of this activity. During Roman times, well-engineered tunnels were constructed for roads in rocky terrain, the best known being that for Via Flaminia between Naples and Puzzoli built in 36 B.C., which was 1.5 km long and had a 7.5 m wide by 9 m high cross-section.

Tunnelling through soil, as opposed to rock, appears to have commenced successfully much later. The support systems, composed mainly of timber, were not always entirely adequate, and major failures often occurred with considerable loss of life. In the 19th century, soil tunnels were fairly common. In a report by T. Hultin (1926), there appeared a list of 32 soil tunnels built between 1841 and 1925. Ten of those in the list were built through soft clay *(lös lera)* or blue clay *(blålera)*. The construction methods used relied mainly upon shields in combination with air pressure, but some employed shields without air pressure, and some employed cofferdams or caissons. Permanent tunnel linings varied from bricks and cast-iron to reinforced concrete. The oldest tunnel listed was that under the Thames in London constructed in the period 1825 to 1841, and the newest was the Holland Tunnel under the Hudson River in New York constructed in 1921 to 1925. Of special interest from the point of view of soft clay was the railway tunnel under the St Clair River between Michigan, USA, and Ontario, Canada, built in 1844 to 1890; the tunnel was driven through 4 m of soft clay beneath a water depth of 12 m.

At the same meeting at which T. Hultin presented his historical list of tunnels, Petterson (1926b) gave a lecture on the history of the feasibility studies for a tunnel in the soft Gothenburg clay. It was not to be until about 40 years later, however, that the first tunnel under the Göta River was to become a reality. This tunnel, the Tingstad Tunnel, is discussed below.

It is believed that the Holland Tunnel, which joins New York City and New Jersey, was the first large tunnel built in soft clay. A shield was used for the construction of the tunnel, which had a diameter of almost 9 m. Most of the soil was displaced during the driving. The elevation of the shield was controlled by regulating the volume of the soil entering through the openings in the shield. The shear strength of the soil was very low, and the soft clay flowed into the tunnel in 'sausages' when the grout holes in the tunnel segments were opened. It also squeezed into the tunnel through the annular space between the tail of the shield and the lining (King, 1943; Fox & Nicolau, 1976).

A shield was also used in 1939 during the construction of the subway tunnels in Chicago, as described by Terzaghi (1942b). The tunnels, 7.7 m in diameter, were partly constructed in clay with a shear strength of less than 20 kPa. When the shield was advanced, the clay flowed through the openings at the front of the shield, the area of which could be varied. Relatively large settlements were

observed above the tunnels, which corresponded to the volume of clay which squeezed into the tunnel during the construction.

Considerable difficulties were encountered during the construction of the tunnel for the Holmenkollen Line in Oslo from 1912 to 1914. The tunnel was partly constructed in rock and partly in soft clay. In the sections with soft clay, the tunnel was constructed in an open excavation. The tunnel caused a considerable lowering of the ground water, and large settlements (up to 350 mm) resulted from the high compressibility of the soil (see Section 1.4.2.1). More than 100 structures were damaged up to 260 m from the tunnel, and some of these had to be demolished (I. Vogt, 1914, 1916; G. Holmsen, 1953). The ground water-table was restored after the tunnel was lined. Three volumes of a report on the damage to the structures were presented by Björlykke in 1914, 1921 and 1928 respectively (see also Haukelid, 1930).

Various construction methods, such as compressed air and excavation below water, were used during the extension of the Holmenkollen Line from 1955 to 1965. In some places, the stability was so low that excavation had to be carried out in narrow strips to prevent failure by bottom heave. Lateral earth pressures, strut loads, deformations and settlements were measured during the construction of the tunnel (Norwegian Geotechnical Institute, 1972). Slurry trench walls were used at Studenterlunden in Oslo during the construction of the new subway, and these extended 6 m below the bottom of the excavation to prevent failure by bottom heave in the soft clay. The settlements around the excavation were small (Eide et al, 1972).

Considerable difficulties were encountered during the construction of the Tyholt Tunnel from 1945 to 1950 south of Trondheim, Norway. A shield was used for the part of the tunnel which was driven in medium to soft clay. The sensitivity of the clay was in places very high (100 to 200), and several failures occurred during the construction of the tunnel (Hartmark, 1955).

The number of tunnels constructed in soft clay has increased considerably during the last twenty years as new techniques have been developed. Examples of recent subway tunnels passing partly through soft clays, beside those in Oslo and Tyholt, are to be found in Stockholm, Mexico City and Gothenburg.

The stability of tunnels in soft clay can be evaluated from the stability number, $(\sigma_v - \sigma_a)/s_u$ where σ_v is the total vertical pressure at the centre of the tunnel, σ_a is the air pressure and, s_u is the undrained shear strength of the soil (Broms & Bennermark, 1967). As indicated by Peck (1969), tunnelling can be carried out without unusual difficulties when the stability number is less than about 5; the shield has a tendency to tilt when this approaches 7. The stability can be increased by the use of compressed air, and experience indicates that the maximum possible air pressure corresponds to the outside water pressure, otherwise 'blowout' occurs. The soil cover must be sufficient to prevent the tunnel from being damaged and from breaking through the overlying soil because of buoyancy. In soft clay, the openings in the shield are

often kept small, and most of the soil is thereby displaced during the jacking of the shield; the heave of the soil above the tunnel is then large.

Tunnelling in very soft clay is generally associated with relatively large settlements (affected by chosen construction method), a lowering of the ground water level, and reconsolidation of the remoulded soil around the tunnel. The soil is disturbed around the tunnel to a distance from the centre-line of the tunnel equal to approximately the depth of the tunnel. If the predicted settlements are excessive, it may be necessary to construct the tunnel in an open cut by the cut-and-cover method. In water, tunnels can also be constructed of prefabricated segments which are floated to the site and placed in an excavated channel. In cases where the shear strength of the clay is very low, it is necessary to support the tunnel on piles. This method was used during the construction of the Tingstad Tunnel below the Göta River in Gothenburg. The timber piles supporting the tunnel were driven before the tunnel elements were placed (S. Carlsson, 1968).

An interesting book on the history of tunnelling has recently been published by Harding (1981).

1.9.5 Roads and railways

In ancient times, roads (or rather paths) followed firm ground wherever possible, but it was sometimes necessary for them to cross peat bogs or areas of similarly soft soil. The Romans were particularly advanced in their construction of military roads, and what is left of these indicates that the early road builders were also able soils engineers (see Jumikis, 1967; Plommer, 1973). Much earlier were the ancient British roads described by Dewer (1962), which were found recently during excavations in peat. The roads were constructed of planks tied together longitudinally at each end, with stakes driven vertically every third metre along the sides to hold them in place. These 'causeways' were dated by the C^{14} method as being 4,000 to 4,800 years old. No advances of importance were made in the methods for constructing roads over soft soils for thousands of years until the advent of motorized traffic called for greatly improved roads (Brawner & Tessier, 1969; Lee, 1973).

When the first railways were built during the middle of the 19th century, much of the experience gained from road construction could be used. However, the rapid growth of the railways and the often poor soil conditions magnified the construction problems.

In Scandinavia, few roads were built before the first half of the 19th century. In the middle of the century, a National Road Board was created in Sweden. The Board issued some technical instructions which were mainly concerned with the construction of embankments on soft soil. Brushwood, fascines and timber grillages were commonly used at that time to increase the bearing

capacity. A primitive type of grillage consisting of slender fence poles was often used for smaller roads, while a more 'advanced' type of grillage was introduced which consisted of logs placed in two perpendicular layers. Five or six rows were placed in the bottom layer along the road, while in the upper layer the logs were placed cross-wise close together. Soil from the ditches along the road was placed on the grillage, and this soil layer was covered by sandy clay, till or gravel.

In the 1938 Code of Practice of the Swedish Road Board, the need for geotechnical advice was expressed for the first time for roads constructed on soil of low bearing capacity. A geotechnical engineer, Walter Kjellman (Fig. 1.3), was employed by the Board in 1936. One of his many qualifications was that he had worked with Terzaghi in Vienna a few years earlier, and he would later be a 'breaker of new ground' in Swedish geotechnical engineering.

It was recommended that road embankments on soft ground should be made as light as possible by reducing their height or by using lightweight materials such as slag. If this was not possible, fascines, timber grillages or piles could be used in accordance with Fig. 1.56. Timber grillages were mainly utilized for small roads with a maximum width of 3 m and a maximum embankment height of 1 m. Piles were as a rule used when the depth of the soft soil exceeded 6 m. The weight of the embankment was transferred to the piles partly through friction along the upper part of the piles. If a firm top layer (dry crust) was lacking, a bed of brush wood was placed between the piles, as shown in Fig. 1.56. Today, special concrete caps or slabs are used which are placed on the piles.

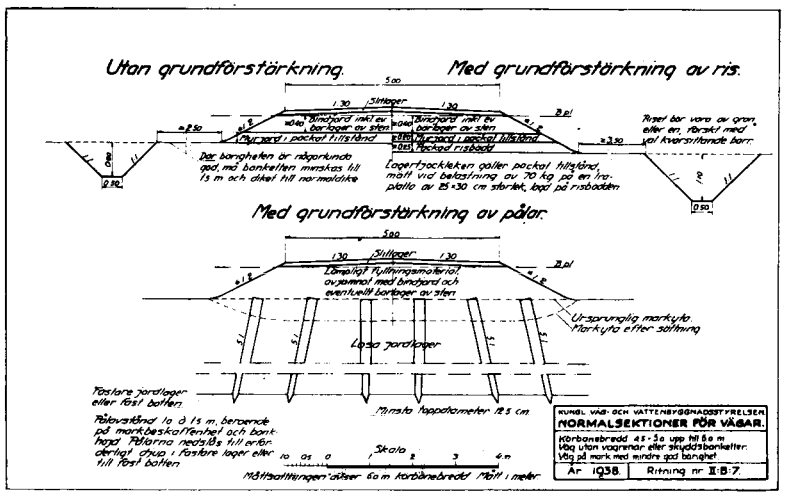


Fig. 1.56. Embankment drawing from the 1938 Code of Practice of the Swedish Road Board. Upper figure shows embankment (left) without strengthening and (right) with strengthening by brushwood; lower figure shows embankment strengthened with piles.

Another road construction method was to displace the soft soil to a firm bottom by overloading or blasting (the latter method is described below). If these methods were not technically or economically feasible, loading berms were often used, as described above, especially when the depth of the soft soil exceeded 6 m.

The construction of roads on soft soil, including the stability and settlement of embankments, was reviewed in a paper by Kjellman (1940b). He criticised the use of fascines and timber grillages, but he admitted that fascines could be of value for low embankments, where they function as reinforcement. (Compare this with the recent use of fabric as reinforcement in soil.) The low weight of the fascines was a favourable factor, but the long-term increase in bearing capacity brought about by the fascines was doubtful if they had been placed above the ground water level. It was also concluded by Kjellman that the beneficial effects of grillages are small in comparison with their high costs, and it was recommended that timber grillages should not be used for roads wider than 4 m. A serious disadvantage was that such roads could not be widened without difficulty. In extremely poor soils, pile-supported concrete decks are now sometimes used, but this method is very expensive.

The use of lightweight fill is a method with long tradition in Thailand with its soft clay. Temples were founded on bamboo rafts (grillages) covered with jars which were turned upside down and filled with burned rice chaff. The latter material is still used today. For small embankments, however, its practical effectiveness is small since a heavy pavement is needed (Fjellerup, 1977).

The same construction methods were used for railway embankments as for roads. The bearing capacity of embankments was increased using loading (pressure) berms, a method which had been used during the construction of the Göta Canal in Sweden at the beginning of the 19th century. This method, as well as the displacement of soft soil by overloading or blasting, was described by the railway engineer Insulander (1869). Timber grillages (often combined with loading berms) and fascines were also used. An underlying layer of soft soil was frequently displaced by overloading when the height of the embankment was large. Insulander pointed out that the height of a test embankment should be relatively large to overcome the "friction" when the underlying soil is pushed to the side. He recommended that the length should be 100 to 130 m and the overload about 2 m.

Narrow single-track railway lines were often supported on timber grillages, especially when there was a dry crust at the surface (see Caldenius, 1925). One disadvantage with this method was the difficulty of widening the embankment for double-track traffic. The bearing capacity of the soft soil was seldom sufficient to carry the additional load from the widening.

Several examples of embankments constructed on soft clay are described in the *Final Report* of the Swedish Geotechnical Commission (1922). The settlements of the embankments supported on timber grillages were frequently

very large because of the consolidation or lateral displacement of the underlying soft clay. It was observed in several cases that the ground surface around the embankment heaved several metres, and there were even cases when the movements had been so large that the grillages overturned.

During the construction of the Bohus Line, the embankments were frequently forced down through the underlying soft soil by blasting which often had to be repeated several times to displace pockets of soft soil trapped below the embankment (see Section 1.9.6 below). The Bohus Line, with its extremely poor soil conditions, on which John Olsson gained his first geotechnical experience, was like a large experimental site in this respect.

The railway through Järna about 50 km south of Stockholm was originally constructed in 1860 as a single-track line. A 100 m long section failed when the line was widened. 'Undisturbed' continuous samples were taken using a 6 m long "split spoon sampler" of 76 mm inside diameter which had a cutting shoe with an inside clearance to decrease the side friction. It was used to study in detail the displacement of the soil and the slip surface caused by the stratification in the varved glacial clay. Piles were used when the depth to firm ground was large (cf. Swedish Geotechnical Commission, 1922, pp 116–118, and Samuelsson, 1926).

The construction methods used in the other Nordic countries to stabilize railway embankments followed in general the 'Swedish pattern', because of the intimate contacts between geotechnical engineers in these countries. Because of several embankment failures and landslides in Finland, a Finnish Geotechnical Commission was appointed in 1922, with T. Brenner as Secretary; several case records were later described by him (Brenner, 1925, 1928).

The records in the files of the Geotechnical Section of the Finnish State Railways of slopes and embankments which had failed were reanalyzed by Helenelund (1953) using the slip circle method and the undrained shear strength of the caly ($\varphi=0$ analysis). The effect of the dry weathered crust was taken into account. The old investigations were supplemented by additional field investigations. Helenelund found that the configuration of the displaced soil could be very complex, as illustrated in Fig. 1.57 for an embankment at Väärlä, where a series of slips resulted in a "pygoidal" shape of the fill. A case where a pile-supported bridge abutment had been damaged by the backfill was also described.

Up to the turn of the century, the most common method to displace soft soil was by overloading. Blasting was tried for the first time in Sweden about 1900 (Lagergren, 1936). A narrow deep ditch was excavated below the centre of the planned embankment so that the embankment material forced the soft soil to the side when the embankment sank during the blasting. However, this method was both expensive and time-consuming, since it was generally necessary to check the configuration of the fill with borings after the blasting. Loading berms were used as much as possible, therefore (Insulander, 1869).

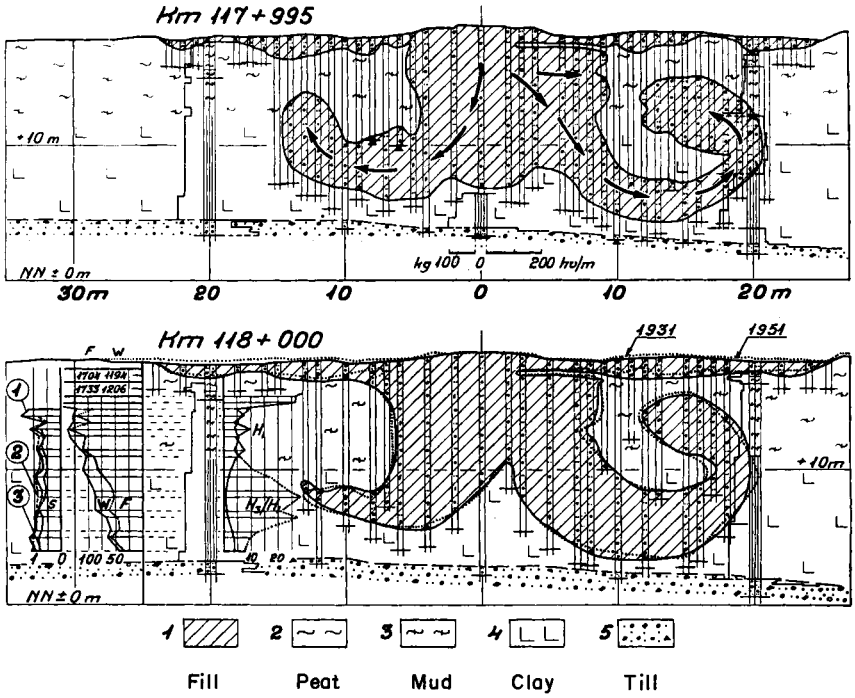


Fig. 1.57. Ground failure of railway embankments at Väärlä, Finland. Arrows indicate the directions of mass-movements (Helenelund, 1953).

(Lagergren pointed out that berms were formed to a certain extent by the displaced soil on both sides of the main embankment during the blasting.) Blasting has been extensively used in Scandinavia in soft clay and peat.

An interesting case of blasting was the widening of an embankment at Djurgården in 1923 on the railway line between Helsinki and Fredriksberg in Finland, where the blasting was done in stages. The volume of soil displaced by the rock fill for the new embankments was very large, with a difference in height between the old and new embankments of up to 7 m, with disastrous effects on the surroundings.

Blasting was used in 1943 during the construction of a 200 m long road embankment constructed on about 12 m of soft clay at Lake Aspen about 20 km south of Stockholm (Rudhagen, 1944). A fill was first placed in a narrow strip along the centre of the road with what was thought to be sufficient overheight to displace the subsoil. It was impossible, however, to displace the soft clay entirely even with a surcharge of 1.8 m. The settlement rate of the embankment at that stage was 20 mm per month, and it would have taken at least ten years to completely consolidate the soil. Blasting was used, therefore, to displace the remaining soft clay (vibration was also considered). The total volume of the fill was 80,000 m^3, and over 6 t of dynamite was used. There were still settlements after the blasting, indicating that some clay still

remained below the fill, but after a couple of years, the settlements had practically ceased. This successful blasting was then the largest in Europe.

The blasting method for soil displacement has gradually been improved but, in principle, it is today the same as it was around 1900 (Casagrande & Wheeler, 1934; Parsons, 1939; L. Casagrande, 1965; Floss, 1970; Laboratoires Ponts et Chaussées, 1971). Recommendations for blasting were issued by the National Swedish Road Authority in 1963.

1.10 STRENGTHENING OF SOFT CLAY

1.10.1 Vertical drains

Several methods have been developed during the last decades to increase the bearing capacity of soft clay, such as preloading, vertical drainage (including the vacuum method) and electro-osmosis. Some of the methods are only a few years old and, thus, are outside the scope of this review; examples are the lime-column method, where unslaked lime is mixed with soft clay to improve the bearing capacity and to reduce settlements (Broms & Boman, 1977), and fabric-reinforced soil, where woven or non-woven fibre materials are placed in embankments and cuts to improve their stability (see Holtz, 1975, who also gives a brief history of techniques to reinforce soft soil). The injection of lime directly into soft clay through boreholes has also been tried, but the results have up to now not been very successful.

Vertical drainage is used extensively in the Nordic countries as well as in Canada to increase the consolidation rate and shear strength of primarily soft clay. This method is used widely also in the USA, USSR and other countries. According to Pilot (1981), the concept of the principle of vertical drains was known by Moreau in 1832, but the method was first used practically in the 1920s. Two different vertical drainage methods were developed independently of one another, namely the sand-drain method by Moran in 1925 (Johnson, 1970b) and the cardboard-drain method by Kjellman in 1937 (Kjellman, 1948c). Kjellman's first approach during the last part of the 1930s included vertical fibre tubes, before he eventually tried thin, rectangular cardboard wicks, which served as the prototype of the paper-plastic drains which are used today (1981). In the USSR, the use of sand drains was first reported by Gersevanov & Polshin (1948).

Moran patented his method in 1925 and called the drains *drainage piles* because they were also driven as compacted sand piles and as displacement cast-in-place piles. At the same time, they acted as pressure-producing piles to consolidate the mass between the lateral lines of drains. The 'drainage piles', as described in the application for the patent, could also be used to stabilize tunnels and excavations in soft soil, in a similar manner to the lime column method of today. Moran also used sand drains to stabilize the mud below a road embankment for the San Francisco Bay Bridge. After laboratory

and field experiments in 1933–34, the California Division of Highways installed the first sand drain system in 1934. The method was further developed, first by the California Division of Highways as a result of the interest of Porter (1936), then by the US Corps of Engineers beginning in 1941, as described by Barron (1944). Terzaghi was also interested at this time in the sand-drain method.

The diameter of the sand drains used in Sweden is generally 160 mm, while in the USA the diameter varies between 460 and 610 mm. In Russia, the diameter is normally 500 mm. The efficiency of sand drains is in general analyzed by the method developed by Barron (1947). Results from several sites where sand drains were used were presented by Stanton (1948). The function of sand drains in Sweden was investigated by Hansbo (1960) in full-scale tests at Skå-Edeby near Stockholm. Sand drains were also tested in 1947 at Arlanda (Halmsjön), the international airport near Stockholm. It was concluded from these studies that it was possible to eliminate practically all settlements within six months to one year.

The use of sand drains was well established in 1948, and methods existed to calculate the settlement rate, but the harmful effects due to the remoulding of the soil by the driving of the mandrels, or the marginal effect of sand drains in organic or stratified soils, such as varved clays, had not yet been established (Housel, 1954; L. Casagrande & Poulos, 1969). Magnan (1981) has compiled more than 300 references on vertical drainage which indicate that the method is commonly used throughout the world and has given rise to much controversy and discussion.

Special machines are used to drive down the Kjellman drain (or wick), which is about 3 mm thick and 100 mm wide. The prototype shown in Fig. 1.58 was tried for the first time in 1945 by the Swedish Geotechnical Institute. The maximum driving depth was 5 m but, later, a machine with a driving depth of up to 20 m was constructed. Results from the field tests at Väsby, (where the drain spacing was 0.7 m, the length of the drains was 5 m and the depth of soft clay was about 12 m), showed that the consolidation rate was much increased by the drains (Kjellman, 1948c). The secondary settlements, however, were relatively large. The results obtained at Väsby from 1945 to 1969 were summarized by Chang (1969, 1981). The Kjellman drain method and the large drain driver were later bought by Pieux Franki, and during the last two decades, the method has mainly been used in Japan.

A modification of the original Kjellman cardboard drain is the so-called *Geodrain,* proposed in about 1970 by O. Wager. The Geodrain has a grooved plastic core which is surrounded by wet-resistant paper which acts as a filter (Hansbo & Torstensson, 1977). Other types of plastic drain also exist today in Japan, France, Italy and India. In Sweden, another plastic-paper drain has recently been developed, called the *Alidrain.*

Preloading by the application of a vacuum has been tried to increase the consolidation rate (Kjellman, 1952). A light membrane is placed over the area

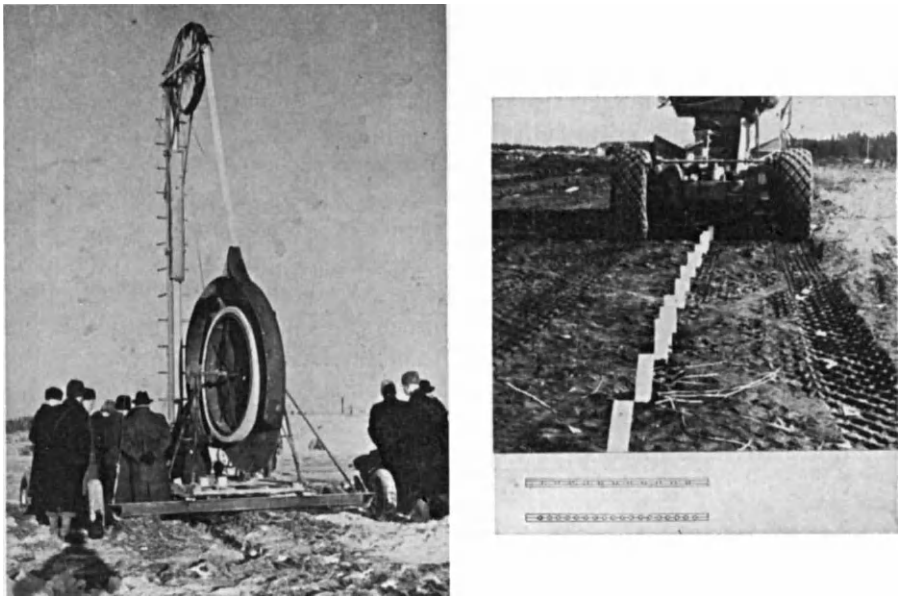


Fig. 1.58. The Kjellman cardboard drain method. The prototype machine of 1945 for depths up to 5 m is shown at left, and a larger machine of about 1947 for depths up to 10 m is seen at right.

to be stabilized, instead of a fill, and the membrane is sealed around the edges so that a vacuum can be applied below the membrane. A full-scale vacuum test was carried out at Väsby in 1947 by the Swedish Geotechnical Institute (Fig. 1.59). The test area was 10 m × 10 m, and cardboard drains were installed to a 4.8 m depth with the original small drain driver (Fig. 1.58). The spacing of the drains was 0.5 m, and the average vacuum was 80 kPa. The maximum settlement after 110 days was 0.54 m. There were some difficulties in handling the relatively brittle plastic sheets but, with the improved plastic materials which are available today, it should be possible to develop the method further.

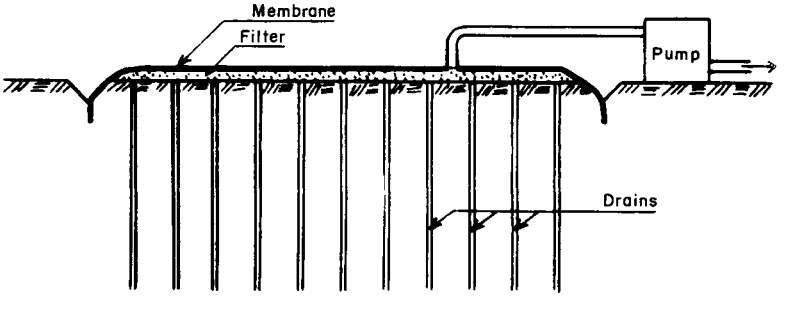


Fig. 1.59. Principle of the Swedish vacuum method (Kjellman, 1952).

1.10.2 Electro-osmosis

Dewatering and strengthening of soft clay by electro-osmosis was first tried by Endell & Hoffman (1936a) and by L. Casagrande (1941, 1947, 1952). This method is based on the observation by Reuss (1908) that water in an electric field moves towards the cathode. Helmholtz (1879) explained this phenomenon and derived the basic equations which govern the electro-osmotic flow of water through soils. An oedometer was developed by Dawson & McDonald (1948) in which the effects of electro-osmosis could be studied.

Electro-osmosis was used for the first time on a large scale in Germany in 1939 to stabilize a deep cut in a clayey silt (L. Casagrande, 1941). The method was also tried at a site close to Trondheim in Norway during the Second World War (L. Casagrande, 1947). The method is mainly effective in silty soils (L. Casagrande, 1948), but it has also been used successfully in quick clay (Bjerrum et al, 1967). If aluminium electrodes are used as anodes, there is an additional strength increase as a result of ion exchange.

1.11 CONCLUSIONS

In Scandinavia, the history of civil engineering in soft clay began about one thousand years ago, when the first heavy structures were built. In particular, Stockholm was faced with severe foundation problems about 1250 when the city wall was constructed (Hansson, 1956). An important date in the history of soft clay in civil engineering is 1620, when the city of Gothenburg was founded, and the struggle began to keep buildings from settling and quays from failing in the soft clay which was "given by fate" in the eyes of early Gotenburgers. Of equal importance is the year 1325, when Mexico City was founded on a soft clay of volcanic origin.

Industrial growth in the 19th century forced harbour and railway engineers to try new engineering solutions for structures founded on soft clay. This geotechnical era lasted to about 1913, when a number of large landslides indicated that the limits of the state-of-the-art as it existed at that time had been exceeded. Men such as Terzaghi then appeared to advance the knowledge of soft clay behaviour and to provide practical solutions to the immediate problems. Today sees the beginning of the last chapter of the often dramatic history of civil engineering in soft clay.

It should be remembered that our ancestors solved many problems through experience, after many failures. It is surely a surprise that so many ancient buildings are still standing on soft clay, and admiration must be expressed for those who were responsible for these engineering feats.

Today, decisions are increasingly based on theoretical calculations. Many have pointed out, however, that reality and theory do not always match. The value of field observations was emphasized by Terzaghi (1948) at the Second International Conference on Soil Mechanics and Foundation Engineering,

where he summarized the development of soil mechanics between 1936 and 1948 with the words:

> "As the years went by, the number of known discrepancies between the real and the ideal pattern of clay behaviour increased. Thus, for instance, it was observed that the elastic deformations of natural bodies of clay are commonly very much smaller than those computed on the basis of the elastic constants of undisturbed samples. It was also noticed that the relation between water content and overburden pressure for normally consolidated clays can be very different from what the results of our consolidation tests in the laboratory may lead us to expect."

Research is the basis for our analyses and predictions, but they have to be founded on reasonably 'safe ground', and Terzaghi's warning on a balance between research and practical realities based on experience should be kept in mind. What we have learned from the past can serve the future. Experience and old methods of construction, analysis and investigation might be forgotten, but they often appear in new forms after many years.

ACKNOWLEDGEMENTS

The Authors are indebted to Professor A. Bergfelt for his contribution on the Gothenburg vane borers, and to Dr C.R. Crawford and the late Professor K.V. Helenelund for valuable material about old landslides in Canada and Finland respectively. Special thanks are due to Mr Finn Jørstad, who generously contributed with geotechnical data on Norwegian soils and on landslides of historical interest. Acknowledgement is also extended to the library staff of the Swedish Geotechnical Institute who kindly and with great interest assisted with checking many references.

The Authors appreciate the keen personal interest shown in this chapter by Dr E.W. Brand and Dr R.P. Brenner and their careful editing of the manuscript.

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Chapter 2

ENGINEERING GEOLOGY OF SOFT CLAY

by

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Chapter 2

ENGINEERING GEOLOGY OF SOFT CLAY

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2.1 INTRODUCTION

Engineering Geology tries to relate soil (or rock) material behaviour to its geological history and attempts to express this dependence quantitatively. This is of practical significance, since the ground will respond differently to engineering works depending on the material it consists of and the geological history it has passed through.

Clays, according to the Unified Soil Classification System, are fine-grained soils with more than 50% by weight passing No. 200 US Standard sieve (0.074 mm), which have a medium to high dry strength and toughness (consistency near the plastic limit). A distinction must be made, however, between clay particles and clay-sized fraction. Clays consist of clay minerals, usually platelet-shaped particles of ultramicroscopic size, but which when aggregated, can have dimensions in the order of microns. In engineering practice, it is common to designate the fraction smaller than 2 μm (0.002 mm) as *clay*. Although most of this fraction will consist of clay particle aggregates, other non-clay components can also be present, particularly very fine-ground quartz and feldspars.

The consistency of a clay can be described by its unconfined compressive strength, q_u, or by its undrained shear strength, s_u ($=q_u/2$). Terzaghi & Peck (1967) define as *very soft* an unconfined compressive strength of less than 25 kPa and as *soft* a strength between 25 and 50 kPa. In this chapter, we shall use the term *soft clay* for clayey soils with an undrained shear strength, s_u, of less than 40 kPa.

In this chapter, the various depositional environments are described in which fine-grained soils can accumulate. After its formation, a clay sediment is subjected to physical and chemical processes which leave a distinct imprint, and the most recent geological history largely determines the engineering

behaviour of a soft clay deposit. These processes and their effects are examined in this chapter. Clay mineralogy and the physico-chemical aspects of clays are outside of the scope of this review, and the reader is referred to the excellent treatments of those topics by Grim (1968), Millot (1970), van Olphen (1963) and Mitchell (1976).

Although soft clay deposits occur all over the world, the literature relevant to the engineering geology of soft clays describes mainly the deposits in Scandinavia (particularly Norway and Sweden) and the quick clays of eastern Canada. In these regions, the nature of these deposits has led to some spectacular failures, the analyses of which have contributed significantly to the development of soil mechanics.

2.2 FORMATION OF CLAYEY SEDIMENTS

2.2.1 The clay mineral cycle

Clay minerals are essentially hydrous aluminium silicates which contain Mg, K, Ca, Na and also Fe-ions. They can be broadly classified into four major groups, namely kaolinite, smectite (montmorillonite), illite (hydrous mica) and chlorite. A more complete discussion on these and other clay minerals can be found in the book by Grim (1968). Clay minerals are usually the products of rock weathering. They are formed by the chemical breakdown of other silicate minerals, and are then transported to the location of sedimentation by various agents. The types and the amounts of clay minerals formed are largely functions of climate, parent material, drainage pattern (topography) and vegetation. Climate is regarded as the most important factor. Transportation of weathering products can take place in the form of particles (detrital) or in the form of ions which have been leached from rocks by percolating water.

The clay particles in a sediment can have three kinds of origin (Millot, 1978). The most common origin is *inheritance,* which means the detrital accumulation of previously formed clay minerals which pass into the sediment without any modification. The second possibility is *transformation,* where previously formed clay particles are altered according to geochemical changes in the environment. Two types of transformation are distinguished, namely: (1) *degradation,* which is the removal of ions from the clay mineral lattice and (2) *aggradation,* which is the addition of ions. The third mechanism of clay mineral origin in the sediment is neoformation or *authigenesis,* which is a rather rare process and involves the in-situ crystallization of new clay minerals from ions present in the depositional environment. A typical clay mineral which can form by neoformation is glauconite, but also montmorillonite is possible.

After deposition in a certain environment, the clay sediments may undergo various diagenetic changes depending on the pressure and temperature of the

environment. Young sediments, such as soft clays, have undergone little diagenesis when they become subaerial due to isostatic uplift or marine regression, whereas sediments which are covered by a great overburden, consolidate and dehydrate, and particles become cemented. Further increases in temperature and pressure lead to metamorphism, in which process the clay minerals are destroyed and new minerals (mica and feldspars) are formed. Tectonic processes or volcanic activity can bring the metamorphosed material back to the surface where, first by retrograde diagenesis (Millot, 1978) and then by weathering of the exposed rock, the formation of clay minerals starts anew.

2.2.2 Classification of depositional environments

A depositional environment may be defined as a geomorphic unit in which net sedimentation occurs over a span of time. A number of ways have been proposed to classify depositional environments, and various parameters (physical, chemical or biological) can be employed for their characterization. Among the more important variables are climate, availability and type of sediment, water salinity and hydrodynamic conditions. A depositional environment can further be divided into subenvironments; for example, an intertidal flat can be differentiated into channels, mud flats, sand flats, mixed flats, etc (Reineck & Singh, 1975).

For each depositional environment, it is useful to identify the medium of deposition (e.g. fresh-water, marine or subaerial), the process leading to the deposition of the sediment (e.g. river, wind, waves, tides), and the location of deposition (flood plain, desert, beach, tidal flat). From the various factors

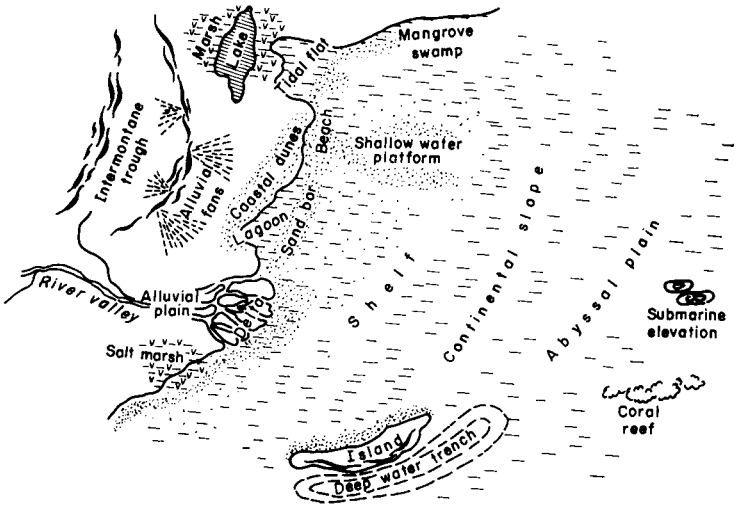

Fig. 2.1. Occurrence and types of depositional environment (Kukal, 1971).

characterizing a depositional environment, it is obvious that these environments will differ in space and time and, therefore, different sediments will be formed. A uniform deposit of soft clay, for example, reflects an environment which has not changed over a long period of time. Figure 2.1 shows schematically a variety of depositional environments.

2.2.3 Rates of sedimentation

In environments in which clayey sediments can develop, such as tidal flats, bays and lagoons, the rate of sedimentation varies irregularly. Thus, in the Gulf of Paria in Central America, van Andel & Postma (c.f. Kukal, 1971) found a deposition rate of 1 m/1,000 years in the central parts of the Gulf, of 0.2–0.5 m/1,000 years on average, and of almost zero near the straights.

In tidal flats, deposition is highly irregular; in the Netherlands, for example, average rates of 1–20 m/1000 years have been reported. In such environments, periods of rapid sedimentation alternate with periods of slower deposition or non-sedimentation, and measurements of sedimentation rates over short periods of time always yield much higher average values than observations made over longer time durations. The time of non-sedimentation is always longer than the time of deposition.

Some of the highest deposition rates occur in the deltas of major rivers, averaging 30–100 m/1000 years. For example, McClelland (1967) gave deposition rates in the Mississippi delta of 150 m/1000 years. The rate of deposition varies in different areas of a delta. It increases towards the mouth of the river, and reaches a maximum near the top of the foreset slope (Scruton, 1960). Values of sedimentation rates in various depositional environments have been tabulated by Skempton (1970) and Olsen (1978).

2.3 FACTORS WHICH AFFECT SEDIMENTATION

The sedimentation of soil particles from suspensions is controlled mainly by the factors: (1) current velocity and wave energy, (2) amount and composition of suspension load, (3) salinity, (4) organisms, and (5) organic matter. In the following sections, these factors will be described in more detail.

2.3.1 Current velocity and wave energy

Differences in wave energy and current velocity in a depositional environment can cause significant differences in the textural composition of the sediments. Textural analysis of sediments can therefore provide clues to the nature of the depositing medium. Fine-grained soils, which are of interest here, are deposited in quiet water, where individual clay and silt particles or aggregations of clay particles settle slowly out of suspension. Figure 2.2 shows a qualitative graph derived from experimental data which indicates the velocity at which a grain will be eroded, transported or deposited. This form of graph was originally proposed by Hjulström (1935), but Postma (1967)

recognized the influence of cohesion and degree of consolidation as important factors for grain sizes smaller than about 0.1 mm. The family of curves in the left portion of Fig. 2.2 demonstrates that cohesive fine-grained sediments when consolidated can resist erosion by high velocity water as well as coarse-grained sediments can.

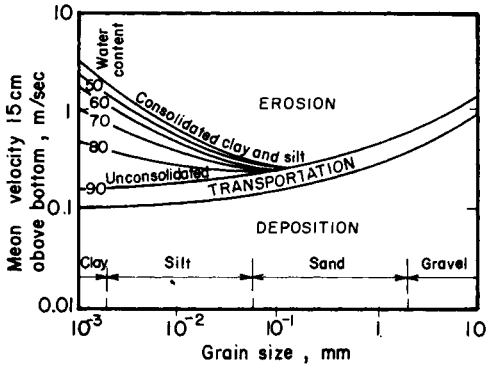

Fig. 2.2. Erosion, transportation and deposition velocities for different grain sizes, with indications of possible values for various stages of consolidation (Postma, 1967).

Several measures of sediment texture have been proposed in order to distinguish deposits of one environment from another. In one such criterion, which is called the *C-M ratio,* the coarsest fraction of a sediment is compared with its median grain size (Laporte, 1979). The coarsest fraction is defined as the grain diameter in the coarsest one percent by weight, and the median grain size is the diameter of grains in the fiftieth percentile of the sample. Figure 2.3 shows a double logarithmic plot of these two grain size parameters, and it is possible to identify characteristic fields of hydraulic depositional conditions. A number of other grain size parameters have been proposed to obtain information on the conditions during sedimentation; most of these, however, apply to coarse-grained sediments only.

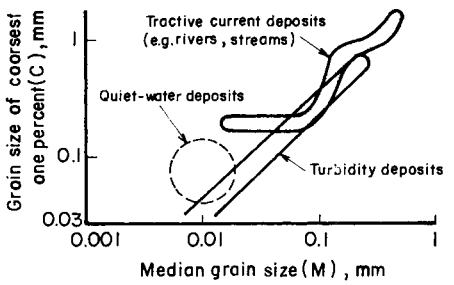

Fig. 2.3. *C-M* diagram to characterize relationship between sediment texture and depositional environment (Laporte, 1979).

2.3.2 Grain size distribution of suspension loads in streams

Kukal (1971) presented some data on grain size parameters of suspended loads in some of the large streams. The grain size median, M_d, (or D_{50}) ranges from 0.06 to 0.007 mm, i.e. within the silt-sized fraction. There is a wide variation in the amounts of the sandy and the clayey fractions. The grain size distribution of suspended material is mainly a function of topography and climate, and generalizations are difficult to make. Kukal concluded that, in about 75% of the cases investigated, rivers from mountainous areas with an irregular profile usually carry large amounts of clay and silt-sized particles but little sand, while rivers in lowland areas transport much clay and fine sand in suspension but usually only small amounts of silt. Meade (1972) stated that, in estuarine waters at low velocities, essentially all suspended particles are finer than 0.1 mm.

2.3.3 Salinity and flocculation

In saline waters, single particles in suspension are unstable and tend to lump together, forming flocs. This process, known as *flocculation,* increases the settling velocity of the suspended material drastically. If particles did not flocculate, they would not be able to settle out even in bodies of water which were only very slightly agitated. For example, a current velocity as small as 0.09 mm/sec can maintain a particle of 10 μm diameter in suspension. Even in still water, the 10 μm particle requires 3 hrs to settle 1 m (Pryor, 1975). Therefore, if clay-sized mineral particles had to settle individually, totally quiet bodies of water would be required. Such a situation rarely exists in nature, with the exception probably of a glacial lake during winter.

Flocculation is influenced by various factors, such as type of clay mineral, salt concentration of the water, organic compounds, interparticle collisions, and temperature. Interparticle collision, in turn, is governed by turbulence of the medium and the concentration of the suspension. Salinity and turbulence are considered to be the two most important factors. Most fine particles flocculate from salt concentrations as low as 0.3% (Kranck, 1973). Smaller particles flocculate more readily because of their larger relative surface area. As the flocs grow larger, particles adhere to them, and with an increasing degree of flocculation the number of collisions decreases. In a natural environment, the medium will be turbulent to a certain extent, and larger flocs can become unstable because of the internal shear in the water acting on the flocs. Kranck (1973, 1975) observed that a bigger increase in grain mode is required to produce a given increase in floc mode. Therefore, the ratio of floc mode to grain mode decreases with increase in particle size.

Regarding the influence of the type of clay mineral, Whitehouse et al (1960) showed experimentally that with pure clays, kaolinite and illite can flocculate at lower chlorinity concentrations than montmorillonite, and consequently different clay mineral species will settle at different velocities at a given

chlorinity concentration. However, Gibbs (1977) pointed out that it is incorrect to extrapolate these results to natural sediments, as turbulence effects can impart flocculation and also cause bottom erosion. Based on investigations on bottom sediments from the Amazon River/Atlantic Ocean area, he concluded that Whitehouse et al's differential flocculation theory is not a likely mechanism, because of organic and/or metallic coatings on the particles which interfere with the process of flocculation. He agreed, however, that the laterally changing composition of the clay-sized fraction should be explained by a size segregation mechanism. In such a process, sorting occurs according to the size of the particles. Montmorillonite has the smallest particle size (mean value $\approx 0.4\mu$), compared to kaolinite (≈ 1–2μ) and illite (≈ 2–4μ). This mechanism, however, assumes that aggregation of particles is not significant, either because turbulence breaks up floccules or because a lower concentration of suspended material results in a smaller number of particle collisions to form aggregates.

Most recently, Hunter & Liss (1979) have extended Gibbs's observations. Performing direct measurements in river estuaries, they demonstrated that changes in clay mineral distribution with distance from the source river can be accounted for only by differences in the particle size spectra and the resulting sedimentation velocities. They also pointed out the importance of surface coatings by metal oxides and/or organic matter around particles, which tend to suppress the influence of differing electrical and surface properties on the flocculation rate.

Flocculation is particularly effective and important in the formation of argillaceous sediments where water of radically different salinities mix, such as deltaic and estuarine environments. Flocculation leads to an open geometric arrangement of the particles. The particle contacts are either edge-to-face or edge-to-edge. Whether this configuration will be preserved when the flocs are buried under further sediment load, depends on the rate of deposition. It is speculated that a slow rate of sedimentation which allows the simultaneous development of bonds is necessary to preserve the open fabric.

2.3.4 Biogenic pelletization

In shallow marine environments, biologic activity contributes significantly to the formation of argillaceous sediments. There are large numbers of organisms that suspension feed, i.e. they pump large quantities of water into their feeding systems and extract suspended solid particles. Some of the material taken in is organic and is used as food by these organisms. The major part, however, consists of mineral (clay) particles which are not utilized and which are excreted by the organisms in their feces. The excrements, because of their shape, are called *fecal pellets*. Pryor (1975) investigated the feeding activities and excreting products of two common filter feeding marine organisms, namely the decapod *Callianassa major* (Say) and the annelid

Onuphis microcephala (Hartman) in the shallow marine environments of the southern Atlantic and eastern Gulf of Mexico coasts of the United States. An analysis of the fecal pellets showed that they were composed of 80 to 90% of clay-mineral particles, 5 to 10% of undigested organic particles, and small amounts of quartz, sand and silt grains. As a result of the digestive processes, however, the clay-mineral composition of the pellets differed considerably from that of the clay mineralogy found in the suspended clay. For example, chlorite was wholly or partly destroyed, and kaolinite and illite were in part disordered.

Fecal pellets have sizes of silt and sand range. *Callianassa* pellets, for example, are approximately 2 mm long and 0.75 mm in diameter, and about 1.4 mg in dry weight and about 2.3 mg in wet weight, but smaller sizes have also been observed. Figure 2.4 shows a sketch of a *Callianassa* fecal pellet. *Onuphis* fecal pellets are smaller and have the shape of flat ovoids.

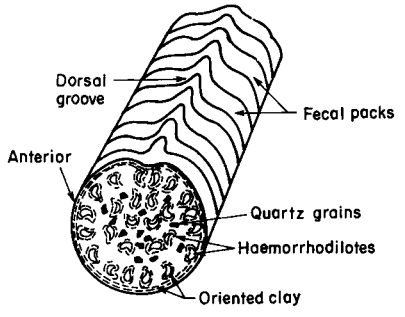

Fig. 2.4. Diagram of *Callianassa major* fecal pellet (Pryor, 1975).

Both organisms live in sand-dominated environments, and due to their size, the pellets are transported and deposited together with sand-sized grains along the bottom by traction. Clay in pelletized form can therefore accumulate in high energy environments. Pryor (1975) observed that the two organisms studied can produce fecal pellet mud as thick as 4.5 mm/yr and produce as much as 12 metric tons (dry weight) of pelleted mud per km^2 per year. He concluded that biogenic pelletization may be one of the most important processes in the deposition of shallow marine muds, particularly in inter-deltaic areas.

2.3.5 Influence of organic substances

Suspensions from which fine-grained sediments settle out contain various amounts of organic matter. Most of this organic matter consists of complex, heterogeneous, brown or yellow acidic polymers, which collectively are known as *humic matter*. Jackson (1975) reviewed the present knowledge on such materials, but there exists relatively little literature on the influence of humic

matter on the sedimentation of clay particles and on how they affect the initial structure of such sediments. Humic matter can be adsorbed on clay particles or become enclosed in the aggregates, particularly in saline solutions. Rashid et al (1972) found that, in a solution of 3.5% NaCl, clay minerals were able to adsorb more than 2.5% of humic acid as compared to 0.4% in fresh-water. Adsorption also increased with lower pH. Adsorption can have two effects which are important for the future clay structure: adsorbed humic matter may coat the clay particles and 'protect' them from flocculation, so that the particles are kept in a dispersed state; humic matter, on the other hand, can also cement the clay particles to form stable aggregates. Greenland (1971) believed that the hydroxides of polyvalent metals at the clay particle surfaces, such as iron and aluminium, can become polymerized, and strong bonds between clay and organic materials develop. Flocs of sufficient size to settle out are thus created. The deposition of clay as clay-organic aggregates might therefore be as common or even more common in natural systems than flocs caused by salt flocculation (Meade, 1972; Frey & Basan, 1978).

The occurrence of organic matter in recent clay deposits is of great importance to an understanding of some of the causes of clay sensitivity, as will be shown in Section 2.10.3.

2.4 DEPOSITIONAL ENVIRONMENTS OF SOFT CLAYS

2.4.1 Fluvial environment

Sedimentary deposits laid down by a river are known as *alluvium*. While delta deposits also consist partly of river sediments, they are discussed here separately, as they are primarily subaqueous and not subaerial deposits. One possible classification of river depositional environments is that suggested by Kukal (1971):

(1) River channel, which includes the areas in the proximity of the stream line, shoals at the margin of the channel, and point bars.
(2) Natural levees (bank deposits), which consist of all deposits which have been formed by lateral flooding of the channel.
(3) Flood plains (overbank deposits), which include sediments deposited during floods above the channel, and bank sediments.

In the lower reaches of a stream, close to its mouth, alluvial deposits can attain great thickness and extent, and are then often called *alluvial plains*. The bulk of such alluvial plain deposits consists of flood plain deposits laid down during the periods of high flooding. Sediments of the alluvial plain pass into the river delta, and the transition is gradual, and separation into the terrestrial and marine environments is often difficult.

Most river deposits are coarse-grained, and these will not be discussed further. Fine-grained deposits, however, also accumulate in flood plains, and

typical grain size distributions have been reported as: sand 5–10%, silt 20–40%, clay 35–60%. The median particle size, M_d, of flood plain deposits varies between 0.005 and 0.06 mm. Organic material ranges from 1 to 10%, and this originates mostly from detritus which has settled from suspension with the clay particles.

Flood plain deposits are all characterized by bedding and laminations, sometimes intercalated with fine sand. Thick homogeneous deposits of clay are therefore not encountered. Extensive flood plain deposits, for example, occur on the Mississippi alluvial plain. The history of deposition of such clays further implies that the cycles of wetting and drying to which these deposits are subjected lead to post-depositional changes, such as desiccation, weathering and the formation of shrinkage cracks. Flood plain deposits, therefore, do not remain 'soft', as they are subject to successive desiccation in thin layers within a few months after each layer has been formed (Terzaghi, 1955).

Terzaghi also pointed out that, in contrast to flood plain clays, a clay which was sedimented in an oxbow lake, which is a channel-fill deposit, usually remains in a normally consolidated state with a liquidity index close to unity. The oxbow deposit then dries out at the surface only, the clay underneath the crust remaining soft. Later on, flood plain deposits may cover the crust, and the soft layer consolidates under the influence of gravity only. Oxbow deposits are therefore highly compressible soils embedded in stiff clays and silts, and they can cause problems of differential settlement when not properly detected in a site investigation.

2.4.2 Lacustrine and glacio-lacustrine environments

Lacustrine sediments have probably the most varied composition of all deposits, as they contain not only clastic sediments but, in many cases, considerable amounts of chemogenic sediments (carbonates, evaporites) and organogenic sediments (gyttja, sapropel, etc). The classification of lakes can be based on various criteria, such as climatic zone, origin, drainage conditions, hydrochemical properties and hydrobiology. The most common division is into the three groups of oligotrophic, eutrophic and dystrophic lakes, according to the quantity of oxygen and nutrients present (Kukal, 1971).

Clastic sediments dominate in oligotrophic lakes (i.e. lakes with high oxygen but low nutrient content). The grain size distribution is controlled largely by the hydrodynamical conditions of the lake, topography and depth of the lake bottom, as well as by the relief of the environment and the supply of materials by the affluents. Clays are usually quite abundant in lacustrine sediments. In general, materials at the edges of lakes are coarser grained.

Of considerable interest to geotechnical engineers are the clayey deposits which were laid down in proglacial lakes of North America and northern Europe after the last glaciation. A considerable literature on the engineering properties of such glacio-lacustrine clays exists. Woods et al (1962) described

the distribution of lacustrine clays in the United States. Large areas of clayey glacio-lacustrine deposits occur in the Central and Eastern Lakes and the Western Lakes sections of the Lowlands Province. Prominent among these are the sediments of ancestral Lake Agassiz and Lake Chicago. The situation in the Chicago area has been described by Peck & Reed (1954), while data on Lake Agassiz deposits can be found in the paper by Rominger & Rutledge (1952). Wu (1958) reported geotechnical properties of glacial lake clays from four locations in the Great Lakes area. The major proglacial lake system in Canada and its significance in the formation of soft soils has been outlined by Quigley (1979). Figure 2.5 shows the enormous areas of Canada which were covered by such water bodies, both fresh and marine.

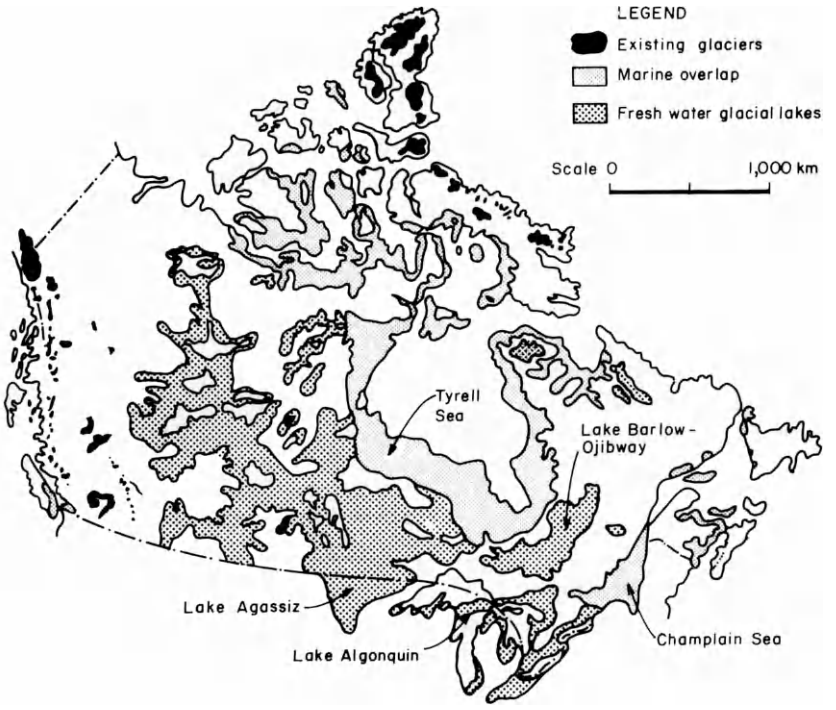

Fig. 2.5. Distribution of marine and fresh-water glacial and postglacial lakes in Canada (Quigley, 1979).

Glacial lake deposits are often composed of light-coloured materials of mainly silt size, alternating with darker layers containing mainly clay. A clay deposit with such a statification is called a *varved clay*. Figure 2.6 shows the grain size, water content and consistency variation within a single varve as determined by Eden (1955). A single varve represents the deposition of one year, i.e. it consists of summer silt and winter clay. On average, the summer

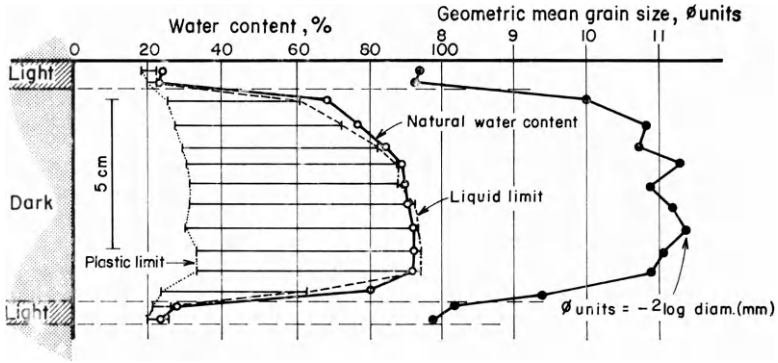

Fig. 2.6. Variation of grain size, water content and plasticity within a single varve from Steep Rock Lake, Ontario (Eden, 1955).

layer has a grain size > 2μm of about 80%, and the clay-rich winter layer has a clay-sized fraction (< 2μm) of approximately 80%.

The most important role in the formation of varved clays is ascribed to turbidity currents (Kuenen, 1951; Gilbert, 1975; and others). Kenney (1976) distinguished three major types of lake currents which control the formation of varves. These are: (1) heavy density turbidity currents, (2) interflows, and (3) low density overflows. The first mechanism, illustrated in Fig. 2.7a, has been described by Quigley (1979). A cold stream with a heavy sediment load (≈ 1 g/litre or more) enters the lake. The density of the flow is higher than that of the lake water (which, however, is still around its maximum density, because the temperature is between 0° and 6°C). Such a heavy density current may flow over great distances along the lake bottom and deposit a graded bed

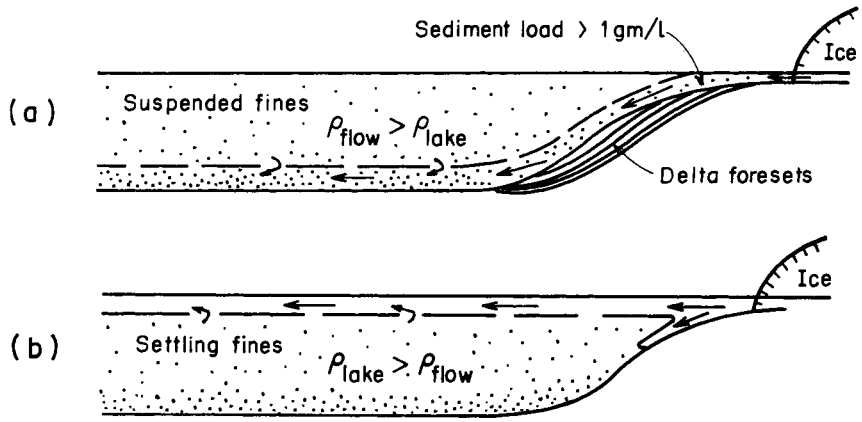

Fig. 2.7. Varved clay sedimentation in a fresh-water proglacial lake: (a) summer layer deposition by heavy density turbidity currents, (b) winter layer deposition by settling of fines from low density overflow (Quigley, 1979).

of sand and silt. During winter, the sediment load of the incoming stream is reduced, and the suspension has a lower density than the lake water. Low density overflows are thus characteristic of winter conditions, as shown in Fig. 2.7b. Clay particles under these conditions will settle out slowly and relatively undisturbed.

In postglacial lakes which are no longer in contact with the ice front, the suspended sediment loads are greatly reduced (normally < 0.1 g/litre). Quigley (1979) stated that, under these conditions, streams enter lakes as overflows and interflows, and heavy density bottom flows only occur as products of submarine slumping or during erosive floods. Deposition of the sediments then takes place by year-round settling, silt and fine sand settling out during summer and clay during winter.

A very unusual lacustrine soft clay deposit of engineering significance is Mexico City clay, which underlies most of urban Mexico City. According to Zeevaert (1972), the closed basin of the Valley of Mexico was formed at the end of the Pliocene epoch. A large lake occupied practically the entire basin which was first filled by water-transported materials from the surrounding hills. These materials formed a series of gravel, sand and silty clay deposits several hundred metres thick. Overlying these deposits is the soft Mexico City clay (Tacubaya formation), which in the centre of the city extends from about 7 m to 33 m (Fig. 2.8). It dates from the late Pleistocene and originates from volcanic ash and other pyroclastic materials which were transported by wind and water into the lake.

Typical geotechnical properties of Mexico City clay are shown in Fig. 2.8. Remarkable are the extremely high water contents (sometimes higher than 500%) and Atterberg limits. In a recent detailed investigation (Mesri et al, 1975), it was found that the sand-sized fraction (5%–10%) in the clay consists primarily of concretionary particles or ooliths composed of calcium carbonate. The silt-sized fraction (55%–65%) appears to be composed of siliceous shells and shell fragments of diatoms, while the clay-sized fraction (20%–30%) constitutes 10% interlayered smectite together with biogenic or volcanogenic silica. Only 5%–10% of the soil is taken up by organic material. It appears, therefore, that the volcanic ash particles released great amounts of silica into the lake water which initiated the bloom of diatoms. Kukal (1971) stated that diatoms can originate at SiO_2 concentrations as low as 1 to 5 ppm, and optimum development occurs at 5 to 20 ppm of dissolved SiO_2. The highly porous shells of the diatoms are able to hold large quantities of water, and it is believed that the diatom content in the soil is largely responsible for the unusual geotechnical properties, such as high plastic limit, low specific gravity (2.35), and a relatively high friction angle.

2.4.3 Deltaic environment

When a river reaches the sea, its current velocity drops rapidly and, consequently, the sediments carried along by the river's flow are deposited at

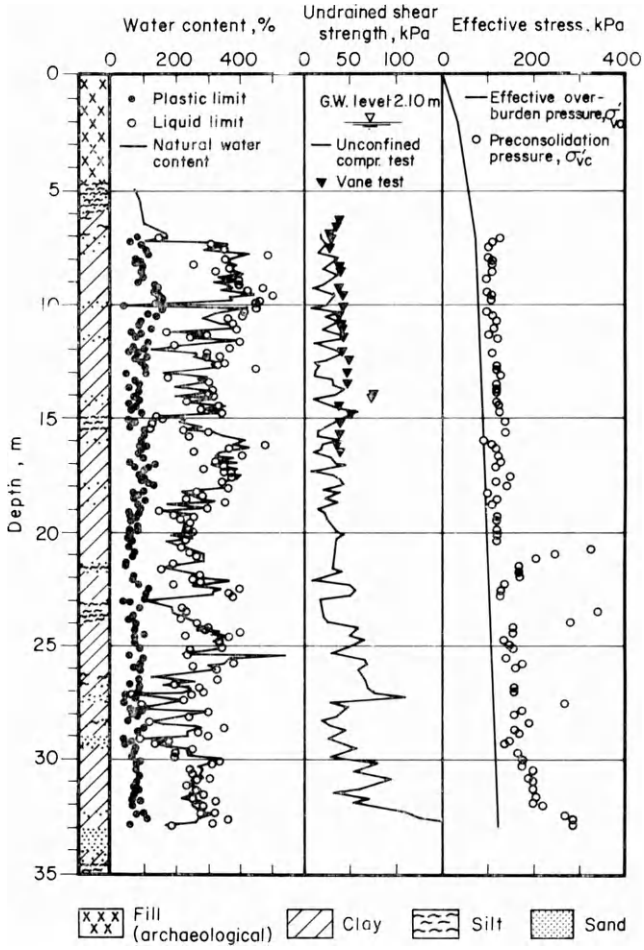

Fig. 2.8. Typical geotechnical properties of clay in Mexico City (Marsal, 1957).

its mouth. The sediment body built up in this way, and which is formed at the boundary between continental and marine (or lacustrine) environment comprises a *delta*. The coarser part of the suspended load is deposited close to the river mouth, while the fine-grained portion is settled well off-shore where bottom currents are weak. Deltaic sediments of major streams extend over the entire shelf and even over the continental slope. Near the influx of the fresh-water into the sea, a water layer of lower density can form at the surface, since river water, even when loaded with suspended material, has a lower density than sea-water (for which the density is typically 1.027 Mg/m^3). This process plays an important role in transporting the river suspension to distant parts of the sedimentary basin.

Factors which govern the depositional processes and the deposits of a delta are: (1) river regime (amount of suspension, manner of influx), coastal processes (subsidence, relief of coast, width of shelf), (2) presence of currents, i.e. energy conditions (tidal, turbidity, etc), and (3) climatic factors (Kukal, 1971; Reineck & Singh, 1975). Deltas have been classified into various types, such as *birdfoot* (e.g. Mississippi), *lobate* (e.g. Lena), *cuspate* (e.g. Nile), *arcuate* (e.g. Niger) and *estuarine* (e.g. Ganges) (see, for example, Allen, 1978). Birdfoot deltas are most likely to contain major deposits of fine-grained materials (Way, 1978).

Although large deltas are very complex sedimentary structures, usually three basic components can be distinguished. These are the *topset*, *foreset* and *bottomset* structures (Fig. 2.9). Another subdivision of the delta is into a *subaerial* and a *subaqueous* part. The subaerial part typically consists of a lower delta plain and an upper delta plain. The subaqueous part of the delta is that portion which lies below the low tide water level. The seaward-most portion of the subaqueous delta is composed of the finest material, and is referred to as the *prodelta*. Prodelta clays have lateral continuity and low lithologic variation, and in many cases they exceed 6 m in thickness (Wright, 1978). In large deltas, distinctive topset and foreset beds are difficult to recognise, and the terms *prodelta, intradelta* and *interdistributary environment* have been introduced, largely for the Mississippi delta, which is probably the best studied delta in the world (see, for example: Fisk et al, 1954; Kolb & Kaufman, 1967; Kolb & Van Lopik, 1958). The term *prodelta* is somewhat synonymous to *bottomset*.

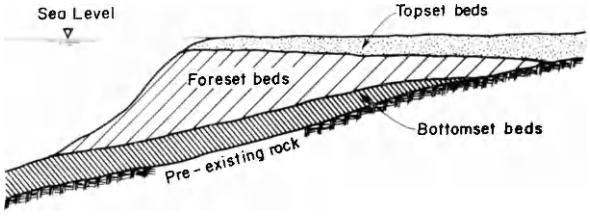

Fig. 2.9. Components of a delta body.

Prodelta clays are usually highly plastic and of homogeneous appearance. Shell remains are common throughout. In reality, however, as revealed by X-ray radiographs, these clays are thinly laminated, and they often show fractures, contorted bedding and complex displacements (Kolb, 1978). The fractures and distortions of the layers are thought to be a function of the rate of deposition. The denser coarse-grained materials which cover the low-density prodelta clays when the delta advances, generate differential loading which leads to peripheral slumping, radial tensional faulting, diapirism and deep-seated clay flowage (c.f. Wright, 1978).

The topset beds of a delta or the interdistributary zone of a delta are partly subaerial and partly subaqueous, and the sediments are very heterogeneous. The subaqueous part consists of shallow bays or intertidal flats, depending on the range of the high tides. Muddy sediments are deposited between the distributaries and the tidal channels, but these are frequently interbedded with crevasse splay deposits (Wright, 1978). The subaerial portions are characterized by marshes and swamps which form highly organic deposits, i.e. organic clays or peats intercalated with fine clastics.

Large deltas, therefore, contain a number of subenvironments, both terrestrial and marine, and deposition takes place in water with a salinity which varies from that of fresh-water to that of sea-water. Deltas in combination with large coastal plains, therefore, are often locations of major deposits of soft marine clay. Good examples are the flat deltaic lowlands of Southeast Asia, which can be found along coasts with low wave energies. Major deltaic plains have developed at the outfalls of rivers served by large catchments, such as the Irrawaddy, Chao Phraya, Mekong, Kelantan and Pahang Rivers. The formation and distribution of recent sediments in the Southeast Asian region was reviewed by Cox (1970a). The marine clays are in general light-to-dark grey, soft, silty clays, which have attained thicknesses between 5 m and 30 m at the coast-line.

2.4.4 Bays and lagoons

A *lagoon* is a shallow water body between a barrier island or sand-bar and the mainland, and is parallel to the coast and connected to the open sea by one or more outlets (Pickett, 1978; Reineck & Singh, 1975). A *bay* is a recess in a coast-line usually well protected from near-shore currents. From a sedimentological point of view, lagoons and bays are very similar. Even from a geomorphological point of view, the boundaries between the two are not sharp, lagoons being more closed than bays. The water in bays and lagoons may be brackish, normal or even hypersaline.

The composition and distribution of bay and lagoon sediments depend on such factors as: (1) size and depth of bay, (2) width of connecting straits, (3) inflow of river water to the bay (amount of material brought in, effect on salinity, hydrographic conditions), and (4) topography of coast. The last factor affects mainly the vertical distribution of coarse-grained sediments (Kukal, 1971). Fine-grained sediments are usually predominant in deeper bays. Silty clays and clays are mainly deposited in extensive lagoon bottoms, whereas in the channels where the velocity is higher, sandy deposits predominate.

Three fundamental sedimentary structures or environments have been distinguished in lagoonal deposits (Stewart, 1956), namely:

(1) One which is fine-grained and homogeneous, found in the middle of the lagoon or in a river-mouth entering the lagoon.

(2) One which is regularly banded or layered, found near the barrier islands, and which is sandy.

(3) Muddy organic-rich sediments, which are stratified and mottled due to secondary disturbance produced either by burrowing organisms or by the escape of air bubbles from water-logged clayey sediments.

Detailed studies on lagoonal sediments have been reported from the Gulf Coast (Rusnak, 1960; Lankford & Rogers, 1969; etc) and from the Atlantic Coast (Pickett & Ingram, 1969; etc). Geotechnical profiles of typical lagoonal deposits in Italy have been given by Calabresi & Manfredini (1976).

In tropical regions, lagoonal sedimentation is often marked by the deposition of predominantly calcium carbonate sediments, but this occurs only if there is a deficiency in the supply of terrigenous clastic material. Carbonate deposition is favoured by warm marine waters and low relief coastal environments. The carbonate is supplied from the skeletal remains of dead calcareous marine organisms. More information on these types of sediment in tropical lagoons can be found by reference to Jordan (1978).

2.4.5 Tidal flats and marshes

Tidal flats are defined as depositional environments along flat coasts which are subjected to marked tidal rhythms. They can be found where spits, bars, barrier islands, etc provide protection from waves. During low tide, much of the area lays bare, while it is flooded during high tide. The constant flow of water both in and out of the tidal flat develops an intricate system of drainage channels that characterizes this landform. Tidal flats are sometimes divided into *supratidal*, *intertidal* and *subtidal* parts, the latter being below the low water-line.

The supratidal zone is vegetated by halophytic plants. In humid tropical areas, mangrove vegetation is common, since mangrove can exist in brackish, normal marine or weakly hypersaline waters. This zone and part of the intertidal zone can be termed (salt) *marsh*. They are covered by water only a few times each year, either during exceptionally high tides or, for marshes which are part of a delta, during river floods. The sediments of the intertidal zone are mostly fine-grained, but it is still possible to differentiate this zone into *mud flats, sand flats* and *mixed flats*. Sand flats are found near the tidal inlets, where currents are relatively strong.

Processes which promote the deposition and accumulation of fine-grained sediments in shallow water by tides have been described by van Straaten & Kuenen (1957, 1958) and Postma (1967), who explained them as lag effects. Two lag phenomena must be considered. When a current over a tidal flat slackens, a particle which starts to settle will not be deposited below the point where it started to drop out but will be carried some distance beyond this point. This is called *settling lag*. When the flow reverses, a higher current

velocity (according to Fig. 2.2) is needed to erode the same particle. This phenomenon is called *scour lag*. Because of a decrease in tidal velocity landward, there is a distance velocity asymmetry which results in a net landward transport further onto the tidal flat with each tidal cycle.

Sediment will also be trapped by the vegetation of the marsh area. Marsh sediments are usually highly organic and poorly sorted. Physical structures of marsh sediments show discontinuous, wavy-to-parallel laminations, or continuous laminations and thin bedding, while biogenic structures involve root structures and burrowing animals. More details on salt marsh environments can be found in the review by Frey & Basan (1978).

Tidal flat areas, as subenvironments of large deltas, are probably the most significant depositional environments in which today's marine soft clays have been formed, particularly in tropical and subtropical areas. Typical descriptions of tidal flat sedimentation have been given by Evans (1965), van Straaten (1961), Pestrong (1972), Ragotzkie et al (1959) and many others.

2.5 SEA-LEVEL CHANGES AND ISOSTATIC UPLIFT

2.5.1 Epochs of the Quaternary period

The Quaternary period was characterized by the occurrence of ice ages and resultant drastic climatic changes, and by world-wide sea-level fluctuations. This period is divided into the *Pleistocene* epoch, which is the time during which the ice ages occurred (in Germany often called *Diluvium*) and the *Holocene* or *Recent* or *Postglacial* epoch, which covers the time span from the last glaciation to the present. In Northern Europe, the last glaciation was the Weichsel stage (or the Würm stage in the Alps), which reached its maximum at about 17,000 to 20,000 B.P. (B.P. = before present), and the Holocene covers about the last 10,000 years. In northern North America, however, the postglacial epoch, began only about 6,000 years B.P., i.e. with the retreat of the Wisconsin ice-sheet. By general consensus, the Holocene/Pleistocene boundary is taken by most researchers at about 10,500 to 10,000 B.P., although no international agreement has been reached yet.

All terrestrial and near-shore soft clays are of late Pleistocene or Holocene age. In previously glaciated regions, they were deposited in proglacial lakes, river valleys and basins invaded by the sea, or in deep fjords while the ice sheets were retreating, and the land rose. In non-glaciated areas, soft marine clays were deposited in deltaic coastal plains into the transgressing sea.

2.5.2 Fluctuations in sea-level

During the ice ages, the continental ice-sheets expanded and contracted repeatedly, and sea-levels correspondingly fluctuated down and up. It has been suggested that the world's sea-levels were at a maximum of about 137 m below the present levels during the last (Wisconsin) glaciation stage, and that

the potential sea-level rise during an interglacial was approximately 60 to 75 m above the present sea-level (Kraft, 1978). Values given by other authors differ somewhat (e.g. Kenney, 1964, gives 119 m below the present level as the lowest sea-level elevation). Of particular interest is the rise in sea-level over the last 20,000 years, i.e. when the Wisconsin ice-sheet started to melt. The associated marine transgression is known as the *Flandrian transgression*.

Many researchers have tried to establish time—depth curves for eustatic sea level rise. This can be accomplished (1) from the age of fossil remains of plants or shelled animals which lived and/or were deposited near the sea-level, using the C^{14} dating method, and (2) from geomorphologic features, such as emerged strandlines or submerged terrestrial features on the continental shelf. For example, the presence of an erosion surface in the soil stratigraphy indicates a lowering of the sea-level with respect to land, and a subsequent deposition above the erosion surface indicates a rising sea-level with respect to land. Fluctuations in sea-level can also be inferred from palaeoclimatic records in that, for example, the sea-level will rise during warmer periods. Evidence on climate and temperature oscillations can be found from the analysis of former lake levels in now arid areas and from studies of pollen profiles (Kenney, 1964).

The C^{14} dating of fossils is considered to be the most reliable method of dating, provided the data come from relatively stable coasts. An example of a sea-level versus time relationship is given in Fig. 2.10. The discrete points in this figure are all based on radiocarbon dating results. The continuous curve is that of Kenney (1964), which is based on the various geological and climatological evidence mentioned above. On this curve, the rise in sea-level prior to 10,000 years B.P. is marked by two major regressions. These have been explained by glacial readvances or substages. The sea-level curve also exhibits periods of standstill, which have been deduced from the formation of marine terraces.

The sea-level rose sharply until about 6,000 B.P., and then it remained at a more or less constant level. During the last 3,000 years, the rate of sea-level change was about 35 mm/100 years (Flint, 1971). At present (1900–1950) the average rate of sea-level rise is 1.2 mm/year (Fairbridge, 1966). The sea-levels 3 m to 4 m higher than the present level, shown by the points for Australia in Fig. 2.10, are disputed (Guilcher, 1969). However, beaches have been discovered 3 m to 5 m above the present sea-level on the supposedly stable west coast of Malaysia, and dated about 5,000 years B.P. (Haile, 1971), and in the Ayuthaya area of Thailand, of estimated age about 3,000 years (Rau & Nutalaya, 1980). These discoveries indicate that a somewhat modified time—depth curve might apply for these areas.

Recently, attempts have been made to model the relative sea-level change by taking into account the interaction between ice loads, water loads and a deformable earth (e.g. Clark et al, 1978). This is illustrated in Fig. 2.11, where (a) the weight of the ice deforms the earth, (b) the ice mass attracts

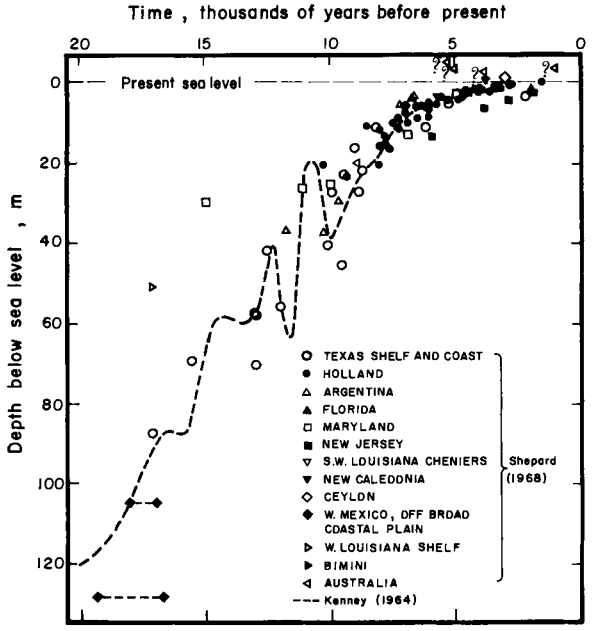

Fig. 2.10. Sea-levels during the past 20,000 years as determined by C^{14} dated organisms from relatively stable coasts (Shepard, 1968), and the eustatic sea-level curve composed by Kenney (1964).

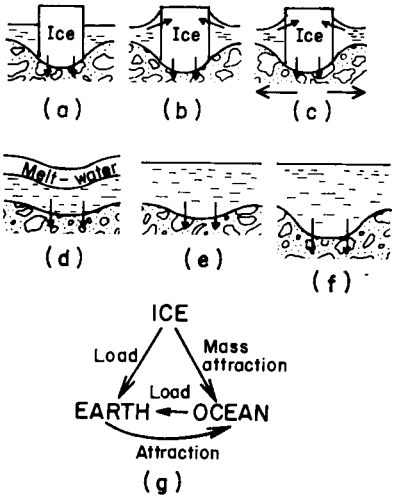

Fig. 2.11. Interaction among ice loads, water loads and the deformable earth (Clark et al, 1978).

the water, (c) the transfer of matter deforms the geoid, (d) the weight of the meltwater depresses the earth differentially, (e) more water flows into this depression, which (f) causes additional deformation of the ocean floor. Figure 2.11g indicates the interrelationship. Results of such models have shown that the sea-level rise due to ice-sheet melting was not uniform everywhere, and that zones of emerged beaches and regions where submergence is dominant can be predicted. The deformation in the geoid can be of the same order of magnitude as the oceanwide 'eustatic' rise in sea level. Clark et al (1978) concluded, therefore, that there are no 'stable' regions where eustatic sea-level can be measured, because deglaciation and the addition of water to the ocean basin deform the earth and change the observer's reference point.

2.5.3 Glacial isostatic deformation

The Scandinavian ice-sheet in Europe, and the Laurentide ice-sheet in northern North America, reached great thickness during the maximum of the last ice age. Large ice-sheets caused crustal warping because, it is believed, their loads exceeded the strength of the material of the upper mantel beneath the crust. This situation causes the crust beneath the ice to subside in basin-like form. The deformation is partly elastic but mainly plastic, and flow within the mantle transfers material outwards away from the basin depression to compensate for the extra load. With deglaciation, this process is reversed, and the crust bulges up (Flint, 1971). The outer limit of a region deformed by an ice-sheet can be deduced from the observation of strandlines.

It is estimated that both the Laurentide and the Scandinavian ice-sheets at their maximum were about 3 km thick in their central part. For the Laurentide ice-sheet, the total rebound in the Hudson Bay region since retreat started about 18,000 B.P., is about 800 m, of which about 200 m have taken place since deglaciation in 8,000 years B.P. (Quigley, 1979). For Scandinavia, the situation is similar. For example, around the Oslofjord, land uplift amounted to about 220 m relative to the present sea-level (Bjerrum, 1954). Today, in central Scandinavia and central Canada, the rise at the locations where the ice reached its greatest thickness may exceed 10 mm/yr (Fairbridge, 1966).

If it is possible (by means of radiocarbon dates and/or other pertinent geological information) to construct a curve of absolute crustal movement versus time for a certain location, then this can be compared to the eustatic sea-level curve. Following such a procedure, Kenney (1964) was able to analyze periods of sedimentation (submergence) and erosion/subaerial weathering (emergence) at various sites. Thus, for the Boston basin, he identified three deposition/erosion cycles. This region was at the edge of the ice-sheet and became free of ice before 14,000 B.P. Rebound was less than farther north, where the ice-sheet was thicker. Hence, the flat rebound curve intersects the transgression peaks of the sea-level curve at about 12,200 and 10,800 B.P. Kenney (1964) pointed out that such studies can be helpful in the evaluation of

geotechnical test data of a deposit. If there is a disagreement with the history of its formation with respect to deposition, erosion or weathering, it may be inferred that this was caused by post-depositional chemical processes.

2.5.4 Migration of the shoreline

The depositional environment of sediments in coastal areas is intimately connected to the movements of the shoreline. The migration of the shoreline depends mainly on two factors, namely the rate of deposition and the rate and direction of sea-level changes. A rising sea-level will in general result in a landward migration *(transgression)* of the shoreline, and a seaward migration *(regression)* will result from a falling sea-level. This tendency, however, may be reversed by effects of net deposition and erosion. If, for example, in a delta area of a coastal plain a slowly rising sea-level is associated with a high rate of net deposition, then a shifting of the shoreline in the seaward direction will occur and will appear as a regression. The interaction of these two processes is diagrammatically shown in Fig. 2.12, and examples were given by Curray (1964).

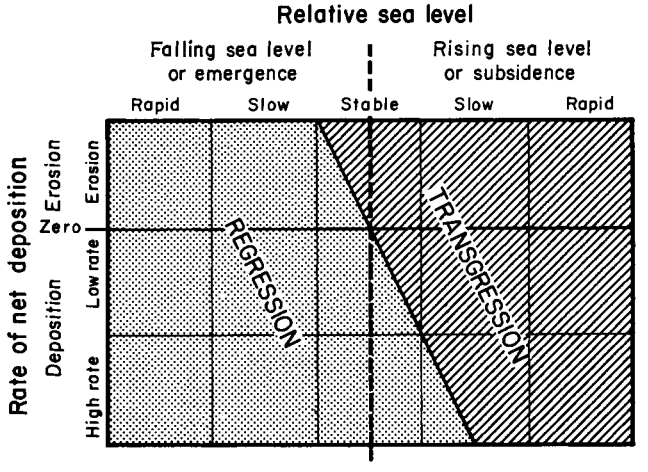

Fig. 2.12. Effects of the rate of change of relative sea-level and of the local net rate of deposition on lateral migration of the shoreline in a direction perpendicular to the coast (Curray, 1964).

2.6 FABRIC AND STRUCTURE OF CLAYEY SOILS

2.6.1 Definitions and types of particle association

A study of soil structure can reveal a great deal about the engineering geology of a deposit. An understanding of soil fabric is thus essential in the interpretation of post-depositional changes. It will be seen later that such post-depositional changes mainly influence the stability of a fabric. For

engineering purposes, we usually investigate samples in which the clay has already undergone certain geological processes (e.g. consolidation due to overburden). There is very little information on the variation of soil fabric during the consolidation of a natural, freshly sedimented clay.

The term *fabric* is usually applied to the physical arrangement of soil particles and particle groups, and includes particle spacing and pore spaces (or pore size distribution) (Yong & Sheeran, 1973; Mitchell, 1976). Usually, two levels of fabric are distinguished, namely: (1) *macrofabric,* which is the level of fabric which can be observed with the naked eye or hand lens, and (2) *microfabric,* which is the level of fabric requiring observation with at least a polarizing microscope. The term *structure* should be used in a much broader sense, and should include the combined effects of fabric, composition and interparticle forces (Mitchell, 1976). Yong & Sheeran (1973) defined structure as "that property of the soil which provides for its integrity". Often, however, the terms *fabric* and *structure* are used interchangeably.

While it is mainly the microfabric which has received most attention, macrofabric can exert a dominant influence in many engineering problems (Barden, 1972). In the case of soft clays, important macrostructural features are horizontal layering, sand or silt seams, root holes, syneresis cracks, old failure plains, organic inclusions, etc. This section will be devoted to aspects of microfabric and microstructure only.

The development of soil fabric models has been reviewed by Moon (1972) and Collins & McGown (1974). The most important event was probably the introduction of the scanning electron microscope in the late 1960s which, in combination with improved sample preparation techniques, allowed a three-dimensional insight into particle arrangement. Inspite of the excellent means available today for investigating soil fabric, there still exists no universally accepted system for its classification. On the other hand, a rather bewildering number of terms have been proposed to describe soil fabric features. Van Olphen (1963) classified possible particle arrangements in suspensions as shown in Table 2.1. In natural deposits, however, single clay platelet arrangements (such as edge-to-edge or edge-to-face 'cardhouse' type arrangements of clay platelets) are rarely found, whereas groups of clay platelets (e.g. with stepped face-to-face aggregation) interacting in various forms can commonly be observed (Barden, 1972).

Table 2.1 Nomenclature for particle arrangements in suspensions (van Olphen, 1963)

Term	Particle arrangement
Dispersed	No face-to-face association of clay particles
Aggregated	Face-to-face association of several clay particles
Flocculated	Edge-to-edge or edge-to-face association of aggregates
Deflocculated	No association between aggregates

A comprehensive study on natural recent soils was made by Collins & McGown (1974). They pointed out that a distinction should be made between studies involving laboratory prepared clays and natural deposits. In laboratory studies, usually monomineralic clays are involved. The particle arrangement is controlled by the type of mineral, concentration of the clay in the suspension and the salt content of the water. All laboratory studies have shown that there seems to be a tendency for face-to-face orientation of the clay platelets into groups, and descriptions such as "stair-stepped cardhouse" (O'Brien, 1971) or "stepped clusters" (Smalley & Cabrera, 1969) have been given to these fabric types. Aylmore & Quirk (1960) called a parallel arrangement of particles in an illitic clay a "domain" or "turbostratic group", since the elementary particles are arranged in a parallel fashion in groups, and these groups are arranged in a turbulent array. In natural soils, the most important factor which has to be taken into account compared to laboratory specimens is the interaction of the clay particle groups with silt and sand grains. In the marine, brackish and fresh-water silty clays studied by Collins & McGown (1974), sand and silt grains were evenly distributed in the microfabric, and generally did not touch each other. Similar observations have also been made by other investigators.

Collins & McGown (1974) divided microfabric features observed in natural clays into three groups, according to level of arrangement and function:

(1) Elementary particle arrangements, which consist of single forms of particle interaction at the level of individual clay, silt or sand particles.
(2) Particle assemblages, which are defined as the units of particle organization with definable physical boundaries and a specific mechanical function, and which consist of one or more forms of elementary particle arrangements or smaller particle assemblages (e.g. connectors, aggregations, matrices).
(3) Pore spaces, within and between the microfabric features described in groups (1) and (2).

Using this approach, they were able to characterize the microfabric of natural soils by the form and function of the individual microfabric features.

2.6.2 Relation between microfabric and depositional environment

Originally, it was believed that a unique microfabric would be associated with a particular type of depositional environment (e.g. fresh-water, brackish, marine). Based on their investigations, Collins & McGown (1974) concluded, however, that there is little evidence to support such a hypothesis. They found that more than one type of microfabric feature, both elementary and assemblage, could exist side by side in the fabric of a natural soil; it is possible that one feature or a set of features might be dominant. Thus, they observed that in marine soils, 'bookhouse' (edge-to-face association of clay platelet

books forming an open fabric) and 'stepped-flocculated' arrangements were more common, while the dispersed or turbostratic group arrangements occurred mainly in brackish-water soils. It appears, however, that it is not possible to predict such a predominance from the physico-chemical conditions in a depositional environment only, because of the many other factors which influence sedimentation and thus microfabric. It should also be kept in mind that the studies involved sediments which had undergone a certain amount of consolidation and were not truly fresh clays. Investigations on fresh muds might lead to somewhat different conclusions.

Pusch (1966) conducted a study of the micropores in a sensitive marine clay from Lilla Edet, Sweden. From electron microscope micrographs, he measured maximum dimensions of all pores he was able to identify, and by condensing the them by a simple statistical procedure (histograms), he compared them with micrographs from fresh or brackish-water clays. He found that the two-dimensional relative pore area in the marine clay was about three times greater, although unit weights and water contents were about the same. It appears, therefore, that particle groups and aggregates represent different densities in marine and fresh-water deposited clays. In a further development, Push (1973a) introduced the P/T value (P = total pore area in a micrograph, and T = total area of micrograph) as a microstructural parameter. Very small pores, however, cannot be taken into account by this method. A frequency plot of the P/T ratio for two different salinities showed that P/T was much higher for soft marine clays than for fresh-water clays with the same preconsolidation pressures and water contents.

A model proposed by Pusch (1973a, 1973b) shows aggregates of closely packed particles which are connected by bridges or links of fine clay plates (Fig. 2.13a). Pusch stated that, in marine clays, the aggregates are large and dense and separated by large voids, while in fresh-water the aggregates are small and relatively porous, and the voids are small (Fig. 2.14). The connectors are the weakest points in the system; Figs. 2.13b & 2.13c show the effect of a sufficiently large shear force which causes a breakdown of some of the links and bridges. This brings about an arrangement into groups of more oriented particles, which also assume a more stable position.

In soft marine and brackish-water illitic clays, Pusch (1973a) did not observe any preferred particle orientation. In soft fresh-water clays, however, larger particles, coarser than about 0.5 μm, tended to align with their long axes normal to the direction of the consolidation pressure.

The type of mineral also has an influence on the particle arrangement. Illitic clays consist often of randomly arranged particles, although 'booklike' aggregates also occur. In kaolinitic clays, booklike aggregates are the most common ones, while in smectites the fabric may be controlled by the adsorbed cations (Pusch, 1973b).

Additional effects on the fabric are caused by the presence of organic matter. Electron micrographs of ultra-thin sections have shown that organic

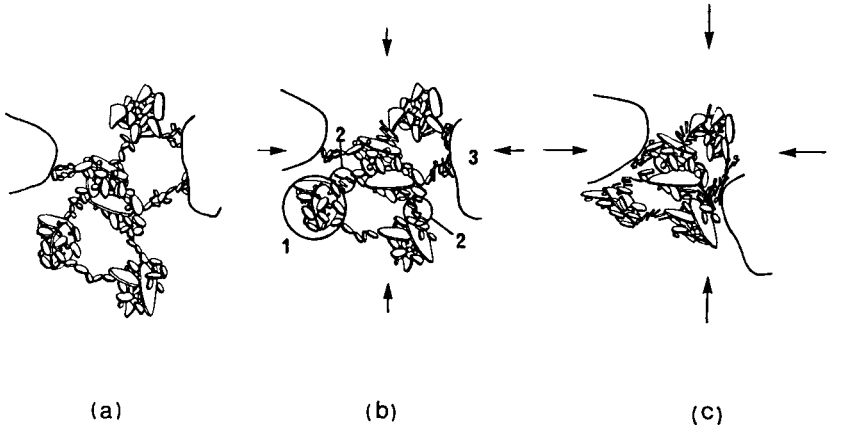

Fig. 2.13. Fabric model showing the deformation of soft clay under shear forces: (a) uncompacted soft state, (b) initial stage of deformation and distortion of connectors between aggregates due to triaxial pressure, (c) increased parallel orientation of linking particles and closer packing of aggregates caused by increased pressure (1 = aggregate, 2 = connector or link, 3 = silt grain) (Pusch, 1973b).

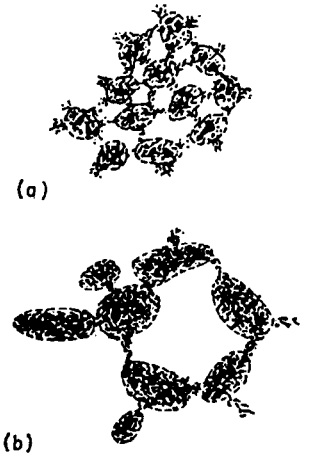

Fig. 2.14. Schematic clay particle arrangement: (a) clay deposited in fresh water, which has small, relatively porous aggregates and small voids, (b) marine clay, with large dense aggregates separated by large voids (Pusch, 1973b).

matter appears in the form of fluffy bodies, but also as distinct objects in clay particle aggregates (Pusch, 1973a).

2.7 GRAVITATIONAL COMPACTION

2.7.1 Compaction of a sediment layer

As muddy sediments accumulate on the sea-floor, the underlying layers are compacted under their own weight. This causes an expulsion of interstitial fluids and a reduction in void ratio or porosity. It should be noted here that the term *compaction* used by geologists is synonymous with the term *consolidation* used by geotechnical engineers. The term *compaction* in a geological sense refers to a decrease in bulk volume (volume of voids plus solids), whereas in soil mechanics, it is used for the process of densification of the soil by mechanical means (tamping, vibrating, etc) in order to expel the air, at more or less unchanged water content. The term *consolidation* in soil mechanics describes a process of reduction in bulk volume by the expulsion of pore fluid under the application of a load. Consolidation in a geological sense refers to the acquisition of structural competency by a sediment by reduction in volume and/or by induration, cementation, etc (Allen & Chilingarian, 1975). Skempton (1970) defined *consolidation* as:

> "the result of all processes causing the progressive transformation of an argillaceous sediment from a soft mud to a clay and finally to a mudstone or shale. These include interparticle bonding, desiccation, cementation and, above all, the squeezing out of pore water under the increasing weight of overburden."

Skempton called the latter process *gravitational compaction*. In the rest of this chapter, the term *consolidation* will be used in the soil mechanics sense.

The state of stress under which a sediment consolidates depends on the overburden, but during the initial stages of sedimentation, it is probably hydrostatic. As sedimentation progresses, the loading becomes triaxial. Finally, as the overburden load becomes large, a uniaxial condition is approached (Wolf et al, 1976).

Consolidation of a clay layer is governed by Terzaghi's principle of effective stress. In the deposit in Fig. 2.15, the *effective stress* (overburden pressure), σ'_{vo}, at point A is given by:

$$\sigma'_{vo} = \sigma_{vo} - u \tag{2.1}$$

where σ_{vo} is the total overburden pressure at depth z ($\sigma_{vo} = \gamma z$, where γ = average bulk unit weight of the material above point A) and u is the pore pressure ($u = \gamma_w h$). Since it is the effective pressure which controls consolidation, submergence of a fully saturated layer under a great depth of water (Fig. 2.16) does not have any influence on the consolidation of the clay, provided the clay is highly porous, which should be the case near the water/

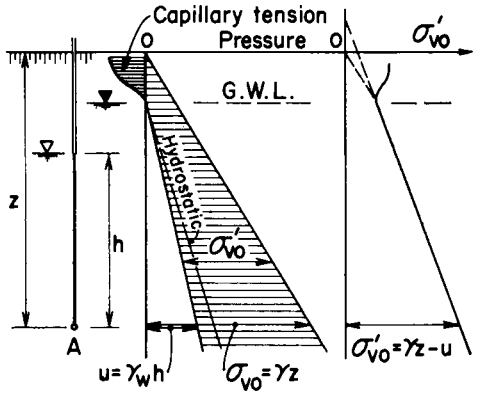

Fig. 2.15. Typical distribution of total overburden pressure, σ_{vo}, pore pressure, u, and effective overburden pressure, σ'_{vo}, in a soft clay deposit (Skempton, 1970).

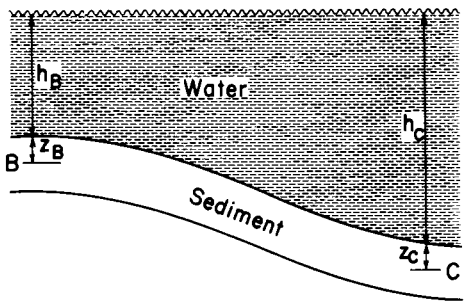

Fig. 2.16. Effect of submergence on effective stress in a sediment layer. If $z_B = z_C$, the effective stresses at points B and C are equal (Wolf et al, 1976).

sediment interface. If the depths z_B and z_C are equal, then the effective overburden pressures at points B and C are the same.

When the porosity is low, Eq. 2.1 still applies to clays, and even to mudstones with a porosity as low as 15% (Skempton, 1970). Skempton cited an equation proposed by Professor A.W. Bishop of the form:

$$\sigma'_{vo} = \sigma_{vo} \left(1 - \frac{C_s}{C}\right) u \tag{2.2}$$

where C_s is the compressibility of the particles, and C is the compressibility of the clay structure. In practice, however, the ratio C_s/C is less than 0.01 to depths of at least 3,000 m (for the cases investigated by Skempton), so that Eq. 2.1 is sufficiently accurate for general application.

In a clay sediment which has undergone a rapid increase in thickness due to a high rate of sedimentation, *excess* pore pressures can develop. These excess pore pressures at any level in the stratum reduce the effective stress under which the material would normally consolidate and, as a result, the shear strength also decreases. A clay in such a state is called *partially consolidated* or *underconsolidated,* which means that part of the overburden stress still has to be transferred to the soil skeleton. Such conditions have been reported by McClelland (1967) for clays in the Mississippi delta. The underconsolidated state could be verified by piezometers. Underconsolidation has been identified by Morgenstern (1967) as a possible factor causing submarine slumps.

Analytical and numerical solutions for the degree of consolidation of a layer whose thickness increases with time have been presented by Gran Olsson (1953) and Gibson (1958). Gran Olsson presented an approximate solution for the degree of consolidation for the case of a constant rate of deposition, while Gibson showed that a closed-form solution of the problem can be found for a rate of sedimentation proportional to $t^{\frac{1}{2}}$ (t=time). For the more general case of an arbitrary sedimentation rate, Gibson was able to reduce the problem to a linear integral equation, a solution to which could be obtained for a constant rate of sedimentation, i.e. where the thickness of the deposit is proportional to time. If we assume 95% consolidation as sufficiently close to the complete dissipation of the pore pressure (which theoretically is only reached after an infinitely long time), then Gibson's work shows that the ratio $c_v/(rh)$ must be more than 10, where h is the thickness of the layer, r the rate of deposition ($h = rt$) and c_v the coefficient of consolidation of the clay. On this basis, Skempton (1970) calculated that, for a typical rate of deposition in an estuarine Holocene clay of 2m/1000 years, a clay layer will be more than 95% consolidated if the thickness is less than 50 m (assuming a c_v value of 1 m^2/yr = 3.2×10^{-4} cm^2/sec). This condition is satisfied for most soft clay deposits. In modern deltas, however, such as the Mississippi delta, the rate of deposition under certain conditions can be much higher, e.g. 100 m/1000 years, which is the reason for the underconsolidated deposits reported by McClelland (1967), which were sedimented in an interdistributary environment (Section 2.4.3).

Based on the work of Gibson (1958), Morgenstern (1967) calculated the degree of consolidation of a clay layer on an impermeable base for a range of rates of sedimentation and c_v values. This is illustrated in Fig. 2.17. A thickness of 15 m is assumed, and the coefficient of consolidation varies from 1×10^{-5} cm^2/sec (representing a clay) to 1×10^{-2} cm^2/sec (representing a coarse silt). The range of rates of sedimentation considered represent abyssal conditions to extreme deltaic conditions. The figure reveals that under-consolidation can occur only for silty clays and clays deposited at deltaic rates.

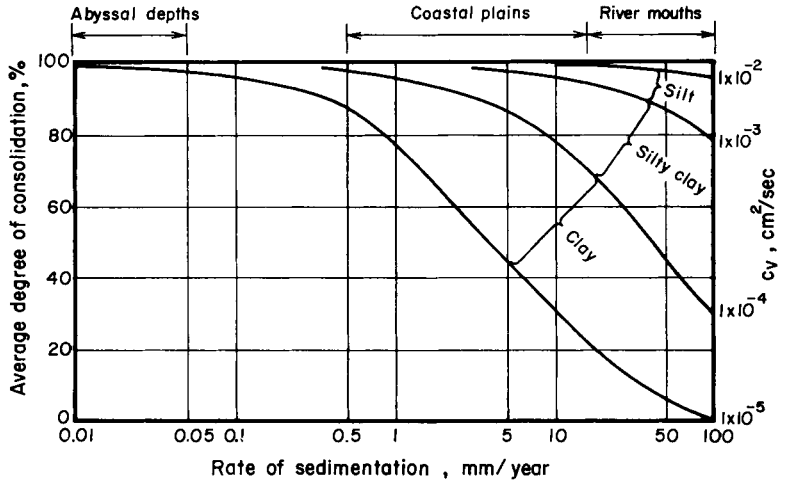

Fig. 2.17. Relatiónship between rate of sedimentation and degree of consolidation for a 15 m thick stratum (Morgenstern, 1967).

2.7.2 *Sedimentation compression curves*

The relationship between void ratio and effective pressure was named by Terzaghi (1941) the *sedimentation compression curve*. Skempton (1970) presented such curves for a large variety of clayey deposits from very recent to Pliocene age. Figure 2.18 shows a sedimentation compression curve to an arithmetic scale. In order to establish such a curve from a clay deposit, it is necessary to obtain samples from various depths, and to plot their void ratios, *e*, versus the effective overburden pressure. It has to be assumed that the samples are fully saturated. The pore pressure at the sampling depth must

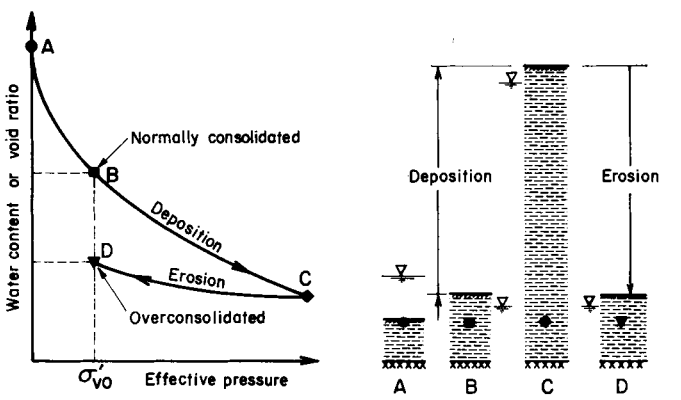

Fig. 2.18. Diagram illustrating normally consolidated and overconsolidated clay (Skempton, 1970).

also be known in order to calculate the effective stress. It is important that the clay samples are normally consolidated, so that the points plot on the line *ABC* in Fig. 2.18.

Terzaghi defined a clay as *normally consolidated* when it had never experienced a pressure higher than the existing in-situ effective stress. In case of an *overconsolidated* clay, the maximum past pressure is larger than the present one, and such a deposit would plot on the curve *CD* (Fig. 2.18).

There are various ways to check whether a natural clay deposit is normally consolidated. Skempton (1970) proposed the following two methods:

(1) From oedometer tests and using Casagrande's graphical procedure to find the preconsolidation pressure, σ'_{vc}, (Casagrande, 1936); the lower limit, $\sigma'_{vc(min)}$, can be obtained by extending the linear virgin portion of the e—log σ'_v curve to intersect with the horizontal line passing through the in-situ void ratio, e_0 (Fig. 2.19). If the clay is normally consolidated, σ'_{vc} and $\sigma'_{vc(min)}$ will straddle the value of σ'_{vo}.

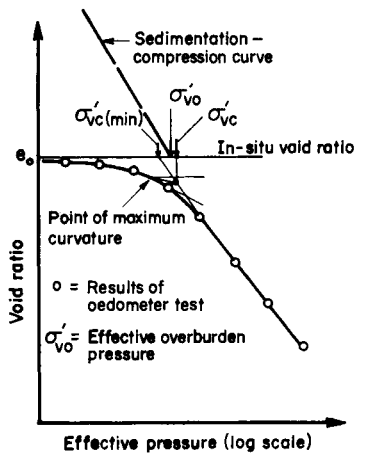

Fig. 2.19. Typical result of oedometer test on normally consolidated clay (Skempton, 1970).

(2) From a plot of the ratio s_u/σ'_{vo} versus depth, where s_u is the undrained shear strength, the clay is normally consolidated if the points lie approximately on a straight-line, i.e. if the undrained shear strength increases in proportion to the effective overburden pressure. Often, there is a small surface intercept of the order of 1 to 5 kPa.

The normally consolidated clay deposits investigated by Skempton (1970) had a wide range of plasticities and water contents, but Skempton showed that their liquidity index, I_L, varied within a narrow range. The liquidity index is defined as:

$$I_L = \frac{w_n - w_p}{w_L - w_p} \tag{2.3}$$

where w_n, w_p and w_L are the natural water content, plastic limit and liquid limit respectively. By means of the liquidity index, the natural water content of a clay can be compared directly with its Atterberg limits, which in turn are a function of the mineralogy of the clay. For normally consolidated non-sensitive clays, the liquidity index decreases with depth.

The argillaceous deposits for which Skempton (1970) established sedimentation compression curves can be grouped into three age ranges, namely: (1) recently deposited muds on the sea-bed and in tidal flats, (2) late Quaternary clays deposited during the eustatic sea-level rise in estuaries and deltas, which usually occur at depths shallower than 30 m, and (3) hard clays and mudstones of Pliocene and Pleistocene age which extend to depths of up to 3,000 m.

Figures 2.20 and 2.21 show geotechnical properties from off-shore cores taken on the sea-bed near Gibraltar, and from cores obtained in an intertidal flat near Avonmouth, Gloucestershire, England, just above low-water neap tides. In the example from Gibraltar (Fig. 2.20), it can be seen that the clay (mud) is normally consolidated. The natural water content and liquidity index decrease with depth, and the shear strength follows a linear relationship with respect to overburden pressure. In the tidal flat near Avonmouth (Fig. 2.21), oedometer tests indicated a preconsolidation pressure of about 10 kPa, a value which was even greater than the total overburden pressure. Skempton stated that such high preconsolidation pressures must be caused by desiccation during exceptionally low tides when the flats become dry; in a discussion to the paper, however, the possibility of other factors, such as cyclic fluctuations of pore pressure and the destruction of organic matter in salt marsh areas when buried by further sedimentation, were suggested as possible causes.

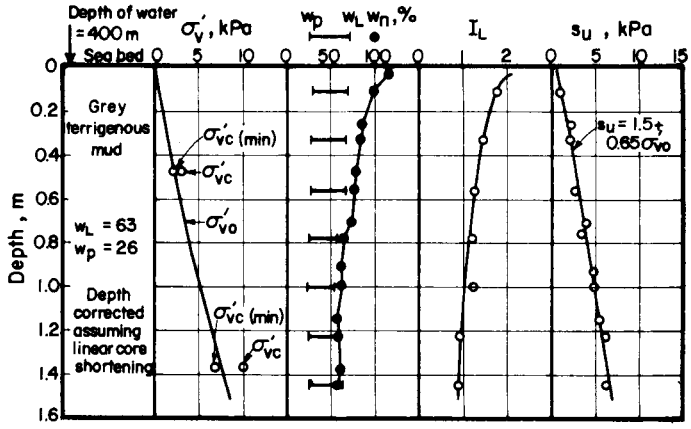

Fig. 2.20. Geotechnical properties of a core from the western Mediterranean near Gibraltar (Skempton, 1970).

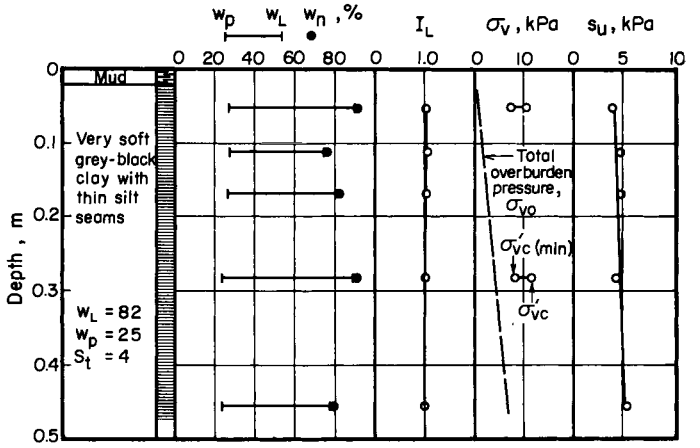

Fig. 2.21. Geotechnical properties of a core from intertidal flats at Avonmouth, England (Skempton, 1970).

From such geotechnical profiles as Figs. 2.20 and 2.21, Skempton plotted the data points onto a graph of void ratio versus effective overburden pressure (Fig. 2.22) by adopting the following procedure: (1) the I_L value for each sample was calculated from Eq. 2.3, (2) the average measured values of w_L and w_p for each stratum were calculated, and (3) a 'corrected' water content,

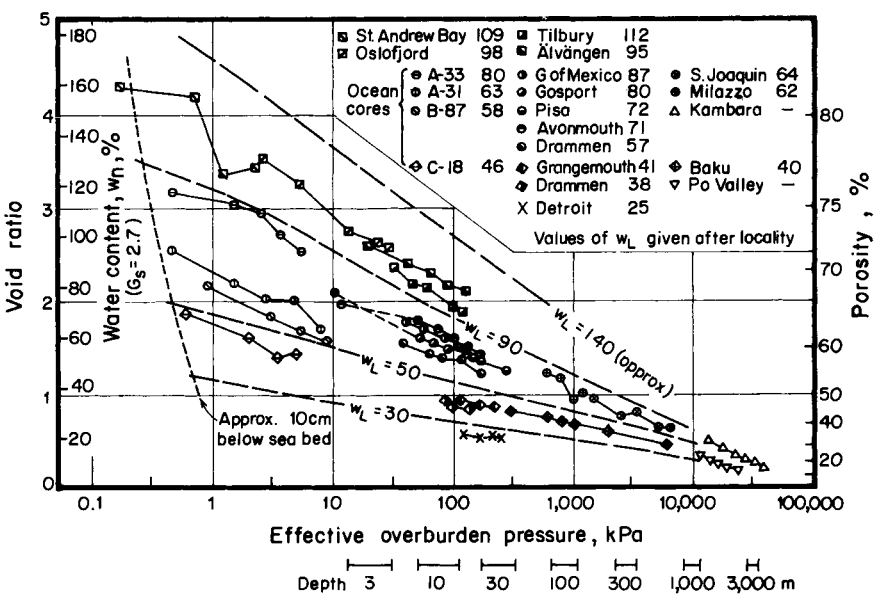

Fig. 2.22. Sedimentation compression curves for normally consolidated argillaceous sediments (Skempton, 1970).

w_n, was evaluated for each sample by using the actual I_L of that sample and the average w_L and w_p of the stratum. The void ratios were obtained from the corrected water contents.

In Fig. 2.23, the mean water contents in the top 250 mm of clay from six sea-bed locations and from three tidal flat locations are plotted against the liquid limits. It is interesting to see that, for each depositional environment, the points fall more or less on a straight-line representing a constant liquidity index. It is therefore possible to estimate from such a graph the mean water content of the top 250 mm of a fresh sediment by knowing the Atterberg limits. It will also be noticed that, in tidal flats, the water content is about equal to the liquid limit, while it is higher for clay deposits on the ocean floor.

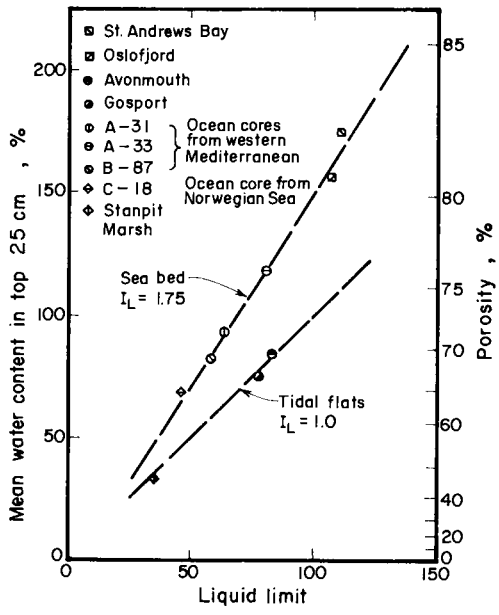

Fig. 2.23. Depositional water contents of clays below the sea-bed and intertidal flats (Skempton, 1970).

Figures 2.24 and 2.25 show deposits of late Quaternary (Holocene) clays from the coastal plain near Avonmouth and from a marsh east of Tilbury Docks, both in England. In the Avonmouth profile, the top 4 m shows an overconsolidated condition, probably as a result of intermittent drying during formation. In this zone, the liquidity index increases with depth. The pore pressure at about 8.5 m shows a deficiency in piezometric level of approximately 2 m, but the variation with depth is hydrostatic to a depth of about 5 m. Void ratios derived from the liquidity index are plotted against σ'_{vo} in Fig. 2.22, and the line connecting the corresponding points is extrapolated

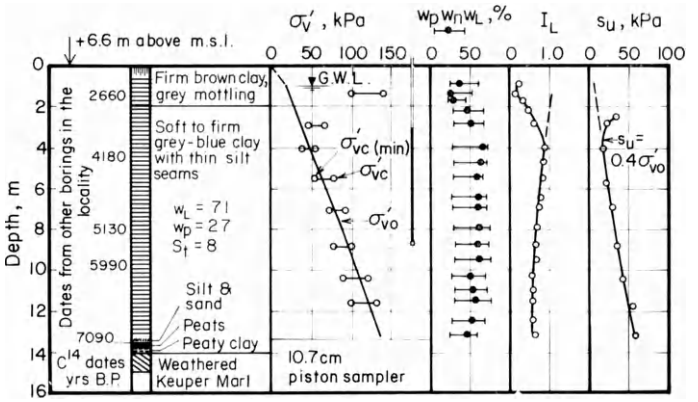

Fig. 2.24. Geotechnical properties of an estuarine clay from Avonmouth, England (Skempton, 1970).

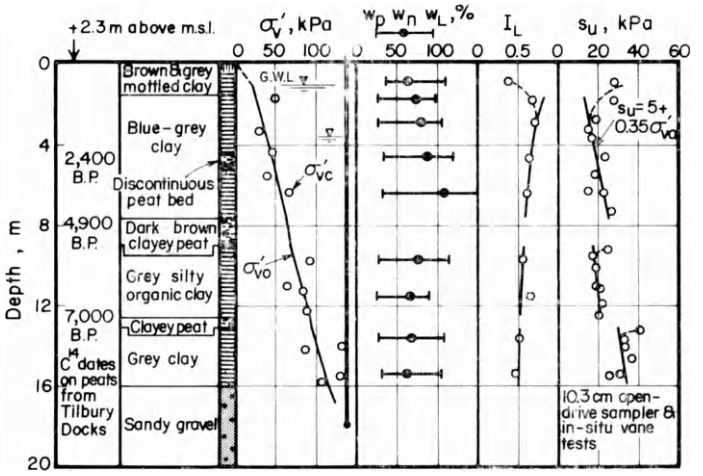

Fig. 2.25. Geotechnical properties of a clay from a marsh area east of Tilbury Docks, London (Skempton, 1970).

back to a point which represents the void ratio and preconsolidation pressure in the clay from the Avonmouth tidal flat (Fig. 2.21).

The boring near Tilbury Docks (Fig. 2.25) was made in a reclaimed tidal marsh. There are three clay beds separated by peat layers in this area. Below about a 3 m depth, the clays are normally consolidated, but there is a thin zone of increased strength just below each peat layer. This could be the result of some desiccation, or it could be due to some interaction between the clay and organic substances from the peat. The pore pressure in the gravel shows changes due to tidal fluctuations.

In Fig. 2.22, sedimentation compression curves for normally consolidated deposits are given for 21 sites. Data from very old deposits (Pliocene and Pleistocene) fall into the lower right-hand portion of the figure, while recent sea-bottom muds fall into the left-hand side. Full saturation was assumed for all calculations of void ratio and water content. An approximate conversion from pressure to depth is shown along the bottom of the graph. Also shown in the figure are four approximate curves for the liquid limits of 30, 50, 90 and 140%. Skempton used these values to group argillaceous deposits into clays of low, medium and high plasticity. Finally, Fig. 2.26 shows the points of Fig. 2.22 replotted in terms of liquidity index instead of void ratio, and it can be seen that all points fall into a relatively narrow band.

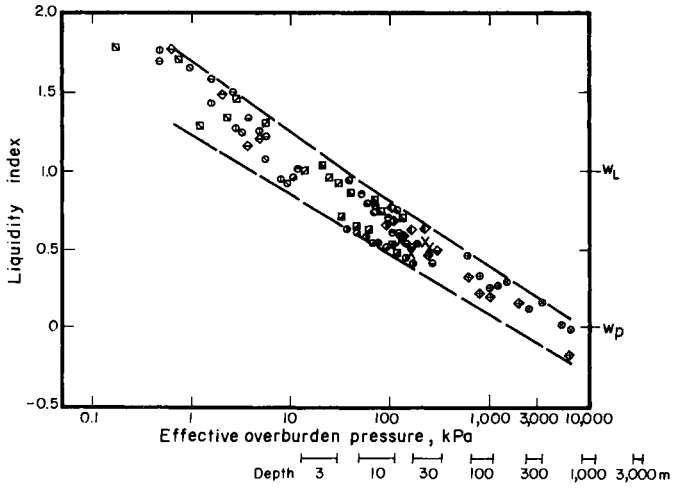

Fig. 2.26. Relationship between liquidity index and effective overburden pressure for the clays designated in Fig. 2.22 (Skempton, 1970).

2.7.3 Other influences on the compaction of argillaceous sediments

A comprehensive treatment of the compaction of argillaceous sediments has been presented by Rieke & Chilingarian (1974). In addition to overburden, there are several other factors which influence the shape and relative position of the sedimentary compression curve, such as particle size, clay mineral type, adsorbed cation, salt concentration in the pore water, acidity, and the presence or absence of microfossils. For soft clays, which usually experience pressures of less than about 100 to 150 kPa, these factors mainly influence the initial void ratio and water content. This has already been illustrated in Fig. 2.23 (Skempton, 1970), where the water content in the top layer of a subaqueous deposit was compared with the liquid limit, which in turn depends on the mineralogy and pore water chemistry.

Many researchers have found that the salinity of the pore water squeezed out during compaction progressively decreases with increasing overburden pressure (see for example von Engelhardt & Gaida, 1963). A detailed treatment of this observation and its implications is given by Rieke & Chilingarian (1974, 1978).

In Fig. 2.27, it is again demonstrated that the initial water content (or porosity) is influenced by the depositional environment (fresh-water, brackish, marine). This figure shows that initial water contents of recent sea or lake bottom sediments are between 50 and 80% (or 100 and 400% when expressed with respect to dry weight), corresponding to porosities of 70 to 90%.

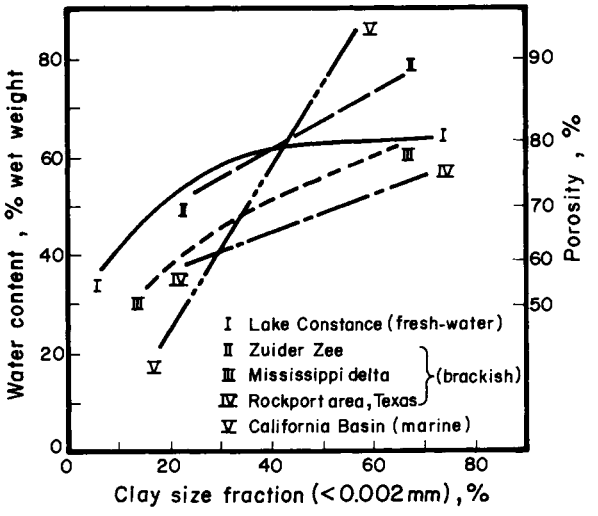

Fig. 2.27. Correlation between water content and clay fraction (< 2 μm) in different depositional environments (Müller, 1967).

2.7.4 *Particle orientation produced by increasing pressure*

Particles of clay minerals seem to develop preferred orientations at very early stages of compaction at pressures near 100 kPa, a process which is enhanced by greater initial water contents and greater amounts of carbonaceous organic matter. In the case of kaolinite sedimented in flocculating and deflocculating pore fluids, Morgenstern & Tchalenko (1967) found that stresses as low as 10 kPa were sufficient to produce orientation. Bowles et al (1969) investigated soft clay samples from cores taken in the Gulf of Mexico; after loading specimens in an oedometer to various pressures between 49 and 6278 kPa, they observed from electron micrographs that, at a pressure of 49 kPa, the microstructure still retained an open random arrangement of particles. This pressure was well above the maximum past pressure of these specimens. Only samples subjected to maximum normal pressures of 3139 or

6278 kPa were characterized by a predominantly high degree of parallelism between the particles. Normally consolidated deposits could therefore be expected to show lower degrees of orientation than overconsolidated clays, but the degree of preferred orientation also depends on the type and concentration of electrolytes in the pore water.

Meade (1966) and Gipson (1966) found no systematic correlation between depth of burial and degree of preferred orientation. Gillot (1970, 1979) investigated the fabric of Champlain Sea sediments in Canada, and found that the fabric is commonly random, but some clays which were overconsolidated showed imperfect orientation. Sensitive clays from Alaska and Norway exhibit a random fabric (Gillot, 1979). Barden (1972) found from electron micrographs that Boston Blue clay, which has been deposited in brackish-water, shows a high degree of plate parallelism. This clay, however, is somewhat overconsolidated, but Barden believed that the orientation was mainly caused by the electrolyte conditions at the time of deposition.

The degree of particle or aggregate orientation (and consequent fabric anisotropy) in a soft clay deposit, therefore, is not a function of the overburden only. The conditions at the time of deposition, such as electrolyte content of the depositional medium and the rate of sedimentation (which control the initial fabric), are probably more important than the increasing vertical stress due to overburden.

2.8 AGING AND DELAYED COMPRESSION OF SEDIMENTS

In a clay layer consolidating under a constant pressure (such as its own weight), the volume decrease does not cease when the excess pore water pressures have dissipated, or, in other words, when the *primary consolidation* phase has ended. Continuous adjustment of the soil fabric under more or less constant effective stress takes place, which results in a more stable structural arrangement (i.e. the free energy of the system becomes a minimum), and this leads to a further decrease in volume. Bjerrum (1967) termed the reduction in volume under unchanged effective stress, *delayed compression*. The term *delayed compression* is often taken as being synonymous with *secondary compression,* although secondary compression was originally defined as the compression that takes place after the excess pore pressures have dissipated, i.e. at the end of primary consolidation. In the following, the two terms will be used interchangeably. A more precise definition of terms has recently been proposed by Leonards (1977).

Figure 2.28 shows a plot of void ratio versus log consolidation pressure published by Bjerrum (1967, 1972, 1973) to illustrate the effect of time (geological age) on the compressibility characteristics of a normally consolidated soft clay. Bjerrum defined a clay which has been deposited recently and is at equilibrium under its own weight without having undergone significant secondary or delayed compression as a *young normally consolidated clay*.

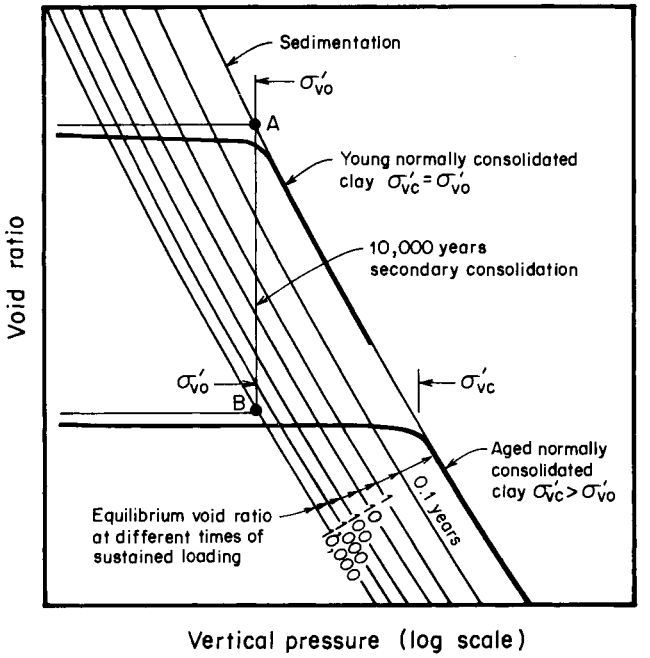

Fig. 2.28. Effect of geological history on the compressibility of normally consolidated clays (Bjerrum, 1972).

Such a clay is just able to carry its own weight, and any additional load, such as a fill or a building, would cause large settlements. If, on the other hand, the clay is left under a constant effective stress for hundreds or thousands of years, delayed compression causes a decrease in void ratio, as indicated by the vertical path *AB* in Fig. 2.28. Because of its enhanced resistance against further deformation, the clay is able to carry a load in addition to the effective overburden pressure, and the compression behaviour of an undisturbed sample in an oedometer test would give a curve similar to that indicated by the *aged normally consolidated clay* in Fig. 2.28. The pressure σ'_{vc} (denoted by Bjerrum as p_c) is greater than the effective overburden pressure, σ'_{vo}, and, for a given time of loading, σ'_{vc} increases linearly with σ'_{vo}; in other words, the ratio $\sigma'_{vc}/\sigma'_{vo}$ is constant throughout a homogeneous clay deposit (Bjerrum, 1967, 1972, 1973). The pressure σ'_{vc} (or p_c) is often called *critical pressure* or *apparent preconsolidation pressure.*

Since the $\sigma'_{vc}/\sigma'_{vo}$ ratio depends on the amount of secondary compression the clay has undergone under the existing overburden pressure, and the amount of secondary compression increases with plasticity, the value of $\sigma'_{vc}/\sigma'_{vo}$ increases with the plasticity index of the clay. This is illustrated in Fig. 2.29. The increased resistance to compression due to aging is also reflected in a gain in strength; Fig. 2.30 demonstrates how the ratio s_u/σ'_{vo} is influenced by

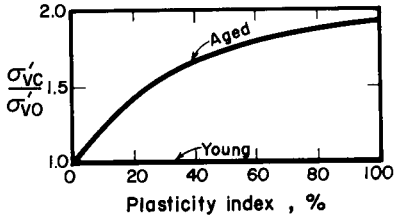

Fig. 2.29. Typical values of preconsolidation ratio, $\sigma'_{vc}/\sigma'_{vo}$, in normally consolidated late glacial and postglacial clays (Bjerrum, 1972).

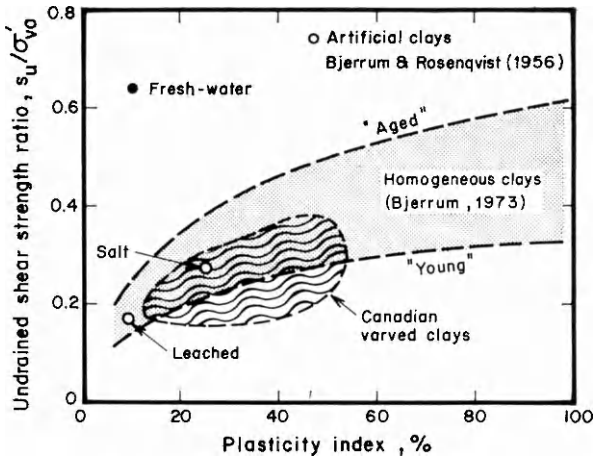

Fig. 2.30. Variation of strength ratio, s_u/σ'_{vo}, with plasticity for uncemented, normally consolidated glacial-lake and marine clays (Kenney, 1976).

the age of the clay (s_u from vane tests). Since late Quaternary soft clays are usually between about 3,000 and 10,000 years old, they have undergone some secondary compression, and they exhibit a preconsolidation pressure below the weathering crust (i.e. $\sigma'_{vc} > \sigma'_{vo}$).

In practice, it is usually difficult to construct the diagram for a clay deposit of the kind shown in Fig. 2.28. An estimate, however, can be made as follows. From a laboratory oedometer test, the one-day void ratio—log pressure curve is determined. A parallel second curve is then drawn through the point (e_o, σ'_{vo}), which is a curve corresponding to the loading age of the deposit. If the deposit has, for example, a loading age of 10,000 years, then the space between the one-day curve and the 10,000 year curve represents approximately six log cycles of time. About 50% of the delayed compression will then take place during the first year, and about 80% within 100 years (Bjerrum, 1973).

Laboratory experiments which demonstrate the validity of Bjerrum's concept (Fig. 2.28) have been conducted by Berre & Iversen (1972) and Rizkallah et al (1977).

2.9 PHYSICAL AND CHEMICAL ALTERATIONS OF SOFT CLAY SEDIMENTS

2.9.1 General

In this section, post-depositional processes of both a physical and a chemical nature will be described, all of which can have a profound effect on the geotechnical properties of soft clays. These processes are desiccation, joint formation, chemical weathering, leaching and cementation. They are often inter-related; for example, the generation of fissures by desiccation (or also by frost action) at the surface of a clay layer facilitates the entry of water to bring about chemical weathering processes.

2.9.2 Desiccation and crust formation

When the surface of a sedimentary deposit is exposed to the air, the zone located immediately below the surface starts to dry out. The loss in water by evaporation causes water to flow from the deeper regions of the soil profile towards the surface by capillary action. This capillary rise is associated with a decrease in pore water pressure to a negative value, which in turn causes an increase in effective stress (Fig. 2.15). The tension in the pore water has the same mechanical effect as a surcharge. If a clay sediment is very compressible, desiccation by surface evaporation can produce a significant decrease in void ratio, and the layer assumes the character of a *drying crust* (Terzaghi, 1955). In cold climates, the top of this drying crust can be heavily fissured by frost action, and the water content can reach values as low as the plastic limit (see Table 2.6).

Below the drying crust, processes of chemical weathering (Section 2.9.4) in association with leaching (Section 2.9.5) produce a zone of weathering, the lower end of which is usually taken as the depth at which the undrained shear strength attains its minimum. In engineering terms, the drying crust together with the weathering zone is often designated as the *weathering crust* or simply as the *crust*.

While the thickness of the drying crust depends mainly on climatic conditions (i.e. humid or dry climate) and on the position of the ground water-table and its seasonal fluctuations, the depth of the weathering zone is governed by the mass permeability of the clay. Thus, in Scandinavia, the low-lying, poorly-drained plastic clays of Sweden have a crust thickness of 1 to 3 m, while in the low-plasticity well-drained marine clays of Norway, the thickness is from 6 to 8 m (Bjerrum, 1973) (Fig. 2.35). A similar range (6 to 8 m) is found in the relatively well-drained marine clays of the St Lawrence Valley in Canada, but considerable variations can be expected here because of the rather complex geological history of the area, details of which have been given by Karrow (1961), Gadd (1975) and others.

Figure 2.31 shows a sensitive cemented Champlain Sea deposit from St Alban, about 80 km west of Quebec City, a site which was described by La Rochelle et al (1974). In this profile, the depth of weathering only extends to

about 2 m. British estuarine clays (see Fig. 2.24 and Skempton & Henkel, 1953) show a crust thickness of about 4 m. In the marine clays of Southeast Asia, such as Bangkok clay (Fig. 2.32), which are exposed to a tropical climate, the crust rarely exceeds a depth of 4 to 5 m (Moh et al, 1969; Cox, 1970b). The duration of exposure to subaerial conditions is probably as important as climate. Thus, fluctuations in sea-levels (or lake levels) associated with advances and retreats of ice fronts have generated in some marine-glacial and

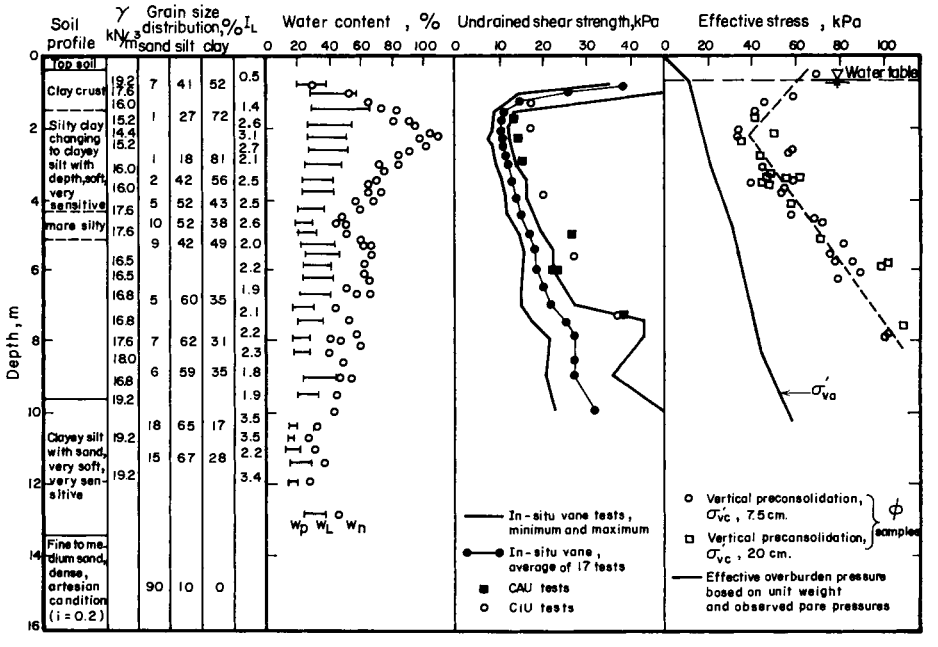

Fig. 2.31. Geotechnical profile of a cemented sensitive Champlain Sea clay at St Alban, Quebec (Leroueil et al, 1978).

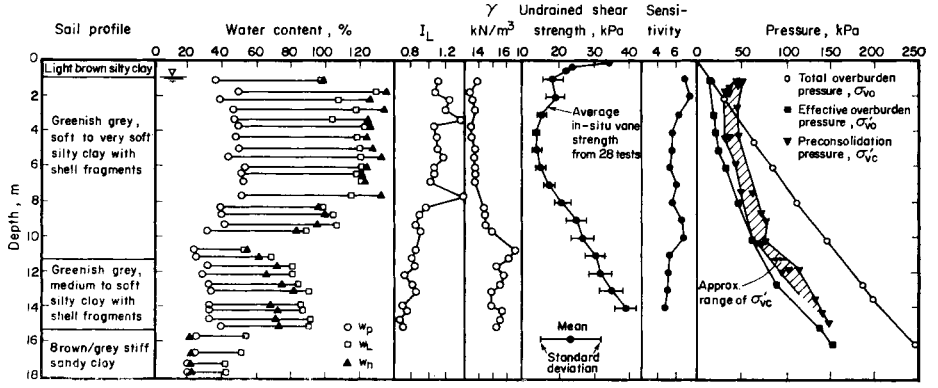

Fig. 2.32. Geotechnical profile of soft Bangkok Clay at Bangpli.

postglacial clays several drying crusts below the surface. An example is the Lake Agassiz deposits reported by Rominger & Rutledge (1952).

2.9.3 Jointing

Some normally consolidated clays, particularly all flood-plain clays and many precompressed clays, are weakened by a network of cracks or joints. The jointing in flood-plain clays can be accounted for by seasonal variations in water content which cause alternating expansion and contraction (Terzaghi, 1955). Two mechanisms of crack or fissure formation should be distinguished. If the formation is due to volume reduction or shrinkage because of desiccation, the cracks are termed *mud cracks*. They usually form a pattern of orthogonal polygons (hexagonal polygons are an exception), and the crack spacing as a rule-of-thumb is about ten times the crack depth. Subaerial drying is the most common cause of mud crack polygons, but their initial formation under water has also been observed (Neal, 1978).

The other mechanism of crack or joint formation is of colloid chemical origin, i.e. the dewatering and shrinkage of a gel during aging. This process is known as *syneresis*, and it occurs in natural submerged clay sediments. Skempton & Northey (1952) found that a clay which they remoulded at its liquid limit and stored under water for one year developed fissures. The water content of the clay close to a fissure was slightly lower than that of the bulk. They attributed this observation to syneresis. Eide (1968) observed that the soft Bangkok clay was transected by a system of fissures. The surfaces of the fissure plains were irregular and smooth slickensided, and the fissuring appeared to be more pronounced in the deeper samples. Eide suggested that these fissures were the result of syneresis.

Terzaghi (1955) stated that the walls of the joints of flood-plain deposits may be "dull or slickensided, fairly even or intricately warped". He also pointed out that, in normally consolidated deposits, the occurrence of joints has no particular engineering significance. Joints, however, might be troublesome in laboratory testing of a clay specimen containing such discontinuities, as they can often only be detected after the specimen has been sheared.

2.9.4 Chemical weathering

Weathering affects all soil and rock deposits and has a profound impact on their geotechnical properties. A broad definition of *weathering* which is suitable for geotechnical purposes was given by Kenney (1975) as:

> "Weathering is those processes which cause structural disintegration and decomposition of geological materials under the direct influence of the hydrosphere and atmosphere. Disintegration is physical breakdown of the structure of a material, and decomposition is chemical alteration of the constituent minerals and matrix materials."

Emphasis is given in this section to chemical processes, since these influence fabric stability.

Chemical weathering can take place both subaqueously and subaerially; however, as soon as a clay deposit rises above the water-level, weathering processes are intensified. Weathering from the top of a profile can be caused by percolating rainwater which contains dissolved oxygen. The depth of weathering is thus determined by the depth of percolation and the oxygen content of the pore water. Weathering can also proceed from below by upward flowing of artesian ground-water in combination with leaching. There are basically two processes which determine the result of weathering, namely: (1) decomposition of minerals, and (2) ion (or base) exchange. Environmental variables which control the direction and rate of these processes are acidity (pH value), redox potential (Eh) and temperature. The most detailed studies on the weathering of soft clays have been carried out with Norwegian clays; in these marine clays, the most common clay minerals are illite (hydrous mica) and chlorite (Bjerrum, 1954).

The initial ionic composition of the pore water of a marine clay consists predominantly of Na^+, K^+, Mg^{2+} and Ca^{2+} ions, with Na^+ dominating. The relative concentration of ions may also be maintained after some of the salt has been removed by leaching. If percolating rainwater containing CO_2 enters the clay, the pH value is reduced. This reduction can be enhanced by the formation of humic acids from decomposing organic matter. At low pH values, the minerals in the clay are attacked and then start to disintegrate, which causes the release of ions into the pore water. Thus, Mg^{2+} can be released from chlorite (which is the least stable mineral in these clays), K^+ from disintegrating feldspar and mica (including illite), Ca^{2+} from carbonates (e.g. microfossils), Al^{3+} from chlorite and feldspar, and Fe^{2+} and Fe^{3+} from chlorite (Bjerrum, 1967).

Once the ions are released into the pore water, they become available as exchange ions in the diffuse double-layers of the clay particles. Thus, sodium, which has a low replacing power, can be exchanged for potassium if the potassium ion concentration is increased due to the decomposition of feldspar and illite. Similarly, polyvalent ions (e.g. Ca^{2+}, Mg^{2+}, Al^{3+}, Fe^{2+}, Fe^{3+}) replace monovalent ions, and decrease the electro-negativity and hence the repulsive forces. It is believed that, in Norwegian marine clays, the K^+ ions are the dominant ones during the early phases of weathering, and their ability to develop strong bonds provides additional strength to the fabric of a young clay (Bjerrum, 1967).

Ions of iron and aluminium released by the decomposition of minerals can precipitate as hydroxides, which can act as cementing agents. At pH values between about 5 and 9, these hydroxides are practically insoluble. It is possible that such hydroxides have positive charges (e.g. exist in the form of $Fe(OH)_2^+$) and could act as bridges between clay particles (in combination with organic matter; see Section 2.3.5). Divalent iron in the weathered crust will be oxidized to trivalent iron. This is illustrated in Fig. 2.33 in a clay profile from Manglerud, Oslo. It can be seen that, in the top portion (weathered crust) of

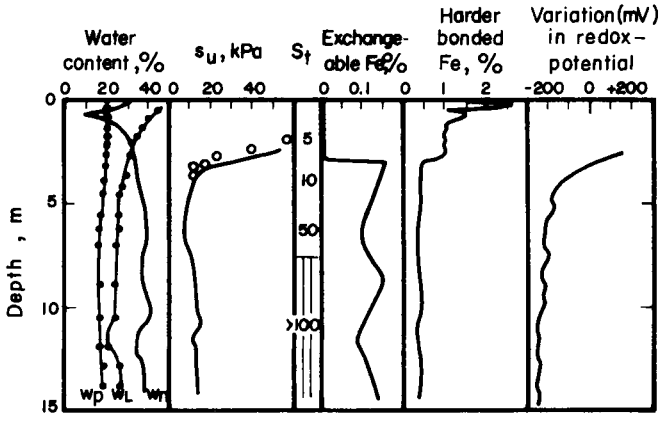

Fig. 2.33. Quick clay profile from Manglerud, Oslo, illustrating the oxidation of divalent to trivalent iron in the weathering crust (Moum, 1967).

the profile, the amount of exchangeable iron, which is in divalent form, has been greatly reduced compared with the lower part of the profile. The oxidizing environment, indicated by the redox potential, has oxidized Fe^{2+} to Fe^{3+}, which in the form of hydroxide is 'harder' bonded (Bjerrum, 1969a).

A situation in which weathering occurs at the base of a clay deposit by upward flowing artesian ground-water is illustrated in Fig. 2.34, which shows a cross-section through a clay deposit at Drammen, Norway. The geotechnical properties and the results of a detailed geochemical investigation carried out by Moum et al (1971) are given for boring No. II in Fig. 2.35. The concentration of Na is seen to increase towards the surface in accordance with the direction of flow; however, the contents of Mg and Ca at the bottom are distinctly higher than in the centre portion of the profile, marked as *quick clay*. This is the result of weathering by the flowing water, which may contain small amounts of dissolved carbon dioxide capable of attacking carbonates and releasing Ca, or which may contain oxidizing agents which could decompose chlorite, releasing Mg. Bjerrum (1969a) called this mechanism of weathering

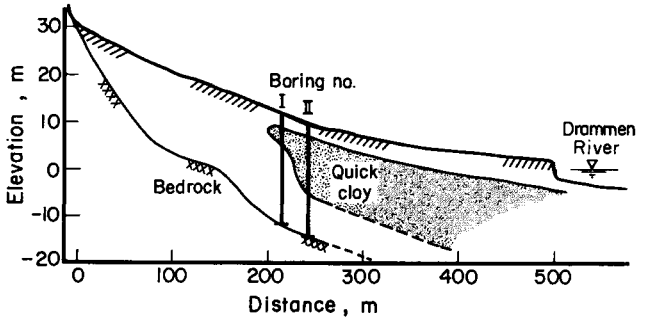

Fig. 2.34. Geological conditions at Drammen, Norway (Moum et al, 1971).

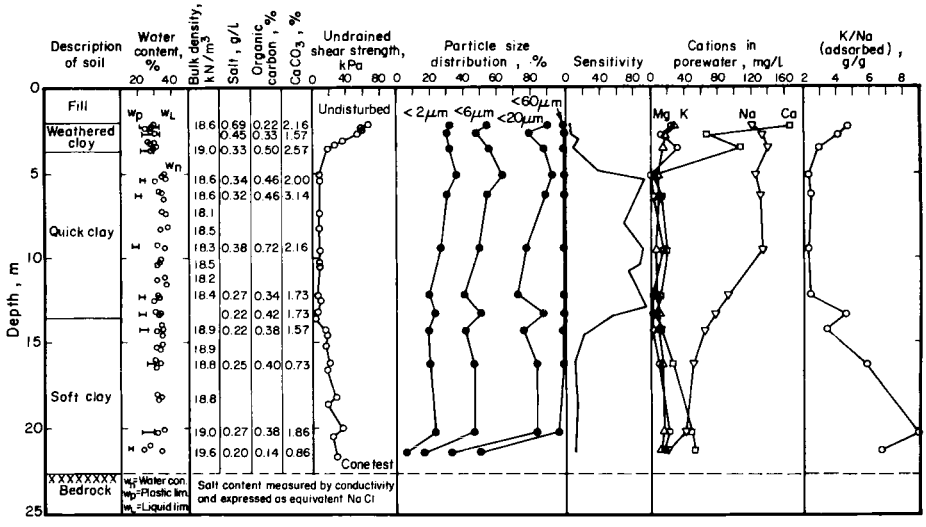

Fig. 2.35. Geotechnical and geochemical data of boring II (Fig. 2.34), showing the relationship between sensitivity and pore water chemistry (Moum et al, 1971).

mild weathering. It will be seen later (Section 2.10.3) that this process is beneficial to the clay in that it 'desensitizes' it.

Chemical weathering processes affect the plasticity, strength and compressibility characteristics of a soft clay. Of particular interest is the effect on the remoulded shear strength, since this quantity dictates the clay sensitivity. As seen in Fig. 2.35, the K/Na ratio is an indicator of weathering (note that K and Na are concentrations of adsorbed ions), the higher undisturbed strengths correlating well with the increased ratios. The cementing actions of iron and aluminium hydroxides, however, are thought to be of greater significance in increasing the shear strength (Moum, 1967; Bjerrum, 1969a).

Moum & Rosenqvist (1957) investigated the water content and strength distribution in the vicinity of a root hole in a weathered clay profile in Oslo. In root holes and fissures, surface water with dissolved oxygen can penetrate relatively deeply into a soil profile. A weathered crust of stiff clay with increased strength will therefore surround such root holes (Fig. 2.36).

Mitchell (1976), using data from Drammen clay (Moum et al, 1971), Swedish clay (Söderblom, 1969) and Leda clay from Canada (Penner, 1965), all clays of low salinity, found a good correlation between sensitivity and percent of monovalent cations ($Na^+ + K^+$) in the pore water (expressed in meq/litre). The amount of monovalent ions in sea-water is only about 75 meq/litre, which according to Mitchell's plot would correspond to a sensitivity of around 12 to 15. It can therefore be speculated that a lower remoulded shear strength (or higher sensitivity) is due to the increase in the percentage of monovalent cations in the pore water compared to that of sea-water. This

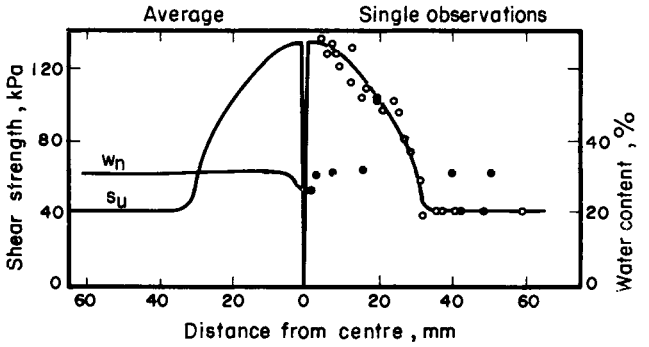

Fig. 2.36. Water content and shear strength variations around a root hole at 4.5 m depth at Uelands Gate, Oslo (Moum & Rosenqvist, 1957).

can only occur when the divalent ions are removed or complexed out of the system, as postulated by Söderblom (1974a) (see Section 2.10.3).

The effect of weathering on strength and compressibility has been explained by Bjerrum (1967) in the manner illustrated in Fig. 2.37. As a result of weathering, the clay has developed an increased resistance against deformation, which reveals itself by an apparent precompression value, σ'_{vc} (sometimes denoted by p_c) (Fig. 2.37a). Experimental verification of this process is shown in Fig. 2.37b, where a sample of Norwegian clay from Drammen was consolidated at a pressure exceeding the previous overburden. The sample was then permeated with a solution rich in K^+ ions. When the K^+/Na^+ ratio had increased from 0.21 to 34.2, this sample was further loaded in small increments. The critical pressure that developed amounted to 550 kPa, compared to 353 kPa under which the sample was consolidated. In addition, the e—log σ'_v curve shifted to the right. Hence, the compressibility was decreased by the weathering process. As can be seen in Fig. 2.37b, a characteristic feature of a weathered clay is that its e—log σ'_v curve does not show a pronounced change at the σ'_{vc} value (Bjerrum, 1973).

2.9.5 Leaching

Leaching may be defined as a process which removes material in solution (e.g. salts) from a section in the soil profile. The process can take place under a hydraulic gradient or by diffusion. Both mechanisms have been simulated in laboratory experiments. Interest in leaching gained momentum after Rosenqvist (1946, 1953) proposed his salt leaching theory to explain the behaviour of quick clays in Norway.

The geological events which lead to the leaching of a soft clay deposit begin when it becomes dry land, although Rosenqvist (1953) thought that leaching can also occur in subaquatic sediments near the shore, since fresh groundwater mixes with salt-water only slowly. Leaching requires the availability of fresh-water, which may be in the form of infiltrating rainwater (leaching from

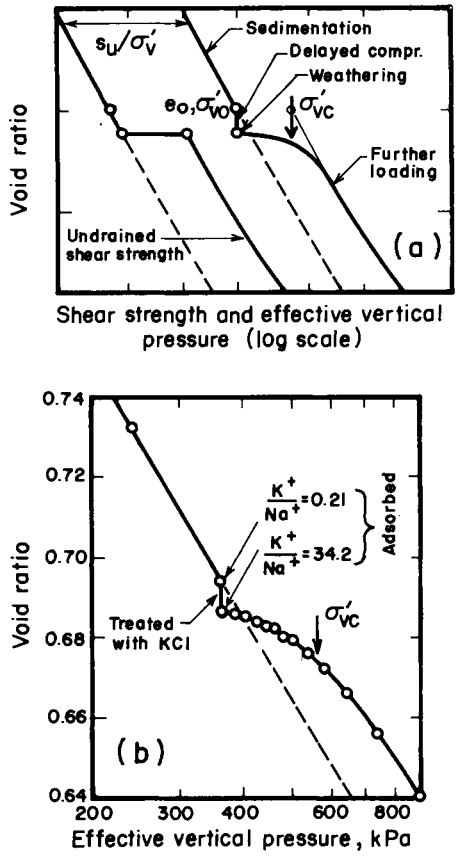

Fig. 2.37. Effect of weathering on strength and compressibility of a normally consolidated clay: (a) general principle, (b) result of laboratory test on a lean clay from Drammen (Bjerrum, 1967).

the top), or slow upward-moving artesian ground-water as described by Bjerrum (1967) for the Drammen Valley (Fig. 2.34). Leaching is thus intimately connected with chemical weathering. In the Scandinavian and Canadian marine clays, leaching has been facilitated by the enormous isostatic uplift of the land. In the soft clay deposits located in the flat deltaic plains of Southeast Asia, leaching has been caused mainly by rain-water and by the annual flood-water of the rivers. The salt content is therefore low near the surface and tends to increase with depth; it also diminishes with increasing distance from the present coast-line, as illustrated in Fig. 2.38 for the case of Bangkok clay.

Extensive laboratory investigations on leaching of both natural and artificially sedimented clays have been carried out by Rosenqvist (1946), Skempton & Northey (1952), Bjerrum & Rosenqvist (1956), Söderblom (1969), Torrance (1974), and Woo & Moh (1977). The main objective of these experiments was

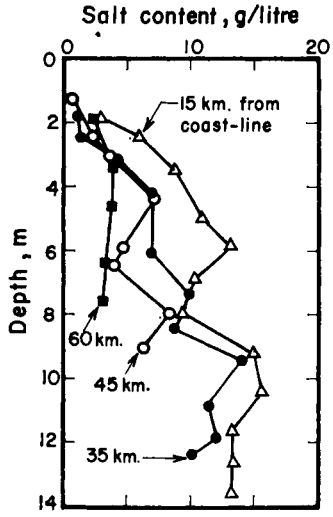

Fig. 2.38. Variation of salt content in soft Bangkok clay with depth and with distance from the coast-line.

to assess the effect of leaching on the sensitivity. Some important results of these investigations will be summarized here.

Bjerrum & Rosenqvist (1956) resedimented and consolidated a leached marine clay from Åsrum whose natural salt concentration in the pore water was only 0.05 g/litre. Resedimentation was accomplished in salt-water containing 35 g NaCl/litre. After completion of consolidation, some of the specimens in the sedimentation cylinders were leached. They found that the liquid limit decreased considerably (from 49 to 28%), while the plastic limit diminished only slightly. The plasticity index thus decreased significantly. The water content during leaching remained almost constant and, therefore, the liquidity index increased. Both the undisturbed and the remoulded strength were lower in the leached samples, inspite of no appreciable water content changes. The relative drop in strength of the remoulded specimens, however, was much higher than that of the undisturbed ones. (The effect of leaching on the undisturbed strength can be seen in Fig. 2.30). Bjerrum & Rosenqvist (1956) concluded that the decrease in undisturbed undrained strength probably resulted from an increase in the pore pressure parameter A_f. Since the leached clay represents an unstable fabric, it is possible that shear stresses are able to produce a partial collapse leading to high pore pressures and, thus, reduced strength.

Skempton & Northey (1952) ascertained from leaching experiments with Shellhaven clay from the Thames estuary that the liquid limit and remoulded strength decreased with the removal of salt, while the water content and the undisturbed strength remained practically unchanged. Consequently, the

liquidity index and sensitivity increased. They also found that the remoulded strength was a unique function of the liquidity index.

The decrease in plasticity as a result of leaching is also reflected in a decrease in activity. This is clearly shown in Fig. 2.39 for eight Norwegian clays, where the activity drops from about 0.60 to 0.25 (Bjerrum, 1954).

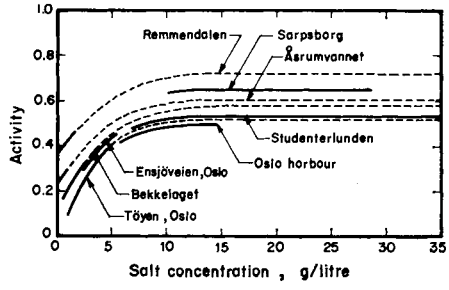

Fig. 2.39. Effect of leaching on the activity of Norwegian clays (Bjerrum, 1954).

From his leaching experiments with Norwegian clays, Torrance (1974) concluded that the liquid limit starts to decrease only when the salinity has fallen below about 5 g/litre, and a sharp drop occurs when the salt content is reduced below 2 g/litre. The same applies to the sensitivity values, as shown in Fig. 2.40. Torrance believed that the salt contents indicated by the curves of the eight clays given by Bjerrum (1954) and shown in Fig. 2.40, which indicate a major increase in sensitivity to start at a salt concentration of about 10 g/litre, were somewhat overestimated, because of the method Bjerrum used for measuring pore water salinity. Bjerrum employed a procedure in which the soil was first dried and than mixed with a known volume of water, while Torrance measured the electrical conductivity directly on pore water fluid squeezed out from a fresh sample. The latter method should give more realistic results, and is recommended. In fact, field measurements of salinities in Norwegian and Canadian quick clays (with $S_t > 50$) show salt contents usually less than 1 g/litre.

The leaching experiments on resedimented Bangkok clay reported by Woo & Moh (1977), also shown in Fig. 2.40, indicate that the sensitivity started to increase after the salt content was reduced to about 10 g/litre. This clay, however, had a different mineralogy to the Norwegian clays and consisted of 45% kaolinite, 15 to 20% illite and 35 to 40% montmorillonite. This composition is typical for soft Bangkok clay (Brenner et al, 1978), and very high sensitivities at low salt contents probably do not occur. Leaching, however, was not carried far enough to prove this. Ting & Ooi (1977) presented data on soft clay from the west coast of Peninsular Malaysia, specimens of which were leached in the laboratory. The sensitivity did not exceed the value of about 20, even at salt contents as low as 1 g/litre.

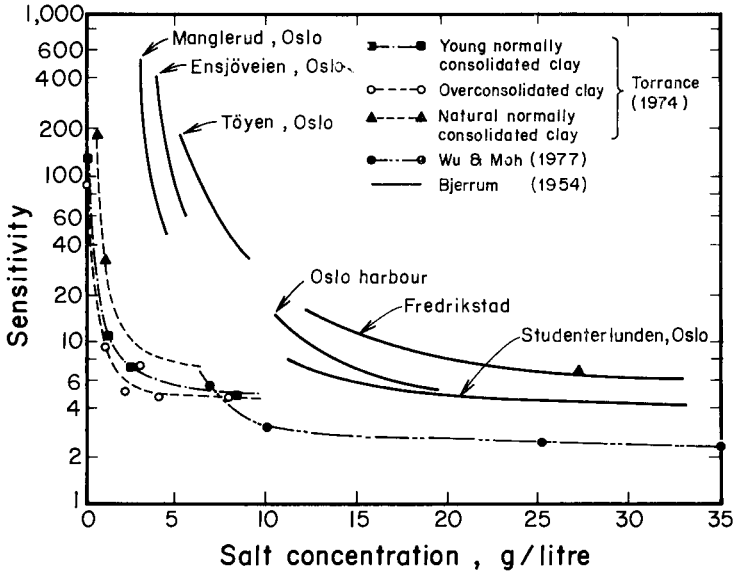

Fig. 2.40. Effect of leaching on the sensitivity of Norwegian clays and Bangkok clay.

Table 2.2 illustrates with some typical values the effect of salt content on liquid limit, remoulded strength and sensitivity of natural clays from Norway and Canada.

Table 2.2 Some data on Norwegian and Canadian clays

Property	Drammen clay (Norway)*		Leda clay (Canada)**	
	Ø. Strandgate	Engene	Ottawa	Chelsea
Natural water content, %	35	31	69	55
Liquid limit, %	37	22	60	31
Plasticity index, %	19	4	—	—
< 2 μm fraction, %	36	35	64	45
Salt content, g/litre	15	<1	1.5	0.34
Remoulded shear strength, kPa	1.2	0.1	2.1	<0.1
Sensitivity	10	80	10–20	>200

* Data from Kazi & Moum (1973) ** Data from Torrance (1975)

Leaching does not only cause reduction in undrained strength—it also increases the compressibility and results in large settlements. It has the effect of reducing the critical pressure, σ'_{vc}, so changing an 'aged' clay into a 'young' (or fresh) sediment (Bjerrum, 1973).

Figure 2.41 illustrates the principle of compressibility increase by leaching. In the Norwegian clay considered, isostatic uplift and increased pore pressure have first produced a slight decrease in effective stress. Then, during leaching,

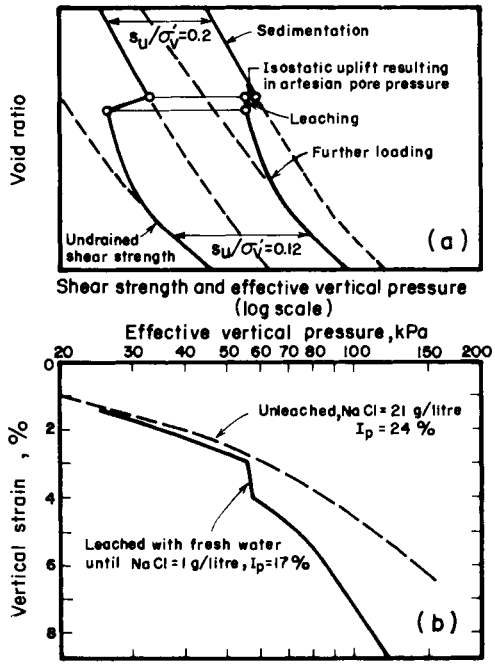

Fig. 2.41. Effect of leaching on shear strength and compressibility: (a) general principle, (b) result of leaching tests in the laboratory with a lean clay from Drammen, Norway (Bjerrum, 1967).

the water content may decrease somewhat. Loading after leaching results in large void ratio changes for small pressure increments, which produces the steep portion of the e—log σ_v' curve characteristic of sensitive clays. Only after a considerable reduction in water content has taken place, the compressibility decreases to a 'normal' value consistent with the plasticity of the clay (Bjerrum, 1967). In Fig. 2.41b, the geological history of a leached clay deposit has been simulated in the laboratory. The undisturbed specimen of clay was gradually loaded to a pressure of 55 kPa, and then leached with fresh water until the salt content had reached 1 g/litre. Subsequently, the clay was further loaded. The different compression characteristics of the leached and unleached clays are clearly visible.

There is also field evidence of a somewhat lower water content in leached clay. Bjerrum (1969b) cited a case in Drammen where quick and non-quick conditions occurred in the same layer. A comparison of the water contents at the same depths revealed that the quick (or leached) clay had a water content which was 1 to 3% lower than the salt clay.

Kazi & Moum (1973) investigated whether the settlement which occurs during leaching is accompanied by a change in fabric. They concluded that leaching does not cause any significant changes in fabric and that the increased

orientation of particles often observed in Norwegian quick clays must have been caused by delayed compression subsequent to the leaching. Leaching will affect the interparticle forces but not the flocculated structure as such.

2.9.6 Cementation

It was mentioned in Section 2.9.4 that products from weathering can form cementation bonds. Cementation is a diagenetic process, and its development depends on the material present during the sedimentation of the clay. Bjerrum (1967) defined a *cemented clay* as one which contains particles which are held together with strong bonds which are of a different nature to the bonds resulting from 'effective friction' and 'effective cohesion' which dominate in non-cemented clays. In recent years, interest on cementation has been focussed on clays of the Canadian Leda-type (Crawford, 1968; Penner & Burn, 1978), since the significance of strong bonds in soft clays is that they increase the shear strength and the sensitivity.

In Norwegian clays, cementation bonds are either very weak or non-existent (Bjerrum, 1967), while observations on the shear strength properties of a Swedish marine clay from the Göta River Valley (Lilla Edet) led Bjerrum & Wu (1960) to conclude that some rigid cementation bonds should exist.

Canadian marine clays, although of similar mineralogy and plasticity to Norwegian clays, often have considerably higher strengths than Norwegian clays. In their undisturbed state, many cannot even be classified as 'soft' clays. The difference appears to be a cementation phenomenon, sometimes in combination with true precompression from previous overburden. Of particular interest is the ability of these (overconsolidated) cemented, low plasticity clays to retain their strength and precompression after they have been unloaded; as pointed out by Bjerrum (1973), this is not found with other clays.

There currently exists no universal agreement regarding the exact nature of cementation bonds. Among the chemical compounds which have been identified as cementing agents in Leda clays are calcium carbonate, amorphous sesquioxides and amorphous silica. Work on this subject has been carried out by Kenney et al (1967), Loiselle et al (1971), Sangrey (1972a), McKyes et al (1974), Ballivy et al (1975), Yong & Sethi (1977), Yong et al (1979a, 1979b) and Moon (1979). Reference should be made to these publications for a detailed treatment of the subject, which is beyond the scope of this chapter.

There is agreement so far that cementation bonds develop between fabric units which are coated by amorphous materials, but it is not yet clear where the amorphous material originates from and when the bonds develop. It is likely that the amorphous matter has been formed by intense glacial grinding rather than by chemical weathering (as postulated by Sangrey, 1972a), and it would therefore already be present together with primary minerals and clay minerals *prior* to sedimentation (Bentley & Smalley, 1978). Cementation would thus develop between coated primary particles, which normally

dominate the clay-sized fraction in Canadian sensitive clays. Aggregation to larger fabric units would subsequently also incorporate clay minerals.

It is self-evident that cementation leads to additional strength of the clay fabric. The strength increase manifests itself in a higher undisturbed strength and in an increased critical pressure, σ'_{vc}. This is schematically illustrated in Fig. 2.42. An experimental verification of an increase in σ'_{vc} was first given by Kenney et al (1967), who showed that the removal of cementation agents caused a decrease of σ'_{vc} from about 500 kPa to 200 kPa (Fig. 2.43); similar results were obtained by Loiselle et al (1971). The compression index, however, appears generally to be unaffected by cementation, which can also be seen from Fig. 2.43.

The effects of cementation on strength have been discussed by Conlon (1966), Townsend et al (1969), Sangrey (1972b) and others. Conlon (1966) investigated a very stiff but extrasensitive clay deposit from the Toulnustouc River, Quebec, in connection with a large landslide. He concluded that structural cementation represents itself by a significant tensile strength, or by a large cohesion intercept. Townsend et al (1969) found that, at low stresses ($< \sigma'_{vc}$), cementation increases strength, and a peak in the stress-strain curve occurs at very low strains, indicating a brittle behaviour. At higher stresses, however, sufficient frictional resistance can be built up after structural breakdown, and the stress-strain curve continues to rise. They and Sangrey (1972b) concluded that the cementation bond strength is independent of the effective normal stress; tensile strength was not considered to be important.

The most significant influence of cementation is on sensitivity. Cementation increases the undisturbed strength, but when amorphous material is present in large amounts, it can increase the remoulded strength, because it can act like a thickener as a result of its chemical activity and higher surface area than the crystalline matter. Yong et al (1979a) believe that this might be due to the extremely high capacity of amorphous material to retain water. Amorphous

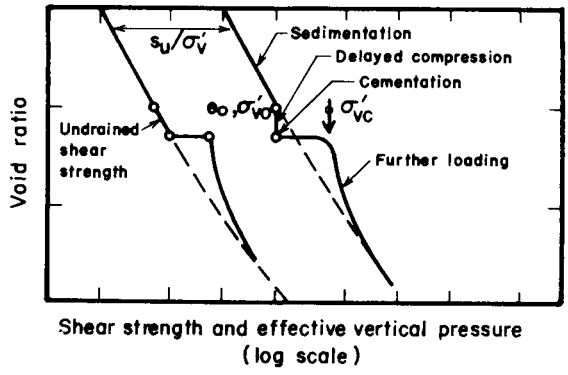

Fig. 2.42. Effect of cementation on strength and compressibility (Bjerrum, 1967).

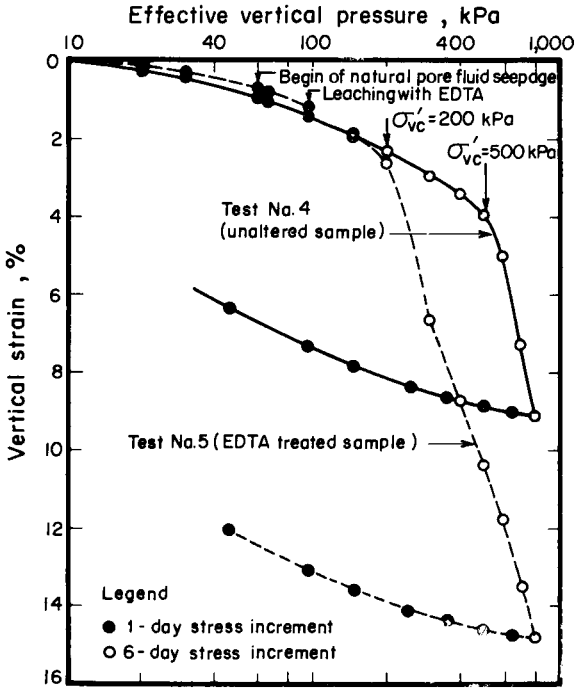

Fig. 2.43. Effect of removal of cementing agents on the compressibility of a clay from Labrador (EDTA = disodium salt of ethylene-diamine tetra-acetic acid) (Kenney et al, 1967).

material thus decreases the sensitivity, and an inverse relationship between sensitivity and content of amorphous material has been presented by Yong et al (1979a) for St Alban and Gatineau clays, although only in the range from $S_t = 5$ to $S_t \approx 20$.

Amorphous material may be responsible for the plasticity displayed by the Champlain Sea rock flour, according to Hendershot & Carson (1978). Seven more or less identical specimens of a remoulded clay were subjected to different treatments to remove amorphous materials. Progressive removal of iron compounds resulted in a marked decrease in the liquid limit, and therefore also in plasticity and sensitivity. From their preliminary results, they postulated that regional and stratigraphic variations in clay plasticity could be explained by the variation in amount and composition of amorphous content.

2.9.7 Summary of effects of physical and chemical processes

The effects of the four main processes of physical and chemical post-depositional changes which affect soft clays are summarized in Table 2.3. It can readily be seen in which direction desiccation, chemical weathering,

leaching and cementation can affect the principal geotechnical properties of soft marine clays.

Table 2.3 Effect of post-depositional processes on geotechnical properties of marine clays

Process	Geotechnical property								
	Water content	Liquid limit	Plasticity index	Liquidity index	Preconsolidation pressure	Compressibility	Undrained strength		Sensitivity
							Undisturbed	Remoulded	
Desiccation	−		±	−	+	−	+		
Chemical weathering	±	+	+	−	+	−	+	+ or −[1]	− or +[1]
Leaching	±	−	−	+	−	+	−	−	+
Cementation		+	+		+	−	+	+[2]	+

+ Increase; − Decrease; ± Little or no change.

[1] Depends on type of clay mineral (see Moum & Rosenqvist, 1961).

[2] Depends on amorphous content.

2.10 QUICK CLAYS

2.10.1 Clay sensitivity and quick clay

Sensitivity, S_t, is usually defined as the ratio of the undisturbed shear strength to the remoulded shear strength, determined either from vane tests or from unconfined compression tests. For highly sensitive soils, it is not possible to measure the remoulded strength in the laboratory by conventional means (such as the unconfined compression test), and alternative procedures have been developed. The most widely used method is the Swedish fall-cone test (see, for example, Söderblom, 1969). The sensitivity determined by this method is defined as:

$$S_t = \frac{H_3}{H_1} \tag{2.4}$$

where H_1 is a value of relative strength determined by means of the Swedish fall-cone on completely remoulded clay, and H_3 is the value for the undisturbed clay. The H-value can be obtained as a function of the cone weight and penetration depth from calibration charts.

Several classifications for sensitivity have been proposed. Some of these are summarized in Table 2.4. The first two classifications are based on the conventional definition of sensitivity, while Söderblom's classification uses the fall-cone values. This last classification was developed particularly for Swedish conditions, where it was found to be more suitable than the others. Sensitivities of 2 to 4 are very common among normally consolidated clays, and even sensitivities of 4 to 8 are frequently encountered. Heavily overconsolidated

clays and most boulder clays are insensitive ($S_t \approx 1.0$) (Skempton & Northey, 1952).

Table 2.4 Classifications of sensitivity, S_t

Skempton & Northey (1952)	Rosenqvist (1953)	Söderblom (1969)*
≈ 1.0 : Insensitive clays	≈ 1.0 : Insensitive clays	< 20: Normal clays
1–2 : Clays of low sensitivity	1–2 : Slightly sensitive clays	20–50: Semi-quick clays
2–4 : Clays of medium sensitivity	2–4 : Medium sensitive clays	> 50: Quick clays
4–8 : Sensitive clays	4–8 : Very sensitive clays	
8–16: Extra-sensitive clays	8–16: Slightly quick clays	
>16: Quick-clays	16–32: Medium quick clays	
	32–64: Very quick clays	
	> 64: Extra quick clays	

* S_t defined as H_3/H_1 (see text)

The English term *quick clay* was probably first introduced by Reusch (1901). Söderblom (1969) suggested that the Swedish term *kvicklera* was probably first used in 1767 by Törnsten; it derives from the old Nordic word *queck* (living). A quick clay was originally defined as a clay whose consistency changed by remoulding from a solid to a viscous fluid.

Söderblom (1969, 1974b) classified quick clays with respect to the amount of work needed for complete remoulding (i.e. to transform them to a liquid). He distinguished between *rapid quick clays* and *slow quick clays*, and introduced a scale from 1 to 10, called *rapidity numbers*. The scale was established based on visual examination of undisturbed quick clay samples, 40 mm in height and 50 mm in diameter, placed in a Casagrande liquid limit device, which was dropped 250 times through a height of 10 mm. Söderblom (1974b) suggested that only a clay with $S_t > 50$ and a rapidity number > 8 should be called a quick clay. (Such clays correspond to rapid quick clays). Söderblom's work shows that, for slow quick clays, sensitivity is a property that appears to develop progressively, and sensitivity and quickness depend, among other factors, on the amount of work done during remoulding (Moon, 1979). This is also illustrated in Fig. 2.44, where it can be seen how the sensitivity of two Canadian cemented quick clays develops with increasing remoulding time or amount of energy input. The undisturbed strength was first measured by a laboratory vane, and remoulding was then performed manually with palette and knife in a small container. The strength was measured every 5 minutes by the vane. For both clays, the full sensitivity was reached only after about 25 minutes of remoulding. These results can have important implications for the interpretation of landslides in quick clays. These slides usually regress backwards (Mitchell & Klugman, 1979), and as more mass becomes involved, the kinetic energy increases, which in turn might increase the fluidity (or sensitivity) of the soil (Moon, 1979).

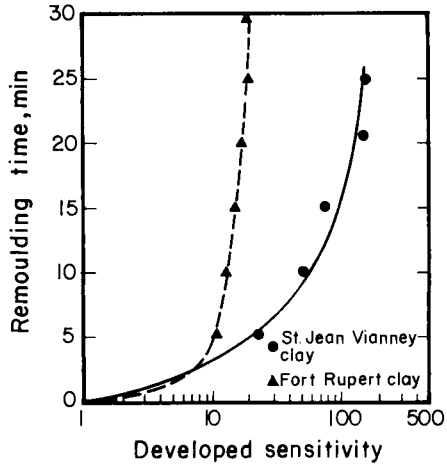

Fig. 2.44. Relationship between remoulding time and developed sensitivity for two Canadian clays (Moon, 1979).

Since the value of the remoulded strength of a quick clay which is cemented, or which has a low rapidity number for other reasons, appears to depend on the energy expended during remoulding and on the strength of the cementation bonds, standard procedures for remoulding should be established.

2.10.2 Occurrence and general characteristics of quick clays

Quick clays are found in areas which were once glaciated during the Pleistocene epoch. They have mainly been identified in northern USSR, Finland, Sweden (e.g. in the Göta River Valley and around Stockholm, Fig. 2.45), Canada (mainly in the eastern part, i.e. in regions covered previously by glacial and post-glacial marine and fresh-water bodies, Fig. 2.5) and Alaska. These areas are characterized by isostatic uplift which took place after the glaciers retreated. Glacial scouring produced large amounts of fine-grained material, which was subsequently deposited in marine, brackish and fresh-water environments. In Norway, Rosenqvist (1975a) reported that evidence exists that pre-Quaternary weathering also played an important role in supplying some of the fine material constituting those deposits. A typical feature of glacially derived clays is that the clay-sized fraction (< 2 μm) contains considerable quantities of non-clay or primary minerals.

Examples of mineralogical compositions for quick clays from Scandinavia, Canada and Alaska have been given by Liebling & Kerr (1965), Bentley & Smalley (1979), Gillott (1979) and others. The clay minerals identified were mainly illite and chlorite, but vermiculite has also been found. The content in smectites is remarkably low in all quick clays, and may even be totally absent. Among the non-clay minerals, quartz is the most abundant, followed by feldspars. Hornblende has also been identified in most samples.

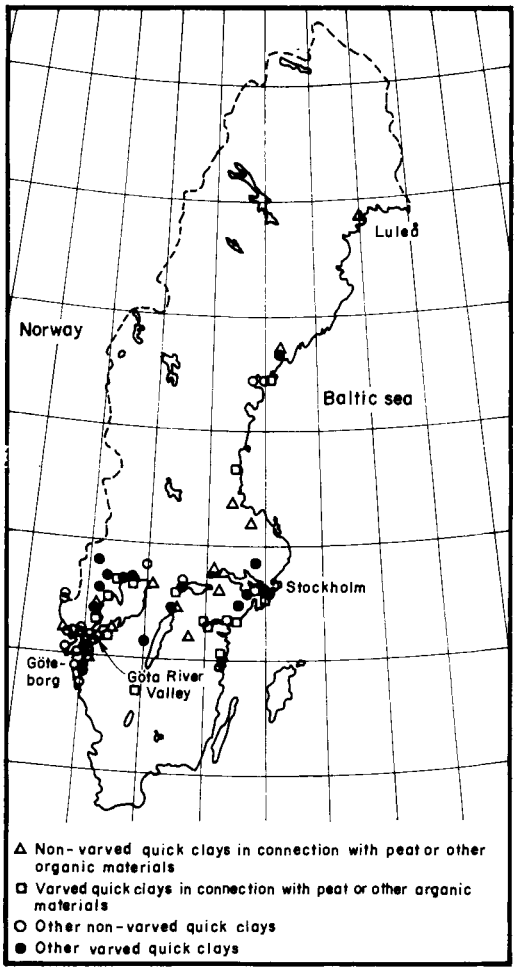

Fig. 2.45. Map showing the occurrence of various types of quick clay in Sweden (Söderblom, 1974a).

The very fine non-clay material can only be caused by glacial grinding, and since some authors (Cabrera & Smalley, 1973) believe that the high sensitivities observed are related directly to this type of material, the restricted distribution of quick clays to previously glaciated areas becomes plausible. In fact, it has been demonstrated (Moss & Green, 1975; Smalley et al, 1978) that, as a result of microfractures, quartz grains can assume the shape of flat platelets which look like mica flakes. These microfractures were observed in nearly all the quartz grains studied by Moss & Green (1975). One or more sets of such microfractures divided the grain into sheets, which were one to a few μm thick. They suggested that single-sheet particles derived from such grains will have

sizes typically in the range from 2 to 20 μm. Glacial grinding or crushing of quartz grains containing microfractures may even produce a considerable number of particles with sizes smaller than 2 μm. It is also possible to produce clay-sized quartz particles by crushing quartz in the laboratory (Smalley et al, 1978). Similarly, the same glacial grinding process may also produce significant quantities of feldspar in the clay-sized fraction, which is relatively unweathered because of the short amount of time elapsed and the low temperature of the depositional medium.

All quick clays have a small plasticity index ($\approx$ 10 to 15), much smaller than one would expect from a material of this type of particle size distribution. Liebling & Kerr (1965) reported the average amount of $<$ 2 μm material of 14 different quick clay samples as 59.3%. Most quick clays do not contain particles larger than 60 μm, and in some quick clays all particles may be smaller than 20 μm (Cabrera & Smalley, 1973; Smalley, 1976). The low plasticity can be explained by inactive non-clay particles. The activity of quick clays is usually lower than 0.5 (see also Fig. 2.39). For quartz, Skempton (1953) proposed an activity value of zero compared to illite at 0.9. Cabrera & Smalley (1973) suggested that some quick clays, because of their mineralogy and low plasticity, should not even be classified as 'clays'.

Quick clays have an open flocculated structure. Collins & McGown (1974) found that all sensitive soils they investigated contained particle matrices in their fabric. Quick clays from Drammen, Norway, frequently showed both connector and irregular aggregation assemblages. Inter-assemblage pores seem to be larger in sensitive clays than in non-sensitive ones. In Norwegian clays, however, quick clays adjacent to normal clays may have the same microfabric. Due to the open fabric, the water content of quick clays is usually high and may reach 80% (Liebling & Kerr, 1965). While higher water contents occur with many normal marine clays outside the glaciated regions, such as Bangkok clay (Fig. 2.32), quick clays are characterized by a high liquidity index, which may reach values above 3.5 (Fig. 2.31).

There exists an approximately linear relationship between the logarithm of sensitivity and the liquidity index, as illustrated in Fig. 2.46. This figures also presents a comparison between Norwegian postglacial marine clays and Canadian varved clays. The Canadian varved clays described so far in the literature do not have sensitivities higher than about 20, and typical values are around 8 to 10. Varved clays of much higher sensitivities, however, have been found in Sweden (Osterman, 1963; Söderblom, 1974a); reasons for their existence are given in Section 2.10.3 below.

Useful information on the geotechnical properties of Canadian varved clays has been published by Eden (1955), Milligan et al (1962), Soderman & Quigley (1965) and others. Kenney (1976) pointed out that, although different in macrostructure, the Canadian varved clays are in many respects similar to the unleached marine clays of Norway. For example, water contents, w_n, are approximately equal to the liquid limits, w_L. (In the silt layer of a varve, w_n

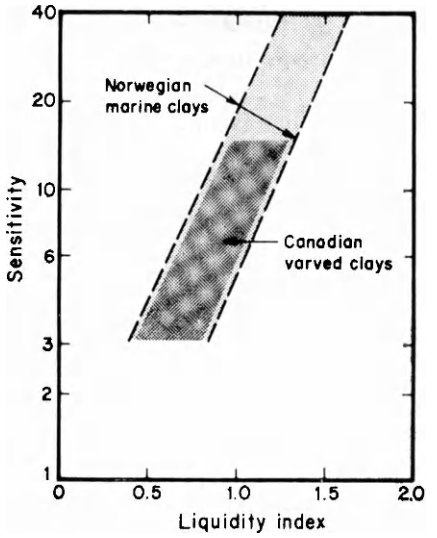

Fig. 2.46. Relationship between sensitivity and liquidity index of uncemented glacial-lake and marine clays (Kenney, 1976).

is often somewhat larger than w_L). Particle size distributions and mineralogies are also very similar. In a varved clay, the plasticity index of the dark winter layer is typically around 40 to 50%, whereas it is only 5 to 10% in the light-coloured silt layer, i.e. it is in the same range as quick clay of marine or brackish origin. Cabrera & Smalley (1973) suggested that, because of this, the silt bands might give rise to quick clay type behaviour, an important factor in the interpretation of failures in varved clays.

Figure 2.30 illustrates the similarity in strength behaviour of the two types of clay. In the varved soils, the undrained strength, as determined by the in-situ vane test, is controlled by the clay layers (Kenney, 1976). Data on varved clays from Sweden presented by Karlsson & Viberg (1967) also fall within the region indicated for Canadian varved clays.

Varved clays, inspite of being lacustrine, have with few exceptions a near-random flocculated fabric, as demonstrated by the work of Quigley & Ogunbadejo (1972). These authors inferred that an open structure could only be preserved with the development of rigid bonds (i.e. cementation, for example by carbonates) after deposition. Kenney (1976) also showed that it is not necessary to assume that flocculation occurs within the lake water to explain either the random fabric or the amount of sediment deposited during winter. It may thus be postulated that clay particles or particle aggregates settle individually, which is possible in the quiet water and with the low sediment concentration ($\approx$ 0.5 g/litre), and that flocculation takes place near the sedimentary interface where particle collisions are more likely.

2.10.3 Factors which affect sensitivity

The sensitivity of a clay is the product of a number of factors which are mostly controlled by physico-chemical and mineralogical aspects. Skempton & Northey (1952) considered four factors in their investigation on sensitivity. They were: (1) mineralogy, (2) particle size distribution, (3) thixotropy (see Section 2.10.6), and (4) microstructural stability (including the role of leaching in producing a meta-stable structure). Mitchell & Houston (1969) identified eight mechanisms which could lead to the development of sensitivity. These factors essentially are all contained in the six different phenomena cited by Mitchell (1976), namely: (1) meta-stable fabric, (2) cementation, (3) weathering, (4) thixotropic hardening, (5) leaching, ion exchange and change in monovalent/divalent cation ratio, and (6) formation or addition of dispersing agents; of these (2), (3) & (5) have been described in detail in the previous sections.

A most suitable form in which to illustrate the influence of the various factors that affect sensitivity has recently been given by Quigley (1979). The factors which produce a high undisturbed strength and those that lead to a low remoulded strength, are listed separately as numerator and denominator respectively in the sensitivity equation (Table 2.5). Most of these factors have already been discussed, but some further comments are appropriate.

Table 2.5 Factors affecting sensitivity (Quigley, 1979)

$S_t = \dfrac{s_u \text{ undisturbed}}{s_u \text{ remoulded}}$	*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 ($w_n \geqslant w_L$). Little consolidation, 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 μm fraction. High primary mineral $\equiv$ low clay mineral content. (3) High zeta potential. (Expanded double-layers = high interparticle repulsion = dispersed or peptized state). Low salinity by leaching (< 2 g/litre). Organic dispersants } (Anion adsorption). Inorganic dispersants } High monovalent cation adsorption relative to divalent cations. (4) Low amorphous content. (5) Low smectite content.

(1) *Slow increase in sediment load.* In order to acquire the necessary open flocculated structure, the rate of sediment deposition must be sufficiently slow to enable bonds to develop to provide the structural rigidity to resist compression.

(2) *Low specific surface of soil particles.* Quick clays are characterized by considerable amounts of rock flour which contain little clay material. The important role the very fine non-clay minerals might play in quick clays was pointed out by Cabrera & Smalley (1973). They proposed that 'short-range' interparticle bonds develop between the non-clay particles which are uncharged, while 'long-range' bonds exist between the charged clay particles. The short-range forces only become important when the particles are very small and their weight becomes comparable with the magnitude of these forces. Sedimentation in such a force field could lead to an open fabric with bonds developing at the particle contacts. Once the bonds are broken, the strength drops to a very low value, since the remoulded material consists of components of low activity and low plasticity.

(3) *High zeta potential.* An increase in interparticle repulsive forces leads to a more dispersed soil fabric and thus to lower strength. If after remoulding a soil, its fabric remains dispersed, the sensitivity of the soil is high. A convenient way to quantify dispersion is the zeta or electrokinetic potential. Any reaction that expands the diffuse double-layer surrounding a clay particle increases the zeta potential and reduces the remoulded strength. The correlation between zeta potential and sensitivity was first pointed out by Penner (1964, 1965) from measurements on Leda clay, and similar data were later published by Yong et al (1979a) for St Alban and Gatineau clays. In each case, there is a near-linear increase in the logarithm of sensitivity with the zeta potential, although the rates of increase are different.

(4) *Dispersing agents and organic matter.* Certain inorganic and particularly organic substances can expand the double-layer and thus act as dispersants or peptizers. Among the inorganic dispersing agents are silicates, phosphates, sulphides and bicarbonates. The work of Söderblom (1974a) provides a good account of the large variety of organic substances and their significance to clay properties. The dispersing properties of organic matter is only one of the possible effects these materials can exert on soil. Organic matter can also act as a cementing gel, or it produces a fissured macrostructure accompanied by syneresis. Details of the nature of cementing organic colloids can be found in the monograph by Söderblom (1974a); for a comprehensive treatment of soil organic matter in general, the reader may refer to Kononova (1966) or to texts on soil science. The literature on soil organic matter as it affects the engineering properties of clays is sparse. Because of the complex interaction between organic substances and clay particles, predictions of their effects on engineering properties are difficult to make.

When organic substances act as dispersing agents they can bind the divalent ions, i.e. Ca^{2+} and Mg^{2+} and change the ionic composition of the pore water. Thus an otherwise 'normal' leached clay can become quick. By the same mechanism it is also possible that fresh-water clay may turn quick. An example was cited by Söderblom (1974a) where a fresh-water sedimented clay below a layer of peat was found to be quick. Söderblom also demonstrated experimentally that a varved clay with normal sensitivity (see Söderblom's classification for sensitivity, Table 2.4) could be transformed into a quick clay by treating it with a dispersing agent. Normal clays and quick clays can thus be found to exist in the same sedimentation area.

2.10.4 Role of salinity in sensitivity

Originally, it was believed that low salinity as a result of leaching was the dominant factor in producing a quick clay (Rosenqvist, 1946, 1953). This concept seemed to work well for Norwegian clays, but Penner (1964, 1965) and Söderblom (1969, 1970) found for clays from Canada and Sweden respectively that the sensitivity is not uniquely dependent on the amount of salt present, and that non-sensitive clays with low pore water salt concentrations also exist. This is illustrated in Fig. 2.47, which shows salt concentrations, expressed as specific conductivity values, plotted against sensitivity for Swedish clays from the Göta River Valley; there is an approximately linear correlation between the conductivity in a clay paste and that of its pore fluid, according to Penner (c.f. Söderblom, 1969). While low salt concentration is a necessary requirement for quick clay formation, it is not a sufficient one, and the original salt leaching theory established by Rosenqvist (1946) is only a special case of a more general concept.

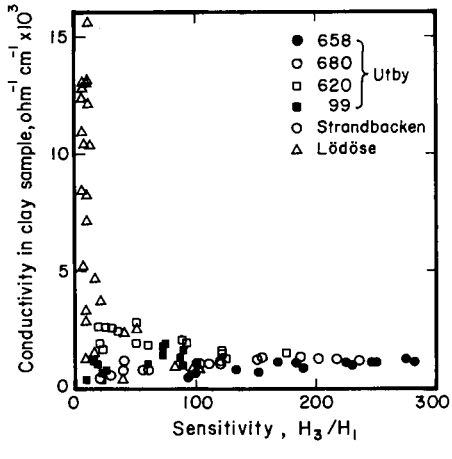

Fig. 2.47. Relationship between conductivity and sensitivity for clay samples from the Göta River Valley, Sweden (Söderblom, 1969).

In his research on the effect of salts on quick clay formation, Söderblom (1969) carried out some leaching experiments, the results of which are fundamental to the understanding of quick clay formation. A natural salt clay was first leached by dialysis (sample enclosed in semi-permeable membrane) against a 3.5% NaCl solution. Subsequently, the clay was dialyzed against distilled water. As a result, the clay became quick. The experiment was then repeated with $CaCl_2$, and no quick clay was formed. In each case, the ionic composition of the pore water in the clay was determined by means of paper chromatography. Söderblom found that, after treatment with $CaCl_2$, some sodium ions were still present, as were considerable amounts of Ca^{2+} and Mg^{2+} ions, and the pore water had an ionic composition similar to a leached natural non-quick clay.

Söderblom explained his observations by means of the Donnan effect which states that, when a colloid system (such as a clay-water system) is dialyzed (leached), the monovalent ions more easily leave the system, since they are exchanged by the divalent ions which replace them in the adsorbed layer around the clay particle. An enrichment in divalent ions therefore results. The ionic composition of the pore water is thus of great importance. Quick clays contain sodium as the dominating cation in the pore water, while non-quick clays have calcium and magnesium ions. This also agrees, of course, with the observations made on the profile of Fig. 2.35.

Söderblom (1974a) also reported that in Sweden two types of ground-water are distinguished, namely: (1) hard water, which contains Ca^{2+} and Mg^{2+} as dominating cations, and (2) soft ground-water, which contains mainly Na^{+}. Clays leached by the hard ground-water have never been found to be quick.

In summary, it can be stated that a quick clay cannot be obtained by leaching alone if the pore water contains Na^{+}, K^{+}, Ca^{2+} and Mg^{2+} cations, since leaching will only cause a relative enrichment of the divalent cations, and the clay will remain non-quick.

2.10.5 General theory of quick clay formation

Rosenqvist (1975b, 1978) formulated a general theory for quick clay formation which incorporates all the main features observed with such deposits in Scandinavia and Canada. The following three processes explaining the quick clay phenomenon are, with minor modifications, directly quoted from his later paper:

(1) The clay particles of non-expanding minerals are sedimented in a flocculated state, because the diffuse layer (or zeta) potential is low, either because of salinity in the water or because of adsorption of strongly-held counterions such as Fe^{3+} or Al^{+3}.

(2) After deposition and slight consolidation, the diffuse layer potential of the minerals is increased. This may be caused by leaching, reduction

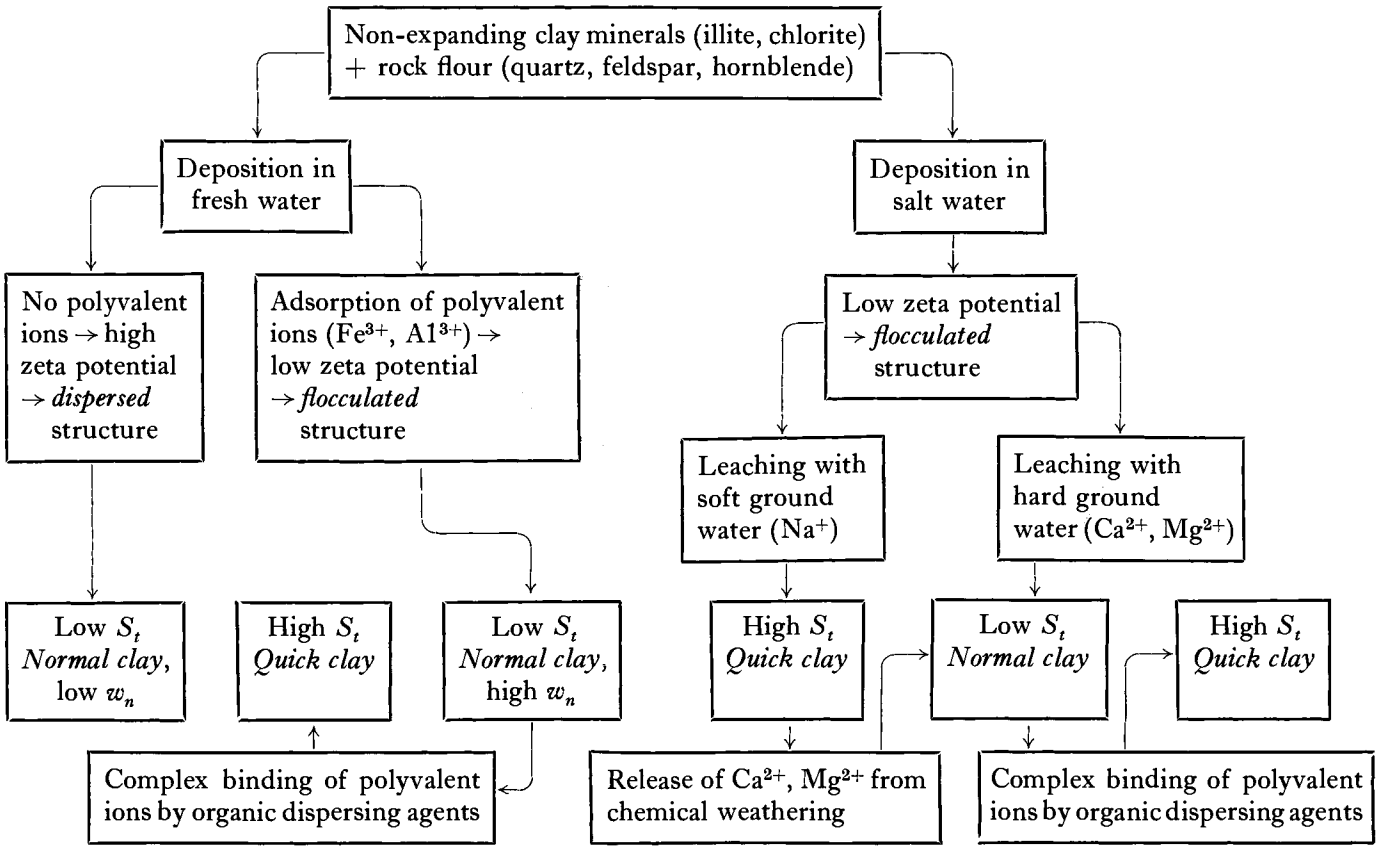

Fig. 2.48. Principles of quick clay formation.

of Fe^{3+} to Fe^{2+} or by complexing of the polyvalent counterions. The original disordered structure of the flocculated clay remains.

(3) By mechanical deformation, an alignment of the minerals takes place. Because of the high mutual repulsion, reflocculation is inhibited, and only after considerable reduction in water content is it possible for the minerals to form new contacts.

Figure 2.48 shows in diagrammatic form the principles of quick clay formation. The various processes have been described in the preceding sections and do not need to be repeated here. One further point is worthy of note which relates to the water content of fresh-water clays. It can be seen that, depending on the chemical environment, fresh-water clays can occur with a flocculated structure and, therefore, with a high water content similar to salt-water clays; this has been reported, for example, by Kenney (1976). It is more usual, however, for fresh-water clays to be found with a denser structure and lower water content (see microfabric studies by Pusch, 1973b, and data by Bjerrum & Rosenqvist (1956) shown in Fig. 2.30).

2.10.6 Thixotropy in relation to sensitivity

Originally, it was suggested that sensitivity could be explained as a thixotropic phenomenon. *Thixotropy* was defined by Mitchell (1960) as "an isothermal, reversible, time-dependent process occurring under conditions of constant composition and volume, whereby a material stiffens while at rest and softens or liquefies upon remoulding". The characteristics of purely and partially thixotropic materials are shown in Fig. 2.49.

The mechanism of thixotropic behaviour of a clay paste has been explained as follows (Mitchell, 1960, Jacobsson & Pusch, 1972). Remoulding of a

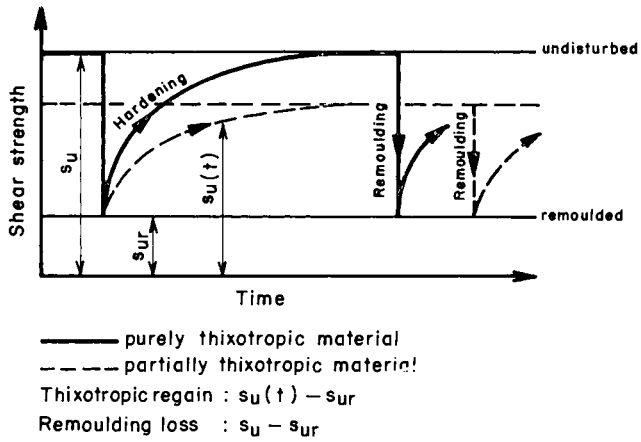

Fig. 2.49. Characteristics of purely and partially thixotropic materials.

thixotropic material by stirring, agitation, compaction, etc causes a breakdown of particle links. The particles then form a fabric which is more oriented and compatible with the energy of remoulding. Once the disturbing forces have ceased, the deflocculated fabric is no longer in equilibrium with the inter-particle forces and, provided attraction dominates, particles tend to flocculate again. The degree of reformation of the original structure depends on the number of very small particles present and the ease with which they can move. The aggregation of particles and particle groups also leads to an arrangement of the adsorbed cation—water molecule structure such that a lower energy state is achieved. Strength regain is also caused by increased pore water tension with time, resulting from the difference in osmotic pressure between particle aggregates and the surrounding medium. The increased effective stress due to pore water tension can account for the observed increase in undrained strength (Mitchell, 1976).

Investigation be electron microscopy of the microstructural rearrangement of Swedish quick clay by Jacobsson & Pusch (1972) showed that the thixotropic strength regain was intimately connected with the restoration of particle links and with delayed reordering of the water phase. The microstructural porosity, P/T, (see Section 2.6.2) decreased with time compared to the value just after remoulding, indicating an aggregation of dispersed small particles. The state of order of water molecules was examined by the spin-echo technique. Thixotropy, however, does not seem to be a satisfactory explanation for the sensitivity of a quick clay, since such a clay will not regain its original undisturbed strength after remoulding unless its water content is decreased considerably.

Skempton & Northey (1952) plotted the ratio of thixotropic regain to remoulding loss of six natural clays (Fig. 2.50). The clays were remoulded at their liquid limits and then stored in containers at constant volume and water content. Strength measurements were made periodically either by laboratory vane or by unconfined compression tests. All clays were of late glacial or postglacial origin, and those shown in Fig. 2.50a were medium to very sensitive, while those plotted in Fig. 2.50b were slightly quick to extra quick (using Rosenqvist's sensitivity classification). The approximate geological age of these clays is indicated in the upper right-hand corner of each figure. From the shape of the curves in Fig. 2.50, it may be judged whether the original strength could ever be regained if the clays were allowed to harden for a time period corresponding to their geological age. It seems very unlikely that the quick clay from Horten, Norway, would ever regain its natural undisturbed strength, while the Detroit clays (which have a very low sensitivity) might be able to do so.

It would seem from the data of Skempton & Northey shown in Fig. 2.50, that low sensitivities could be explained by thixotropy. Mesri et al (1975), for example, stated that Mexico City clay apparently owes its sensitivity ($\approx$ 4 to 20) entirely to thixotropy. Mitchell & Houston (1969), however, pointed out that

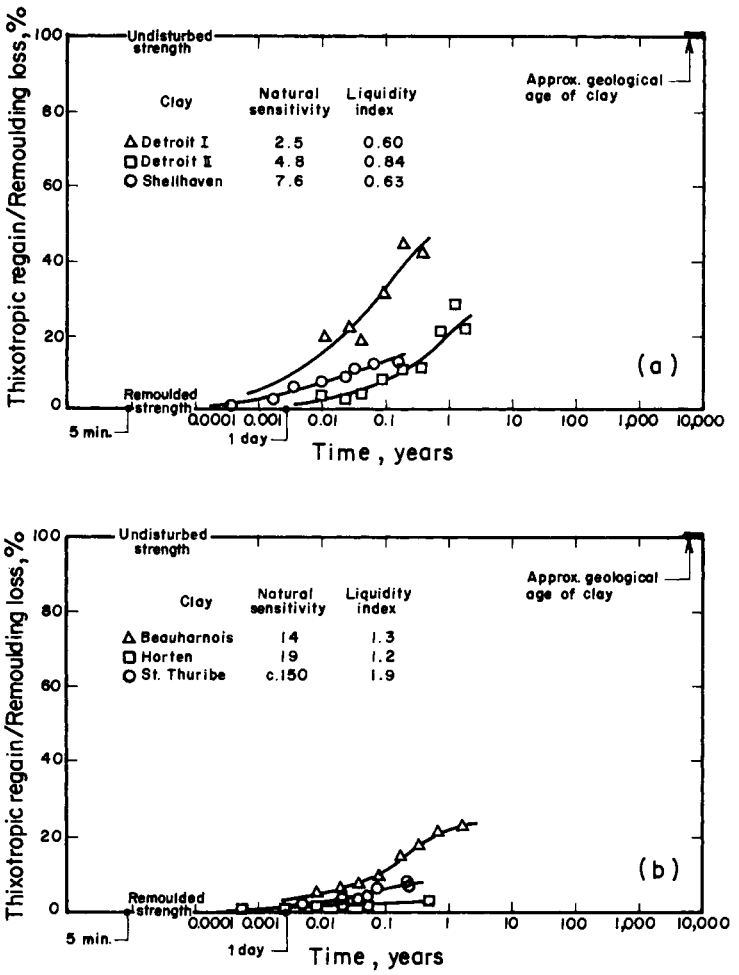

Fig. 2.50. Thixotropic strength regain of clays of various sensitivities: (a) medium to very sensitive clays, (b) slightly quick to extra quick clays (Skempton & Northey, 1952).

these results do not prove that low sensitivity values are actually due to thixotropy. The geological history of a natural clay deposit involves deposition at high water content, consolidation and changes in pore water chemistry, and it bears little resemblance to the laboratory experiments on which the curves in Fig. 2.50 are based, i.e. constant water content and pore water chemistry. In fact, studies by Skempton & Northey have shown that thixotropy depends also on the water content in relation to the liquid limit, and on the pore fluid composition.

Thus, it is not possible to state to what extent thixotropic hardening actually contributes to sensitivity. Mitchell (1960) suggested that sensitivities of up to about 8 may be possible as a result of thixotropic hardening.

2.11 CLASSIFICATION OF SOFT CLAYS

The preceding review of post-depositional processes has demonstrated how the geological history can alter the present geotechnical behaviour of a normally consolidated clay. It is therefore reasonable to devise a classification which is based on geological history, and which is associated with those geotechnical characteristics in which changes can take place. Such properties are: (1) water content and Atterberg limits, (2) variation of vane strength and preconsolidation pressure with depth, (3) shape of e—log σ'_v curve, and (4) sensitivity. Table 2.6 shows a classification of normally consolidated clays proposed by Bjerrum (1972) on this basis. This classification relies heavily on the critical pressure and on the natural water content in relation to the liquid and plastic limits.

Table 2.6 Classification of soft clays (Bjerrum, 1972)

	Classification	Water content	Shear strength	Compressibility
Weathered clays in upper crust	Frost treated, dried-out clays	$w_n \approx w_p$	Very stiff, fissured, with open cracks	—
	Dried-out clays	$w_n \approx w_p$	Very stiff, fissured	Low compressibility
	Weathered clays	$w_p < w_n < w_L$	Shear strength decreases with depth	Low compressibility, curved e—log σ'_v curve
Unweathered clays	Young normally consolidated clays	$w_n \approx w_L$	s_u/σ'_{vo} constant with depth	$\sigma'_{vc} \approx \sigma'_{vo}$
	Aged normally consolidated clays	$w_n \approx w_L$	s_u/σ'_{vo} constant with depth	$\sigma'_{vc}/\sigma'_{vo}$ constant with depth
	Young normally consolidated quick clays	$w_L < w_n$	s_u/σ'_{vo} constant with depth	$\sigma'_{vc} \approx \sigma'_{vo}$
	Aged normally consolidated quick clays	$w_L < w_n$	s_u/σ'_{vo} constant with depth	$\sigma'_{vc}/\sigma'_{vo}$ constant with depth

The importance of the ratio s_u/σ'_{vo} is evident from Table 2.6. To conclude this chapter, a brief review is made here of the much-used relationship between the strength ratio s_u/σ'_{vo} (or s_u/σ'_{vc}) and the plasticity index.

Skempton (1948a, 1948b) recognized that, in a thick clay stratum, a relationship between the shear strength and the preconsolidation pressure, σ'_{vc}, should exist. For truly normally consolidated clays ($\sigma'_{vc} = \sigma'_{vo}$), he first proposed a correlation between s_u/σ'_{vo} and the liquid limit (Skempton, 1948c). As more field data became available, Skempton (1954) showed that a linear correlation of s_u/σ'_{vo} with plasticity index, I_p, could be obtained which was of the form:

$$s_u/\sigma'_{vo} = 0.11 + 0.0037\, I_p \tag{2.5}$$

This relationship was confirmed by additional data from Norwegian clays (Skempton, 1957).

Skempton & Henkel (1953) pointed out that the ratio s_u/σ'_{vo} was not the same when derived from field vane tests as it was from undrained triaxial compression tests. Today, it is generally known that the undrained strength depends both on the in-situ conditions and on the test procedure (because of such factors as anisotropy, strain rate, stress path, progressive failure, etc), and the value of s_u/σ'_{vo} for a clay can vary widely; examples of this have been given by Gregersen & Løken (1979) and Hanzawa (1979). Usually, vane tests give a lower strength than triaxial compression tests consolidated at in-situ stresses. In fact, it can be shown theoretically (Kenney, 1959) by expressing s_u/σ'_{vo} as a function of A_f (pore pressure parameter at failure) and φ' (effective friction angle), that the s_u/σ'_{vo} ratio decreases with I_p when the strength is measured in a triaxial compression test. Values of s_u/σ'_{vo} when correlated with I_p should thus be based on vane strengths.

Bjerrum (1954) proposed a curved relationship between s_u/σ'_{vo} and I_p for Norwegian marine clays which was similar to Skempton's linear relationship, but which gave better agreement in the low plasticity range ($I_p < 20$), into which most of the Norwegian clays fall. Bjerrum (1967) further demonstrated that, in the case of $\sigma'_{vc} \neq \sigma'_{vo}$, if the ratio s_u/σ'_{vc} is plotted instead of s_u/σ'_{vo}, a unique relationship with I_p could still be obtained. Finally, Bjerrum (1972) proposed the relationship for 'young' and 'aged' clays as shown in Fig. 2.30. When the s_u/σ'_{vo} values are multiplied by $\sigma'_{vo}/\sigma'_{vc}$ (Fig. 2.29), the two curves coincide, as pointed out by Mesri (1975). Interestingly, Mesri (1975) also showed that, when the ratio s_u/σ'_{vc} is further multiplied by Bjerrum's vane strength correction factor, μ, a practically constant value of 0.22 is obtained, independent of plasticity index. A similar approach has also been attempted by Yudhbir (1979) starting from Skempton's equation and correcting it for strain rate and anisotropy effects.

Most soft clays with a flocculated fabric fall into the region in Fig. 2.30 between the 'aged' and the 'young' curve, including leached clays. Leaching produces a decrease in I_p, but simultaneously a reduction in undisturbed strength occurs (Section 2.9.5). Canadian quick clays sometimes exhibit s_u/σ'_{vo} values which are much higher than should correspond to their low plasticity index. For example, the clay in Fig. 2.31 has an s_u/σ'_{vo} value of 0.55 and a $\sigma'_{vc}/\sigma'_{vo}$ of about 2.0, but the plasticity index is only around 25. For such cases, i.e. highly sensitive clays with $w_n > w_L$, Bjerrum (c.f. Dascal et al, 1972) proposed the use of a modified plasticity index, I_{pm}, which is defined as the difference between the natural water content and the plastic limit. This is plausible, since at the time of deposition, w_L, was probably higher and was decreased due to leaching, but the rigid cementation bonds in these clays prevented a reduction in strength.

The use of the relationship of s_u/σ'_{vo} or s_u/σ'_{vc} versus I_p is a useful empirical correlation which certainly has its merits, at least on a regional scale. Specific

local conditions may produce deviations, such as those reported by Lumb (1977) who found for Hong Kong marine clays that it was necessary to take into account the sulphur content, in addition to I_p, in order to arrive at a satisfactory correlation. The quality of test data is another factor which should not be overlooked when comparing data from different sites and sources. Unusual mineralogical composition and fabric, such as encountered in Mexico City clay (Fig. 2.8) will also cause large deviations. For Mexico City clay, the $\sigma'_{vc}/\sigma'_{vo}$ value is 1.4 to 1.5, and the plasticity index is of the order of 350% (Mesri et al, 1975).

Schmertmann & Morgenstern (1977) discouraged the use of empirical relationships which are based solely on I_p, stating that I_p is not a variable which characterizes sufficiently the engineering geology of a soft clay deposit, and that global correlations of this kind should be discontinued. This may be desirable from a research point of view if further progress is to be made in the interpretation of the effects of geological history on soil behaviour. On the other hand, soil engineering practice relies heavily on soil parameters which can be determined easily, and empirical relations are an obvious step in an overall data analysis. In this sense, the strength ratio in relation to the plasticity index can often serve as an indicator of geological history, and also as an aid in the assessment of the quality of strength data in fairly homogeneous soft clay deposits.

2.12 CONCLUSIONS

Terrestrial soft clay deposits belong to the youngest sediments of the earth's surface. They were formed in various depositional environments, but clays of marine origin dominate. The most significant geological event in the formation of soft clay deposits was the eustatic change in sea-level after the last glaciation. Of the variables which characterize the depositional environment for a clayey sediment, the salinity conditions are probably most important. The role of organic matter in the suspension medium and its effect on the formation of clay-organic aggregates is still relatively unknown.

The state of consolidation (and thus the depth of burial) of a normally consolidated clay can be judged from its sedimentation compression curve, i.e. the void ratio (or water content) versus effective overburden pressure relationship. The relative position of this curve depends on the plasticity of the clay.

While the depositional environment is responsible mainly for the present macrostructure of a clay deposit, the present geotechnical characteristics (strength, compressibility, plasticity, sensitivity, etc) have in general been determined, in addition to the overburden, by physical and chemical post-depositional processes to which the sediment has been subjected during its short geological history, and which have affected the stability of the microfabric.

In previously glaciated regions, the eustatic sea-level rise was combined with the isostatic uplift of the land. Special conditions with regard to sediment composition, depositional environment and post-depositional processes in these areas have led to the formation of clays of extremely high sensitivities (*quick* clays). These have been troublesome to the construction industry and a danger to property and lives in many instances.

As a result of intensive research in the past thirty years, particularly in Norway, Sweden and eastern Canada, the effects of post-depositional physical and chemical processes on the engineering properties of soft clays are now fairly well understood. The importance of geological and physico-chemical factors in the interpretation and analysis of such geotechnical problems as landslides and the settlement of structures has been clearly recognized. However, there still exists little information on the engineering geology of soft clays in tropical and subtropical regions, particularly in regard to geochemical processes. It is hoped that the increased interest in the development potential of these soft clay areas will also stimulate research into their most recent geological history and its implications for geotechnical practice.

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Chapter 3

EXPLORATION, SAMPLING AND IN-SITU TESTING OF SOFT CLAY

by

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Mr Andresen is the author of a number of papers. He has been involved for many years with the Committees on Sounding and Site Investigation of the Norwegian Geotechnical Society. From 1961 to 1969, he was a member of the International Group on Soil Sampling of the International Society for Soil Mechanics and Foundation Engineering, and he has been a member of the ISSMFE Subcommittee on Site Investigation since 1977.

Chapter 3

EXPLORATION, SAMPLING AND IN-SITU TESTING OF SOFT CLAY

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3.1 INTRODUCTION

Because of the huge quantity of soil involved in any soil engineering problem, a site investigation can, for practical and economic reasons, only cover a very small proportion of the total soil mass. The data obtained, therefore, must be extended on the basis of previous local experience. Generally, the local code of practice will influence the lay-out of the exploration and the selection of equipment and tools. As a rule, exploration is a step-by-step procedure which may be divided into the three main stages:

(1) preliminary investigation,

(2) detailed investigation, and

(3) supplementary investigation and construction control.

3.1.1 Data collection

A preliminary investigation may be limited to the gathering of the available geotechnical and geological information. In the United States, Canada and France, systems for the collection, storage and retrieval of geotechnical data have been developed, and in Sweden there exists a proposal for the development of a central geotechnical soil data bank (Dahlberg, 1974). Geological maps, which include soil data, are currently being developed by government authorities in many countries.

Since 1942, the Soil Mechanics Division of the city of Oslo has registered all soil investigations in Oslo on the basis of voluntary contributions of information from companies and institutions carrying out soil investigations in the area. Approximately 60 maps to a scale of 1:1000 have so far been published.

3.1.2 Codes of practice

In some countries, soil investigations are mostly carried out by drilling contractors under the supervision of an engineer from a consultant, who often supplies special equipment if necessary. In other areas (e.g. Scandinavia and

The Netherlands), the complete soil investigation is carried out by soil consultants and specialist institutions.

The layout of soil investigations is different in different parts of the world. In the USA and Canada, sampling in soft clays is usually carried out from the bottom of prebored holes with open thin-walled Shelby tube samplers. The vane test is seldom performed but, when it is, it is mostly used from the bottom of prebored holes.

In Finland, Denmark, Sweden and Norway, the practice has always been to start a field investigation with a number of simple soundings. The aim of these soundings is to determine the stratification and to facilitate the selection of softer areas which require sampling and more extensive in-situ testing. In Norway, the ratios among soundings, vane borings and sampling in soft clay are of the order of 10:5:1.

In Scandinavia, nearly all sounding, boring and sampling of soft clay is carried out as displacement borings. The information obtained from sounding alone is not used as a basis for calculations, and field vane borings together with undisturbed sampling are considered necessary. In Fennoscandia, the soft clay deposits may be varved, but they are normally homogeneous, and the methods and equipment in use have been designed for these soil conditions.

In The Netherlands, the soils are very stratified, and the Dutch cone penetrometer is especially suited for this type of soil. The layout of soil investigations is quite similar to the Nordic practice, but vane testing is seldom used (because of stratification), and results from cone penetrometer tests are used for calculations. The stratified soil conditions also provide a demand for continuous sampling.

The layout of soil explorations is often standardized and predetermined. However, soil investigation programmes which are continually adjusted in accordance with the information gained are preferable. Supplementary investigations and special tests may be necessary as the last part of a soil investigation. Load tests, test pits and test fills are expensive but, in many cases, justified. Special tests and more detailed and careful investigations may lead to safer structures.

In the opinion of the Author, it is of extreme importance that the project engineer responsible always visits the site during the field investigations.

3.1.3 Drilling rigs

Soft clays are easily penetrated; borings may be carried out manually, and manually operated equipment is commonly used in many countries. Most mechanical drilling rigs are designed for auger drilling and standard penetration tests (Fig. 3.1), i.e. methods originally designed for stiffer materials. In Fig. 3.2 is shown the specification for a multipurpose drilling rig developed by the Norwegian Geotechnical Institute (NGI) in cooperation with the Norwegian Roads Authority.

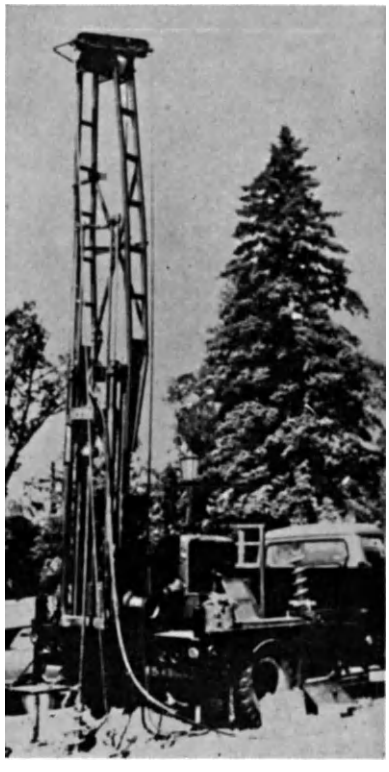


Fig. 3.1. Acker drilling rig.

Special drilling rigs for cone penetration tests have been built (Holden, 1974a; Fig. 3.3). In Fig. 3.4, a drilling rig of Swedish origin for use in softer clay is shown; this rig is designed for fully automatic sounding (Nilsson, 1975) and can also be used for vane boring and sampling.

3.2 SAMPLING

3.2.1 Undisturbed and disturbed samples

Samples are used for identification and for the determination of soil properties, and they are required for nearly all soil investigations. The choice of sampler type and sampling technique depends on the type of soil and the necessity for obtaining *undisturbed* samples. Undisturbed samples can give the necessary strength and compression parameters for solving most geotechnical problems. *Disturbed* (or *representative*) samples can give valuable information on chemical properties, plasticity and water content of the soil, as well as yielding information for classification. In some cases, disturbed samples are sufficient. Usually, however, the difference in cost between undisturbed and disturbed samples in soft clay is small.

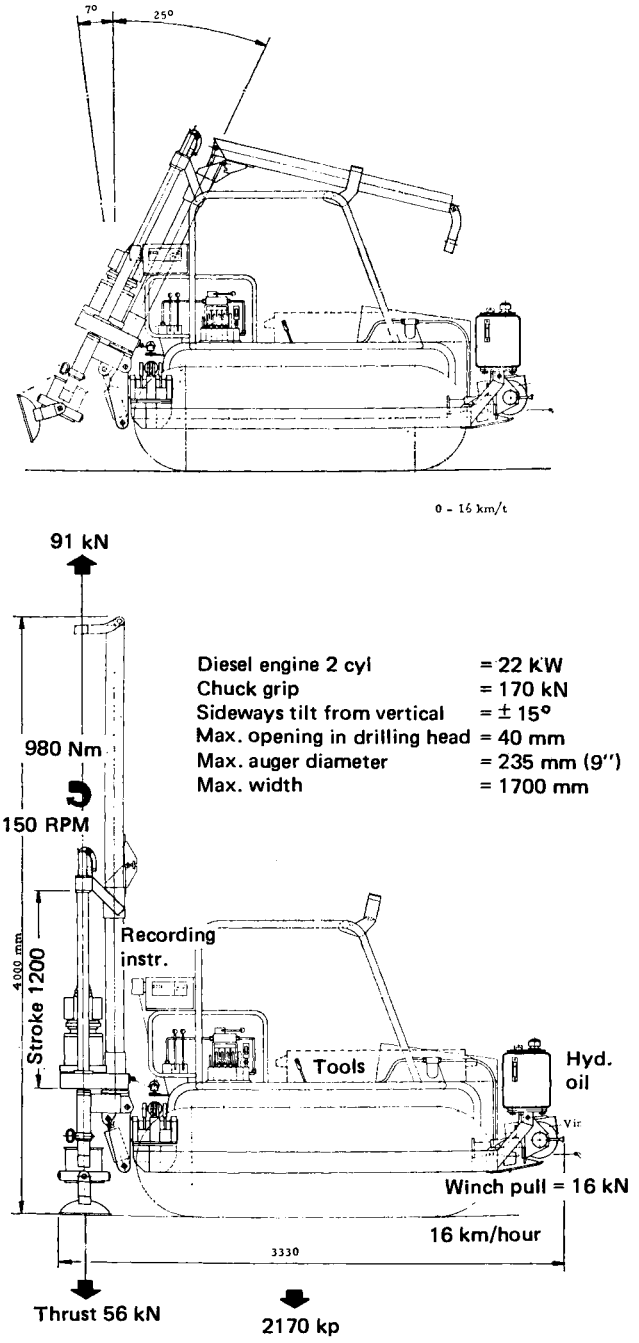


Fig. 3.2. Norwegian multipurpose drilling rig. (Andresen & Rygg, 1975).

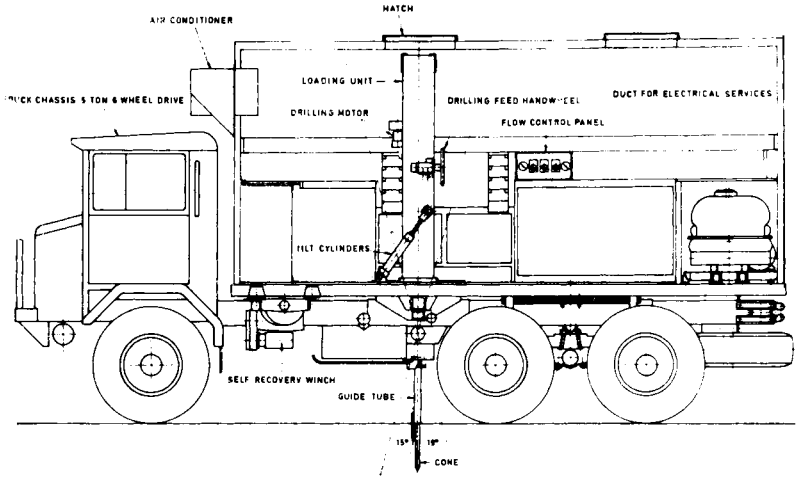


Fig. 3.3. CRB penetrometer testing vehicle (Holden, 1974a).

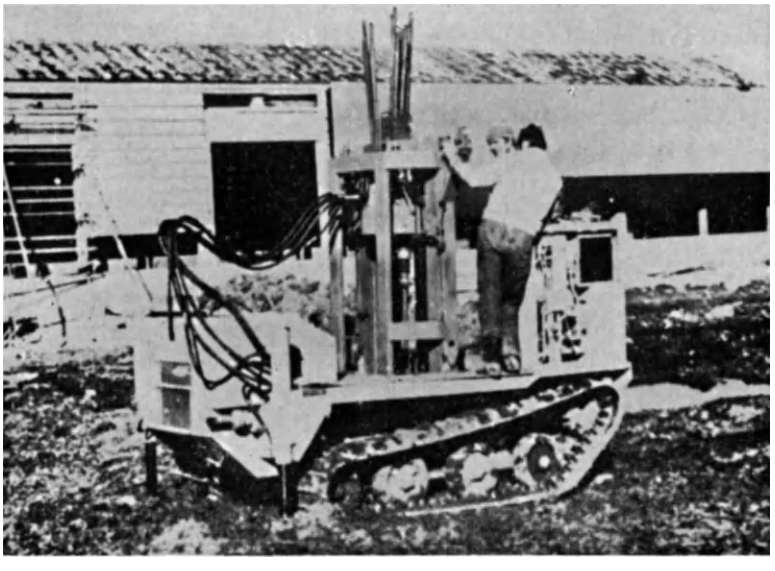


Fig. 3.4. Nilcon static sounding machine (Nilsson, 1975).

Hvorslev (1949) described *undisturbed samples* as follows:

> "Undisturbed samples may be defined broadly as samples in which the material has been subjected to so little disturbance that it is suitable for all laboratory tests and thereby for approximate determination of the strength, consolidation, and permeability characteristics and other physical properties of the material in-situ. The term is to some extent misleading since it is impossible to obtain a truly undisturbed sample, but it is firmly established in engineering terminology and has therefore been retained."

Disturbed samples were described by Hvorslev thus:

> "Representative samples contain all the mineral constituents of the strata from which they are taken and have not been contaminated by material from other strata or by chemical changes, but the soil structure is seriously disturbed and the water content may be changed. These samples are suitable for general classification tests and positive identification of the material, but they are not suitable for major laboratory tests and determination of the structural properties of the material in-situ."

3.2.2 Factors which affect sample disturbance

Fully undisturbed soil sampling is as yet not possible, but high quality samples obtained by a technique in which the disturbance is kept to the absolute minimum are termed *undisturbed*. The processes by which a sample can be disturbed during sampling and laboratory testing are (Hvorslev, 1949):

(1) changes in stress conditions,
(2) changes in water content and void ratio,
(3) disturbance of the soil structure,
(4) chemical changes, and
(5) mixing and segregation of the soil constituents.

These disturbances can be related to sampler design, sampling, handling and storage. Many authors have discussed the factors which influence sample quality, in particular Hvorslev (1949) and Kallstenius (1963).

It is considered by many that block samples are the best samples obtainable. Stationary thin-walled piston samplers are considerably superior to open samplers, for the reasons:

(1) Displaced excess (remoulded) soil cannot enter a piston sampler prior to and during sampling provided the correct handling procedure is adopted.
(2) Piston samplers can be used in cased and uncased boreholes as well as in displacement boring.
(3) A piston is preferable to a check valve in that it creates the retaining vacuum over the sample more effectively during withdrawal, and so helps to minimize sample loss.

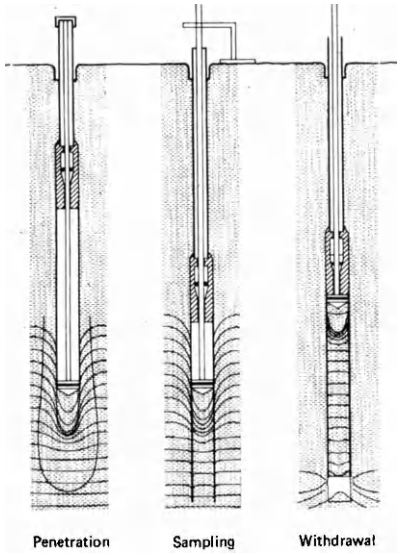


Fig. 3.5. Deformations of soil layers during sampling operations.

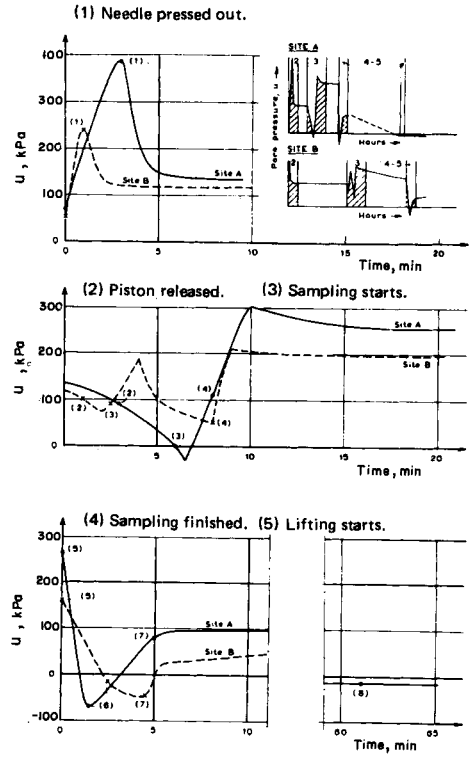


Fig. 3.6. Pore pressures measured 300 mm from top of 95 mm dia. sample during sampling operation (Schjetne, 1971).

The stress changes and deformations which occur beneath a piston sampler are illustrated in Fig. 3.5, and measurements of pore pressure changes during sampling are shown in Fig. 3.6 (Schjetne, 1971).

3.2.3 Sampler design

3.2.3.1 Wall thickness and edge taper

Deformation of soil during sampling causes disturbance of the soil sample. The volume displacement during sampling is expressed in terms of the *area ratio,* which is given by:

$$C_a = \frac{D_w^2 - D_e^2}{D_e^2} \tag{3.1}$$

where D_w and D_e are the external diameter and entrance diameter respectively of the sampling cylinder, as defined in Fig. 3.7. For soft clays, this ratio is kept to a minimum by employing thin-walled tubes. For composite samplers, however, the area ratio is considerably higher. In these cases, sample disturbance is reduced by tapering the outside of the cylinder very gradually from a sharp cutting edge (Hvorslev, 1949, recommended a maximum 10°), so that the full wall thickness is far removed from the point where the sample enters the tube. The relatively large displacements caused by the thick cylinder do not then affect the sample already inside. For practical reasons, the lower portion of the tapered end is generally given a secondary taper of 20 to 45° until a sufficient wall thickness has been achieved to prevent undue damage.

The effect of area ratio on sample disturbance is shown in Fig. 3.8, while Fig. 3.9 indicates the influence of the angle of the cutting edge. The combined requirements for area ratio and edge taper to cause low degrees of disturbance are summarized in Fig. 3.10 and Table 3.1 for samplers of approximately 75 mm diameter (International Society for Soil Mechanics and Foundation Engineering, ISSMFE, 1965).

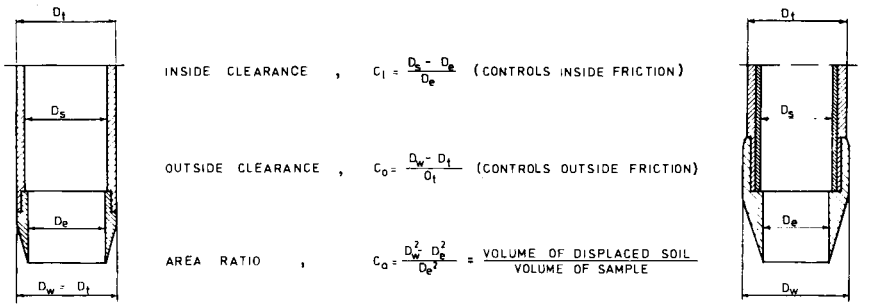


Fig. 3.7. Definitions of *inside clearance, outside clearance* and *area ratio* (Hvorslev, 1949).

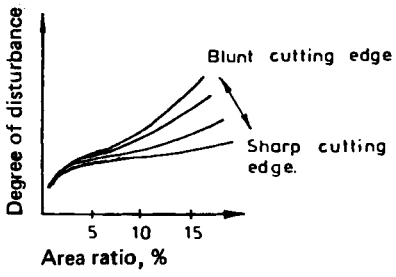


Fig. 3.8. Effect of area ratio on degree of disturbance.

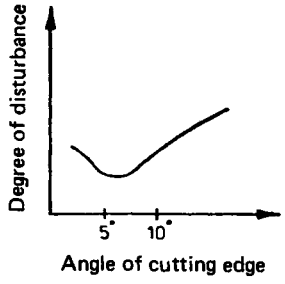


Fig. 3.9. Effect of angle of cutting edge on degree of disturbance.

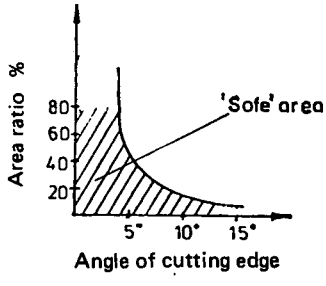


Fig. 3.10. Criterion for relationship between area ratio and angle of cutting edge to minimize disturbance.

Table 3.1. Combinations of area ratio and edge taper for undisturbed samplers

Area ratio, %	Edge taper angle, degrees
6	15
10	12
20	9
40	5
80	5

These combinations can only be taken as a guide, however, since the diameter of the sampler and the nature of the soil will have a major effect on the disturbance caused during sampling.

3.2.3.2 Inside and outside clearances

An inside clearance results when the inside diameter of the sampler steps up not far from the tapered end; an outside clearance exists when the outside diameter steps down not far from the end (Fig. 3.7) (Hvorslev, 1949).

The inside clearance is usually denoted by the *inside clearance ratio,* given by:

$$C_i = \frac{D_s - D_e}{D_e} \tag{3.2}$$

where D_s is the inside cylinder diameter and D_e is the cutting edge diameter. The desirable value of C_i depends on the soil type, sampling depth and sampler type. If the surfaces of the tube are clean and smooth, an inside clearance ratio of 0.5 to 1.0% is suggested for sampling to depths of 20 m in non-swelling clays; this is illustrated in Fig. 3.11. The inside diameter should never be smaller than that of the cutting edge (i.e. C_i must never be negative), and large clearances (>1 to 3%) cause deformations of samples, opening of fissures and swelling of soils which contain gases.

The *outside clearance ratio* is defined as:

$$C_o = \frac{D_w - D_t}{D_t} \tag{3.3}$$

where D_w is the maximum outside diameter of the sampler (or shoe) and D_t is the outside diameter of the sample tube. Although outside clearance increases the area ratio, a clearance of 2 to 3% can be advantageous in clay (Hvorslev, 1949).

3.2.3.3 Sampler dimensions

Sample disturbance decreases with increasing sample diameter down to a practical limit where the sample weight balances the inner forces from adhesion, friction and vacuum (Fig. 3.12). The relationship between sample diameter and sample quality varies with the properties of the clay, and it depends upon the stress history, age, sensitivity and shear strength. Diameters smaller than 50 mm should not be used for undisturbed sampling. For many clays, the best quality samples are obtained at a diameter of approximately 100 mm or more, the gain in quality for diameters from 50 mm to 100 mm usually being small relative to the additional cost and the increased danger of losing the sample in the ground. In Norway, for example, 97 mm piston samples are four times as expensive as 54 mm samples.

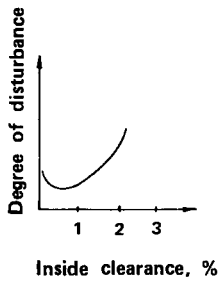


Fig. 3.11. Effect of inside clearance on degree of disturbance.

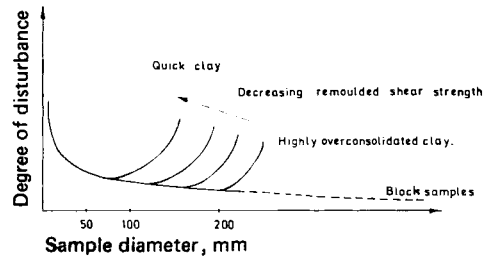


Fig. 3.12. Effect of sample diameter on degree of disturbance.

The possibility of losing a sample increases rapidly with increasing diameter because the weight of the sample increases in proportion to the square of the diameter, while its adhesion to the wall of the tube increases only in proportion to the diameter.

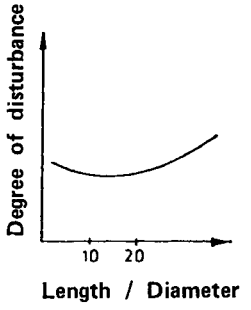


Fig. 3.13. Effect of length/diameter ratio of sample on degree of disturbance.

Sample quality is related to the length/diameter ratio of the sample in the manner shown in Fig. 3.13. The optimum length/diameter ratios suggested for clays of different sensitivities, S_t, are:

S_t	length/diameter
>30	20
5 to 30	12
<5	10

At each end of a tube sample, the soil over a length of approximately twice the sample diameter should not be used where undisturbed specimens are required. In soft soils, where the hole is not prebored and the sampler is pushed into the ground for continuous sampling, the soil within at least three diameters from the two ends should not be used for testing.

3.2.3.4 Friction between clay and sampler

According to Ball (1962), A & L Casagrande wrote in 1960:

"We can now conclude that all development work during the past twenty years has hinged on the assumption that it is chiefly the friction between the sample and the inside of the sampling tube which causes disturbance, and that if this friction were to be eliminated, satisfactory samples would be assured. Hence, almost all important improvements in the design of the sampler and in sampling technique, such as the angle of the cutting edge, the inside clearance, etc. were designed to eliminate this inside friction, and successfully so. But now we realize that in clay of low density, sample disturbance will result in spite of all these precautions because of the excessive friction which is built up along the outside of the sampling tube and which is transmitted in the form of excessive stresses into the clay below the cutting edge. In other words, the clay is already disturbed before it enters the sampling tube."

They continued by suggesting that a rotary coring type sampler might be the best device for undisturbed sampling.

Kallstenius (1963) carried out laboratory tests to measure the friction between soft clay and materials commonly used for sampling tubes. He varied the normal stress and used a rate of shear commonly used for sampling, which resulted in almost undrained testing conditions. His test arrangement is shown in Fig. 3.14, and the results of the test are given in Fig. 3.15. The apparent decrease in coefficient of friction with increase in normal stress was probably because of an increase in pore pressure and lubrication effects as water was squeezed from the clay.

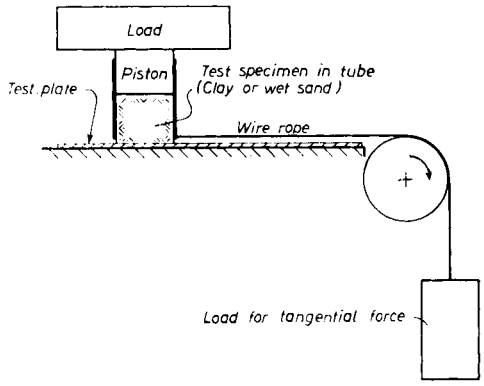


Fig. 3.14. Test arrangement used by Kallstenius (1963) to measure 'friction'.

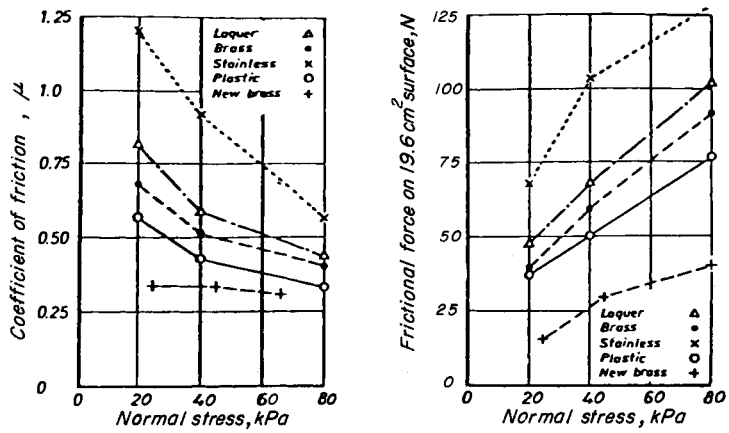


Fig. 3.15. Friction test results obtained by Kallstenius (1963) for clay against sampler materials.

In practice, clay adheres to the soil sampler and causes an increase in the shear strength of the clay directly against the sampler. This effect is not fully understood, and further research is needed to clarify the cause. It might be related to suction forces caused by local changes in pore pressure as dilatancy occurs during the sampling process. The strength increase certainly occurs too quickly to be caused by electrical potential differences.

Of the materials frequently employed for thin-walled sampling tubes, stainless steel is possibly the most common: it offers high frictional resistance to clay, however, as can be seen in Fig. 3.15. Laquered steel has been recommended by Hvorslev (1949), but the laquer has a tendency to peel off, especially at sharp edges. Brass is frequently used for sampling tubes, but it has the disadvantages that it reacts chemically with the clay and thereby disturbs the soil during storage. Plastic (glass-fibre reinforced polyester) can be made strong and sufficiently impermeable with respect to water to meet practical demands, and it has a high resistance to corrosion, but this material can only be used in composite samplers to form the inner tube which contains the sample (see Section 3.2.6.2).

To minimize sample disturbance caused by sticking of the clay to the outside of the sampler, Kallstenius (1963) suggested the following:

(1) Carry out sampling in prebored holes.
(2) Reduce the outside friction by using a lubricated sampler made of hard, smooth material.
(3) Advance the sampler smoothly during the sampling process without any interruption.

Even where piston samplers are used, it is common practice in some countries to sample in the bottom of prebored holes filled with drilling mud which are cleaned prior to each sampling operation (see Japanese Society of Soil Mechanics and Foundation Engineering, 1963). In Scandinavia, this procedure is not commonly used since nearly all sampling is performed as displacement borings, mainly because of the high sensitivities of the soft clays.

In Norway, a reamer (friction breaker) above the sampler has been found to be useful. Begemann (1966, 1971) described a continuous sampler with a special cutting edge and a saw-toothed outside surface, the purpose of which was to prevent jerks and shocks by the friction built up during penetration in fine sand. Begemann's sampler was also reported as very suitable for soft clay, which indicates that a considerable reduction in outside friction is caused by the remoulding of the clay around the sampler (see Section 3.2.4 and Fig. 3.33 below).

3.2.3.5 Prevention of sample loss

In order for the sample to be retained in the sampler as it is withdrawn to the ground surface, the buoyant force of the air or water on the sample

together with the frictional force between the sample and the inside of the sampler must be greater than the weight of the sample plus the tensile force required to separate the sample from the soil below. A vacuum might be created above the sample during withdrawal which assists in retaining the sample in the tube. A vacuum can also form at the bottom of the sample as withdrawal commences, however, and this greatly increases the force required to separate the sample from the attached soil at the base. To prevent this, and to increase the pressure below the sample, the sampler shoe can be provided with an air or water hose. Such accessories, however, render sampling expensive and complicated.

Several kinds of core retainer have been developed, but at present no perfect solution has been devised that does not cause either direct or indirect disturbance to the sample or which does not appreciably complicate the sampling procedure. Sample recovery is dealt with further in Section 3.2.6.4.

3.2.4 Sampling procedure

The sampling procedure is as important as sampler design as far as sample disturbance is concerned. The following paragraphs apply mainly to the use of piston samplers.

When the sampler is brought to the desired sampling depth, it is of great importance that the piston extension rods are properly clamped to a fixed point so that no downward movement of the piston occurs during sampling, although some upward motion can be tolerated when the piston clamp mechanism is released.

The sampling (penetration) speed must be constant and must not be too high or too low. In clay, the friction increases if sampling is interrupted for more than a few seconds. If the penetration speed is reduced below 0.1 m per minute, adhesive forces build up which can cause severe disturbance, as shown in Fig. 3.16.

Overdriving, i.e. a penetration in excess of the stroke length of the piston, should be avoided when taking undisturbed samples.

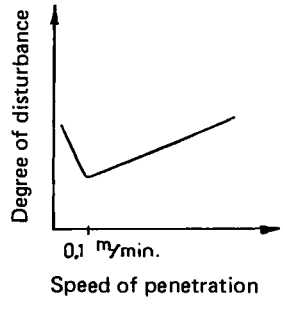


Fig. 3.16. Effect of sampler penetration rate on degree of disturbance.

To prevent loss of the sample, the sampler should remain stationary for a few minutes after sampling and before withdrawal to allow for the dissipation of excess pore pressure. Before starting the withdrawal of the sampler, it should be rotated two or three full revolutions (Hvorslev, 1949) if the sampler is not provided with a cutting mechanism. This rotation might not be necessary with small diameter samplers with high length/diameter ratios.

When the sampler is brought to the surface and the sample tube is to be separated from the sample head, it is important that no vacuum is created between the piston and the sample which will apply a tensile force to the top of the sample. Special mechanical devices can prevent this from occurring, viz:

(1) A vacuum-release screw can be fitted onto the piston.
(2) In composite samplers, the inner tube can be divided into several liners, and the one with the piston in it can be separated from the rest of the sample.
(3) An airtight connection between the sample head and the tube can be located below the top position of the piston.

3.2.5 Open thin-walled samplers

An open thin-walled cylindrical tube with a cutting edge represents the most commonly used device for obtaining undisturbed samples of soft clay in many countries, including the USA and Canada. This *Shelby tube* or *Chicago sampler* is attached by screws to a sampler-head (Fig. 3.17) which is pushed into the undisturbed clay at the bottom of a borehole by the hydraulic force of a drilling rig or by a 'pull-down' method. The sampler-head features vent holes and a ball check valve to permit pressure release in the tube during sampling, to prevent water from re-entering, and to create the retaining vacuum over the sample during withdrawal. The investigations into sample disturbance described by Hvorslev (1949) were mostly related to open samplers.

The quality of samples obtained with open samplers depends to a great extent on careful cleaning of the borehole before sampling. A special clean-out tool with a horizontal or up-turned jet (Fig. 3.18) is often used for this purpose. The sampler dimensions are also of importance, as indicated in Section 3.2.3.3. Common 'standard' dimensions are shown in Table 3.2.

Table 3.2. Common thin-walled sampler dimensions

Diameter, mm	Length, m
50.8 OD × 47.7 ID	0.698 – 0.85 – 1.307
53.9 × 50.8	,,
76.2 × 72.0	,,
127.0 × 105.9	,,

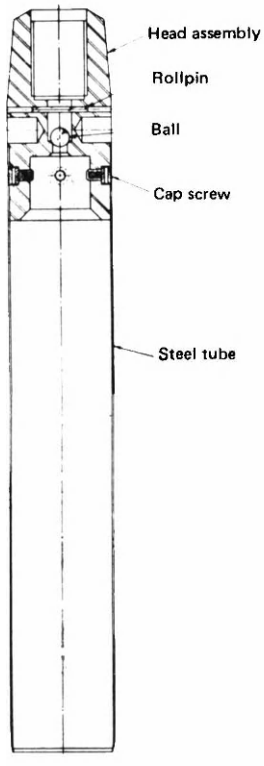


Fig. 3.17. Thin-walled tube sampler attached to an Acker sampling head.

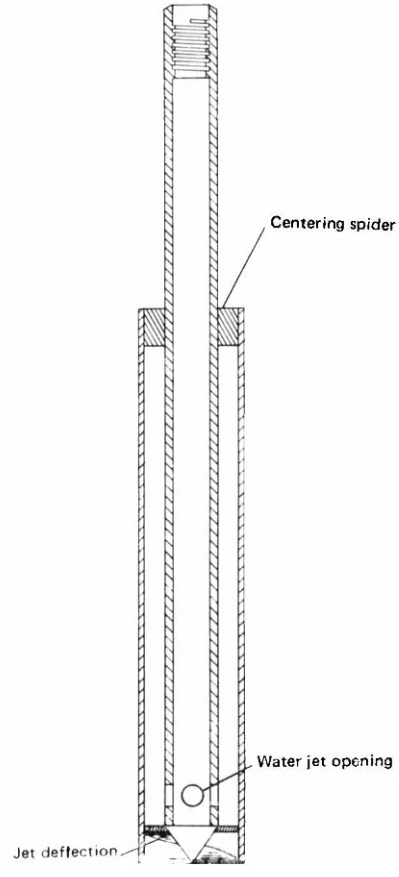


Fig. 3.18. Acker borehole clean-out tool.

3.2.6 Piston samplers

Piston samplers provide a means of obtaining higher quality undisturbed samples of soft clay than is possible with open-tube samplers. They also enable sampling to be carried out without the necessity to drill a hole.

3.2.6.1 Development of piston samplers in Norway

The development of piston samplers and sampling techniques for soft clays has been dictated by many factors, and a brief review of the samplers developed in Norway will help to illustrate these factors. Figures 3.19 to 3.23 show the various mechanical piston samplers used in Norway from 1925 to the present. All of these are of the stationary piston type, which is considered to be the only satisfactory means of taking undisturbed samples of soft clay. The 40 mm sampler was originally designed by Sweden's John Olsson (Fig. 3.19) and was modified by the Norwegian State Railways to the form shown in Fig. 3.20.

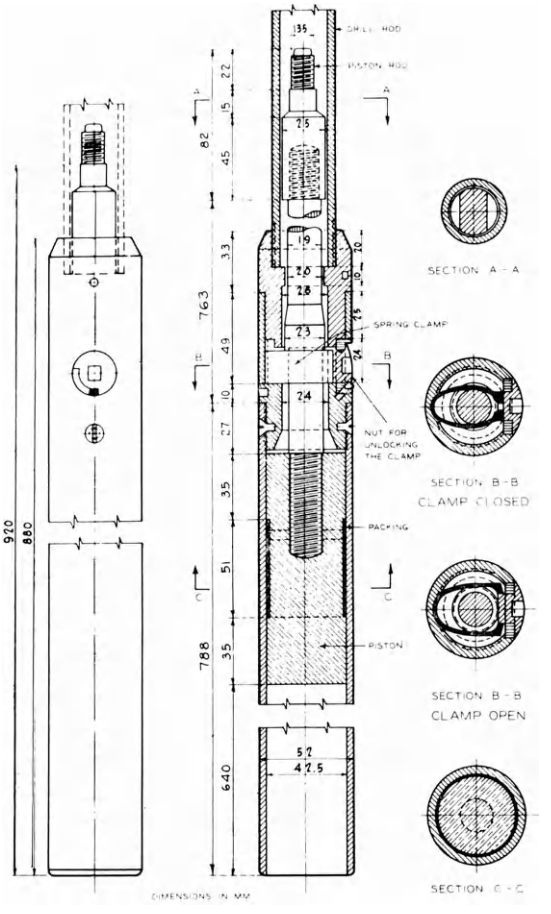


Fig. 3.19. Olsson piston sampler (Hvorslev, 1949).

The 54, 97 & 200 mm samplers shown in Figs. 3.21 to 3.23 were designed by the Norwegian Geotechnical Institute (NGI). The designs of all these piston samplers have been dictated by the requirement of sample quality and the need for simple and economical sampling methods.

Table 3.3 shows the dimensions of the four Norwegian piston samplers referred to above together with those of the Norwegian composite sampler described in Section 3.2.6.2 and illustrated in Figs. 3.26 & 3.27 below. The design of the 40 mm sampler (Figs. 3.19 & 3.20) necessitated the extrusion of the sample in the field, only selected portions being transferred to shorter cylinders for laboratory testing. When designing the 54 mm sampler (Fig. 3.21) in 1952, NGI decided that it was necessary to transport the whole sample tube to the laboratory, a less complicated procedure which reduced the necessary geotechnical 'qualifications' of the drilling foreman.

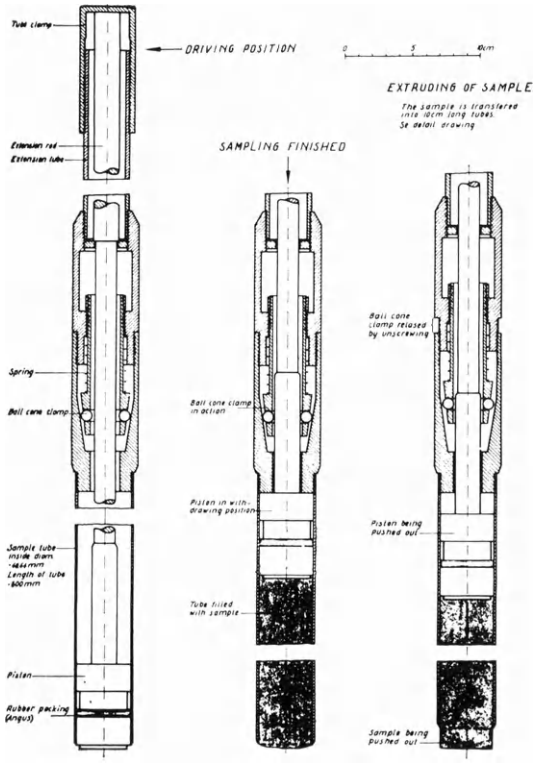
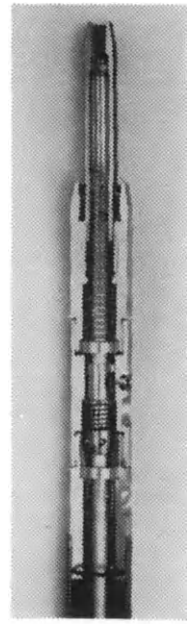


Fig. 3.20. NSR 40 mm piston sampler.



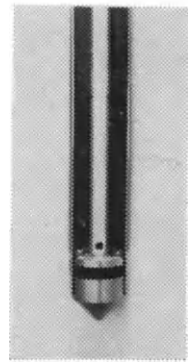


Fig. 3.21. NGI 54 mm piston sampler.

Table 3.3. Dimensions of piston samplers used in Norway

	40 mm sampler (Figs. 3.19 & 3.20)	54 mm sampler (thin-walled) (Fig. 3.21)	54 mm sampler (composite) (Figs. 3.26 & 27)	95 mm sampler (Fig. 3.22)	200 mm sampler (Fig. 3.23)
Inside diameter, mm	40.4	54	54	96.3	200
Outside diameter, mm	44.4	57	64	101.6	220
Sample length, m	0.6	0.4–1.0	0.4–0.8	0.75–1.0	1.0
Area ratio, C_a, %	16	12	44	11	26
Inside clearance, C_i, %	0	0.9	0.6	0.8 –1.3	2
Outside clearance, C_o, %	0	0	2	0	4

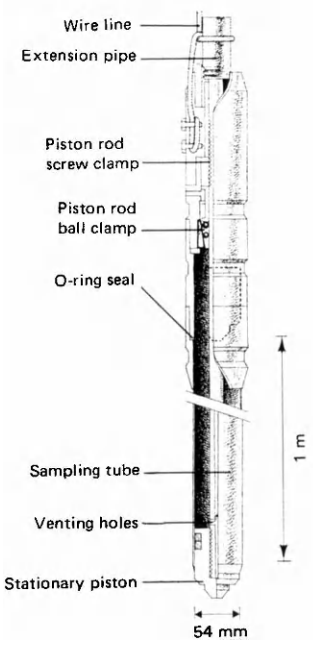


Fig. 3.22. NGI 97 mm piston sampler.

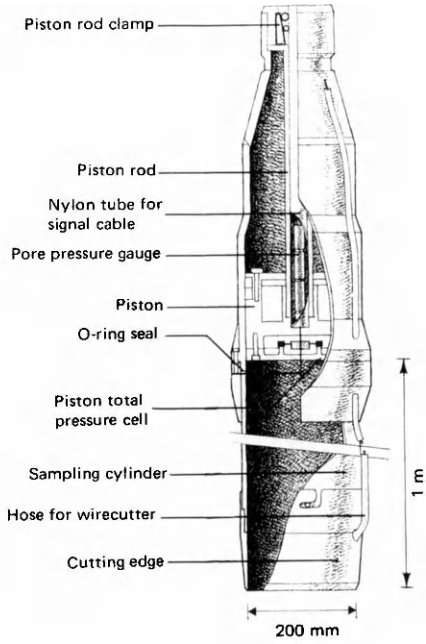


Fig. 3.23. NGI 200 mm research piston sampler, which includes a pore pressure gauge and a total stress cell.

3.2.6.2 Composite piston samplers

Soft clay deposits are sometimes present below layers of stiffer clay or granular material. In order to obtain satisfactory undisturbed samples of soft clay under these circumstances, a composite sampler must be used.

In Sweden, the Swedish Standard I & II composite piston samplers are in common use for the extraction of 50 mm diameter undisturbed samples. A plastic tube receives the sample cut by the outer steel sampling tube. These two samples have been tested in Norway and have been found to give samples of the same quality as those taken with the NGI 54 mm piston sampler. The Standard I sampler (Fig. 3.24) can be used with an inner rod, with wire or with a steel band, whereas the Borros Standard II sampler (Fig. 3.25) has a single-rod extension system. In Norway, the use of inner rods is preferred to other mechanical means because these give better control over the piston. The demands for higher production rates have been met by the use of quick 'snap on' rod connections instead of threads, and by the use of towers which make it possible to handle 10 to 20 m long pipes and tools in one operation (Friis, 1961).

At an early stage, the NGI 54 mm thin-walled sampler (Fig. 3.21) was modified into a composite sampler with steel liners (Fig. 3.26). Mild steel Shelby tubes, commonly used as sample cylinders, corrode, however, and the cutting edge is often damaged. Even stainless steel cylinders seem to cause

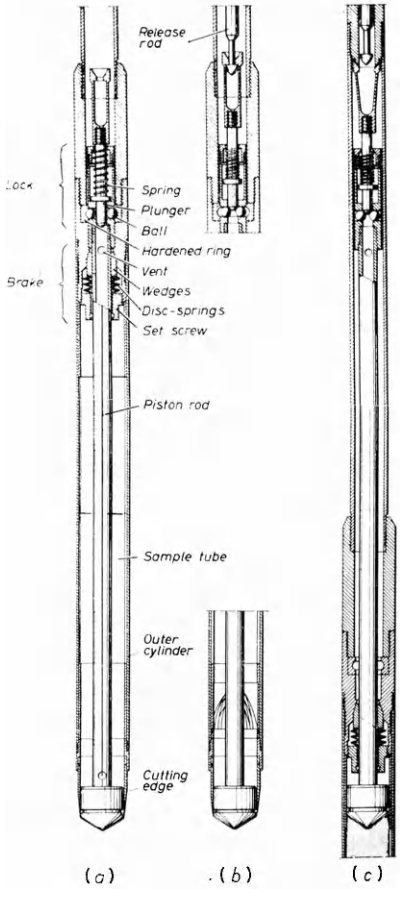


Fig. 3.24. Swedish Standard I 50 mm piston sampler (Kallstenius, 1963): (a) before punching, (b) after releasing the lock (alternative with one catcher), (c) when sampling is finished and inner rod released.

Fig. 3.25. Swedish Standard II 50 mm piston sampler (Borros AB). (Single rod extension; sampling by rotating the rod; reaction taken by fins on outer cylinder).

some deterioration in sample quality when they are stored for long periods. Plastic (fibre-glass) liners, as used in the Swedish Standards I & II samplers, appear to be superior to steel cylinders, and these are being incorporated into a new 54 mm composite sampler (Fig. 3.27) presently being developed by NGI in cooperation with the Norwegian Roads Authority. Apart from the corrosion aspect, a new design of sampler was necessary because of other practical difficulties with the thin-walled steel cylinders; unacceptably poor production tolerances during manufacture, and damage during use, cause cross-sectional variations along the cylinders which knead and remould the samples. Inside clearances sufficient to compensate for these variations will only cause the samples to swell.

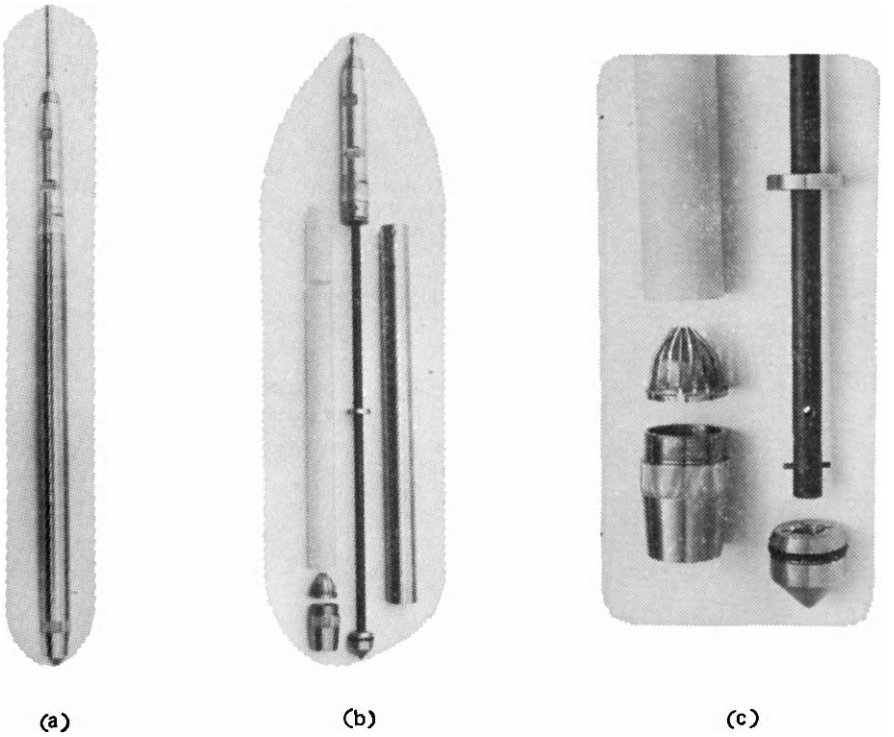


Fig. 3.26. NGI 54 mm composite piston sampler: (a) assembled, (b) disassembled, (c) details of cutter, retainer and piston.

Composite samplers are a little more laborious to use than 'single tube' samplers. Where plastic liners are used, a high frequency of sample loss has been reported because of the low adhesion between the clay and the plastic. The use of liners makes it possible to include confining rings and membranes for direct laboratory application (Pyke, 1977).

3.2.6.3 Hydraulic and gas samplers

A hydraulically operated piston sampler developed by Osterberg (1973) is shown in Fig. 3.28, and a gas operated sampler for offshore use (Andresen et al, 1965) is shown in Fig. 3.29. These samplers have the advantage that the sampling tube itself is protected and clean until sampling commences; the Borros Swedish Standard II sampler referred to in Section 3.2.6.2 has the same advantage because of its single-rod extension.

3.2.6.4 Sample recovery

For thin-walled piston samplers, the introduction of core retainers is difficult. In composite samplers, space is available, but the core retainers

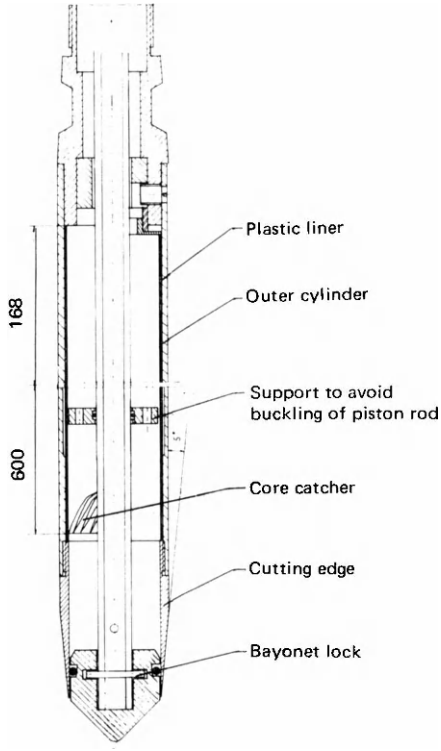
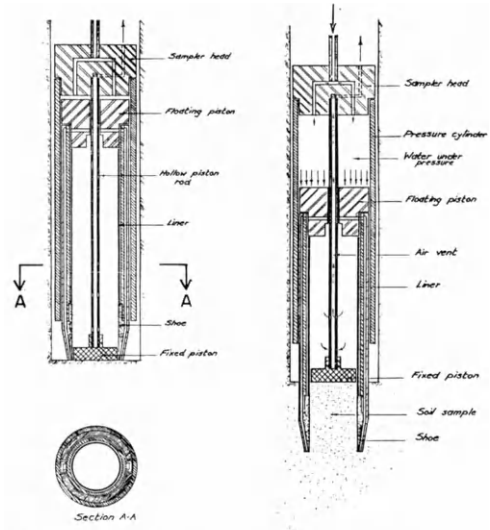


Fig. 3.27. Lower portion of the new NGI 54 mm composite sampler.

cause disturbance of the sample. The core retainers developed by Begemann (1966, 1971) seem to be the best.

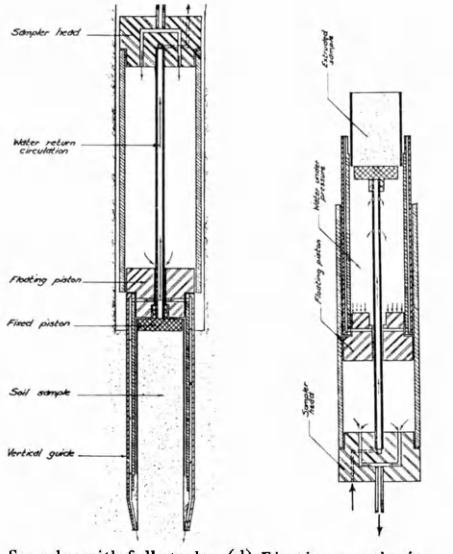
There is a decrease in recovery ratio for increases in sample diameter and sample length. The NGI 97 mm sampler was designed for 1.5 m long samples, but because of poor sample recovery, the length had to be reduced to 1 m. With the NGI 200 mm sampler, a wire cutter of the type shown in Fig. 3.30 (Hvorslev, 1949) together with air injection is necessary. A similar system of a pipe with a cutting-plate has been used with the NGI 97 mm sampler under especially difficult conditions.

The drilling foreman must take special care and precautions when there is a danger of sample loss. These include the cleaning of the borehole, the use of mud, special cleaning of the inside of the sampling tube, and the tightening of packings. One main reason for the loss of samples, or a reduced recovery ratio, is that the water or mud collected above the piston during the lowering and penetration of the sampler has difficulty in escaping upwards during the actual sampling operation. The samplers designed at NGI have holes in the piston rod and through the locking mechanisms of the sampler head. An



(a) Sampler in drill hole (not to scale).

(b) Sampling.



(c) Sampler with full stroke, pressure release, and vacuum breaker.

(d) Ejecting sample in liner from sampler.

Fig. 3.28. Osterberg hydraulic piston sampler (Osterberg, 1973).

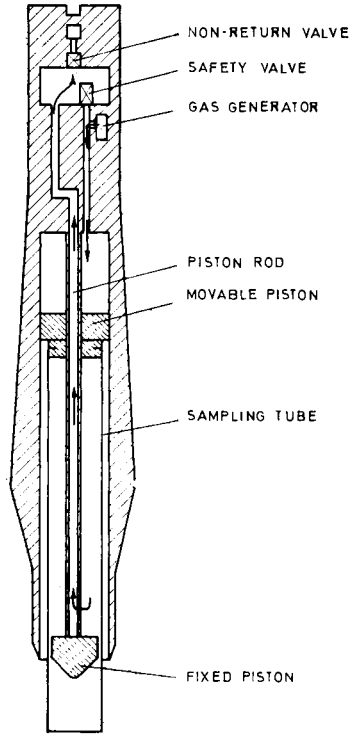


Fig. 3.29. NGI gas operated 54 mm piston sampler.

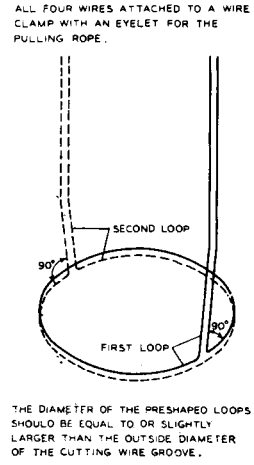


Fig. 3.30. Principle of sampler cutting wires (Hvorslev, 1949)

important part of the preparation before sampling is to check that the water escape passages above the piston are clean.

3.2.6.5 Continuous sampling

It is sometimes necessary or desirable to obtain a continuous sample of a deposit of soft clay; this is particularly so when detection and recovery of thin sand or silt seams is important. The principle of the Swedish foil sampler (Kjellman et al, 1950), designed for continuous sampling, is shown in Fig. 3.31, while a cross-section of the sampler is given in Fig. 3.32. The continuous sample is enclosed in aluminium foil as the sampler is advanced, thus eliminating friction between the sample and the inside of the tube.

The Delft continuous sampler (Fig. 3.33), designed by Begemann (1971), is similar in principle to the Swedish foil sampler, except that a long nylon stocking encloses the continuous sample. It is claimed that this sampler gives better samples at a lower cost than the Swedish sampler.

It is possible to obtain a continuous soil profile without using a continuous sampler and often at a lower cost by taking overlapping piston samples in two closely spaced, parallel boreholes.

3.2.7 Block sampling

Sample quality usually increases as the diameter increases, but there are practical and economic limitations to the size that can be obtained using tube samplers. Where particularly large samples are required for laboratory testing, block samples are obtained. These large samples, which can usually only be obtained from shallow depths, are of excellent quality if carefully extracted from the ground and transported to the laboratory.

For block sampling, the Gilboy method (Hvorslev, 1949; Bertram, 1953) is still in common use (Fig. 3.34). After a column of soil has been trimmed to the desired size, a box without top or bottom is placed over the sample, and the space between the box and the sample is filled with paraffin wax or with a paraffin/Vaseline mixture. After the top of the sample has been trimmed, more wax is added before the top is attached. The sample is cut free from the soil below, turned over, waxed and the bottom plate attached.

For higher quality block samples, a box or cylinder with cutting edges is pressed into the soil under controlled conditions. A reaction frame must then be constructed together with a guide mechanism for the jacking of the sampler. Special care has to be taken to avoid deformation of the box during and after sampling and to avoid changes in water content before laboratory testing.

In 1976, unusual care was taken by NGI in obtaining a series of large block samples of Drammen Clay in order to supply samples for use in the centrifuge at the University of Manchester and in a model test bin at the Delft Soil Mechanics Laboratory. Four cylindrical samples, 945 mm diameter × 517 mm long, and nine square samples, 1,250 × 1,250 × 750 mm long were cut

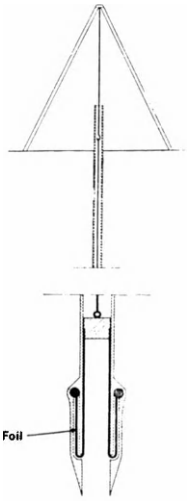


Fig. 3.31. Principle of Swedish foil sampler (Kjellman et al, 1950).

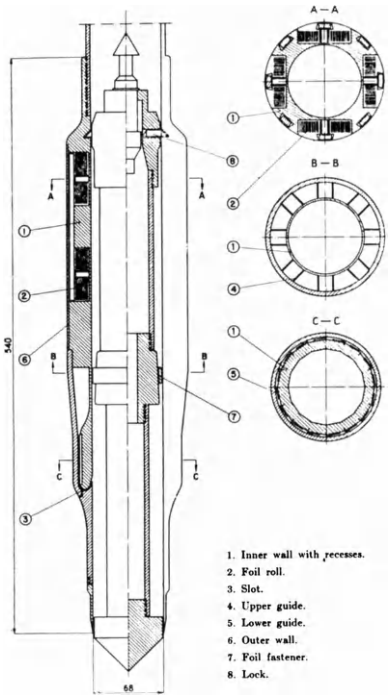


Fig. 3.32. Details of Swedish foil sampler (Kjellman et al, 1950).

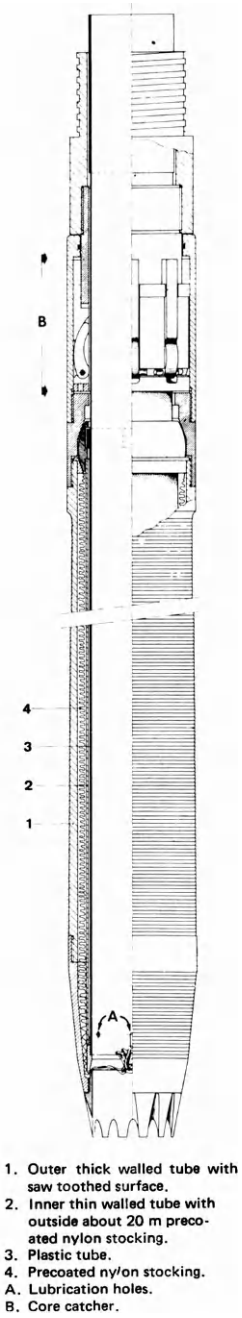


Fig. 3.33. Dutch continuous piston sampler (Begemann, 1971).

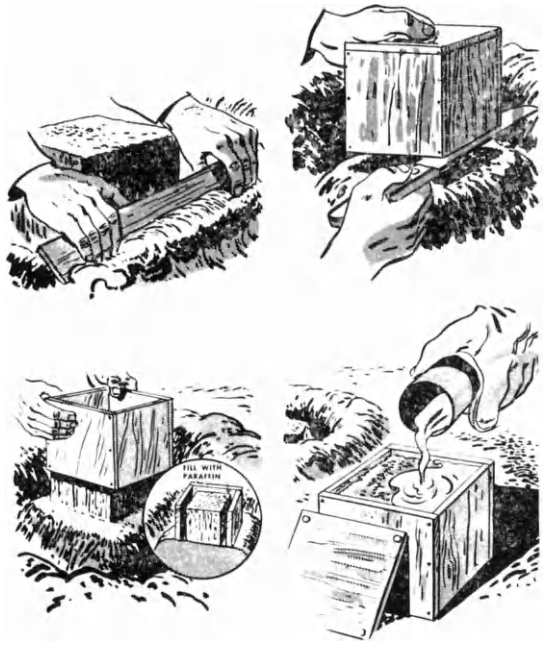


Fig. 3.34. The Gilboy block sampling method (Hvorslev, 1949).

at a depth of about 3 m using the arrangement shown diagrammatically in Fig. 3.35. Photographs of the sampling operation are given in Fig. 3.36. Because of the large dimensions of the samples, the cylindrical shape was better, since it ensured minimum deformation of the sample container; it was

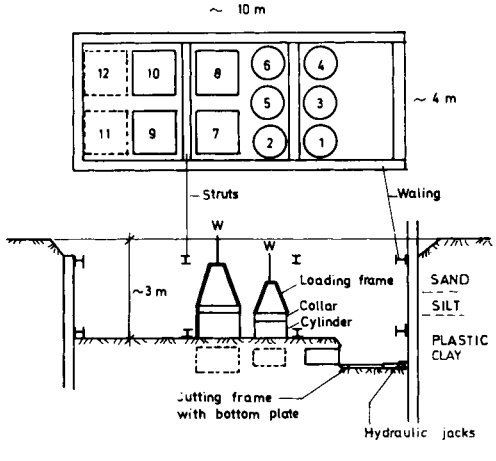


Fig. 3.35. Method employed to obtain large block samples of Drammen clay.

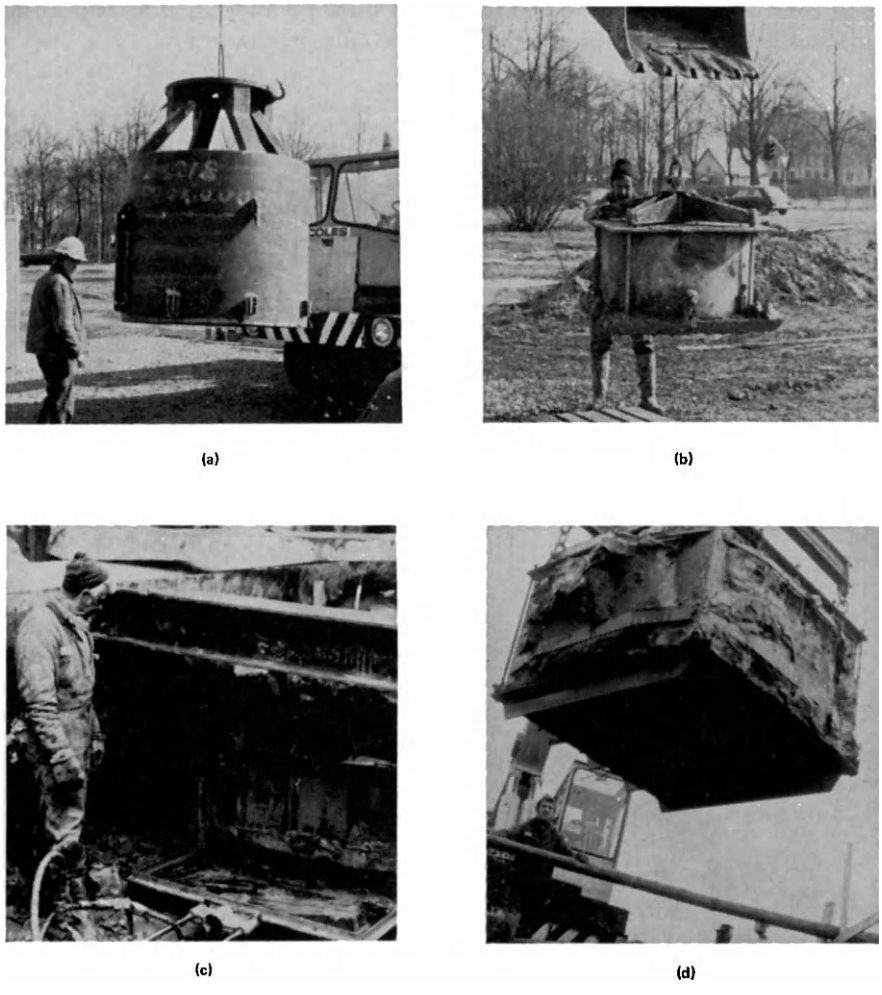


Fig. 3.36. NGI block sampling techniques employed in Drammen: (a) sample cutting cylinder with loading frame, (b) finished cylindrical sample, (c) cutting below box sample, (d) finished box sample.

also easier to press the bottom plate into position. The square boxes required external stiffeners.

3.2.8 Treatment of undisturbed samples

During sampling, primary disturbances of soil samples are caused by the sampling tool. These depend on the type of soil, the shape and quality of the sampler and the manner in which it is handled. After sampling, secondary disturbance effects occur, which might be time-dependent. These disturbances

result from poor sample sealing, vibrations during handling and transportation, and deterioration under unsatisfactory storage conditions.

3.2.8.1 Sealing

When a sample tube is disconnected from the sampler, proper sealing of the tube is essential to avoid loss of water and to reduce disturbance effects during storage. Paraffin wax is relatively cheap and readily available, and it is the most widely used material for the sealing of soil samples. It does have several undesirable properties, however, viz:

(1) it shrinks considerably when it sets,
(2) it becomes brittle and may crack when cold, and
(3) it is difficult and time-consuming to use at high summer temperatures, when it is soft and plastic.

Mechanical seals are usually more reliable and practical than wax. The one generally used in Norway is shown in Fig. 3.37. At the top, a rubber piston is inserted into the tube to hold the sample firmly; at the bottom, a steel plate supports the weight of the sample. The rubber piston is unnecessary when the tube is completely full. Rubber caps seal off each end of the cylinder, the cap colour indicating the top and bottom ends. Another satisfactory sealing device, developed by Holden (1971), consists of two plastic discs with an O-ring between them. It is a simple device, but it is difficult to use for sealing tubes which have been deformed during sampling.

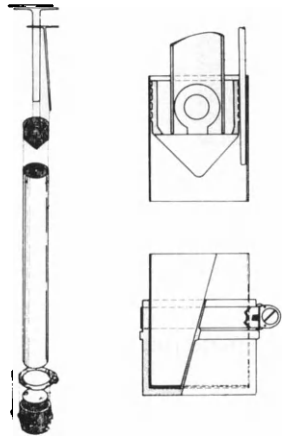


Fig. 3.37. Rubber piston and caps for sealing sample tubes in Norway.

3.2.8.2 Handling and transportation

Vibrations and shocks to undisturbed samples should be avoided as far as possible, although it is generally impossible to avoid some vibration during

transportation. Samples should be transported and stored horizontally in completely filled and properly sealed cylinders.

Kallstenius (1963) carried out a study of the effects of shock and vibration on undisturbed samples of soft clay. A sample which is allowed to fall one metre onto an asphalt floor is subjected to a retardation force of about 500 times that of gravity. Such a force alters the microstructure of a clay and causes a reduction in strength.

3.2.8.3 Storage

Even with the best storage provisions, soil samples are subject to deterioration with time. The redistribution of water, bacterial and chemical activity, and swelling and drying are all time-dependent processes. In order to reduce these to a minimum, the following precautions are necessary:

(1) The storage temperature should be kept in the same range as that of the soil from which the samples were taken.

(2) The humidity should be kept close to 100%.

(3) When the temperature and humidity in storage reach levels which promote fungal growth, this should be retarded by means of ultraviolet light.

Whatever precautions are taken, however, loss of water and chemical and physical changes of the soil will probably take place during protracted storage. It is advisable, therefore, for samples to be tested as soon as possible after they are obtained. When long periods of storage are necessary, control tests should be carried out to monitor any deterioration in samples (Schjetne, 1971).

It is sometimes not possible to place samples under proper storage conditions until some time after they have been obtained. Exposure for long periods to high temperatures can cause drying of the samples; it also promotes the liberation of gases, bacterial growth and corrosion. Freezing can also seriously disturb clay samples. The complete destruction of samples by freezing can be detected by visual inspection, but partial disturbances might not be so easily discovered. The structure of the frozen soil is sometimes lumpy or granular in appearance, which suggests that weak parts of the sample were first affected by the freezing.

Swelling of an undisturbed clay sample can be caused by the redistribution of water or expansion of gases in the pores. These both result from the negative pore pressures which appear in the sample when the external stresses are removed during sampling. One day after sampling, the difference in water content between the centre of a sample and its periphery is much greater than can be explained solely by loss due to surface drying. It suggests that an internal redistribution of water takes place and the mechanically undisturbed core swells because of the availability of free water in the remoulded material along the walls of the sample tube. Since the swelling is a time-dependent

process, it can be reduced by removing the disturbed portions as soon as possible.

3.2.9 Representative sampling

Representative (or disturbed) samples are used for logging purposes and for certain identification and laboratory tests. Any of the methods described previously can be used to obtain representative samples, of course, but more economical methods are available. There is no clear dividing line between undisturbed and disturbed samples, and there are differing opinions on the 'type' of sample produced by a number of well-known samplers. The Delft 29 mm continuous sampler (Fig. 3.38) described by Begemann (1966) is an example of this, but it is probable that the samples obtained with this device are closer to representative than undisturbed, mainly because of their small diameter. This Delft sampler is a very practical tool which is especially suitable for obtaining continuous samples in stratified soft and loose soils. It is generally able routinely to take 6 to 8 m long samples, but it should be capable of taking continuous samples of 20 to 25 m. Its use is, unfortunately, restricted by patents.

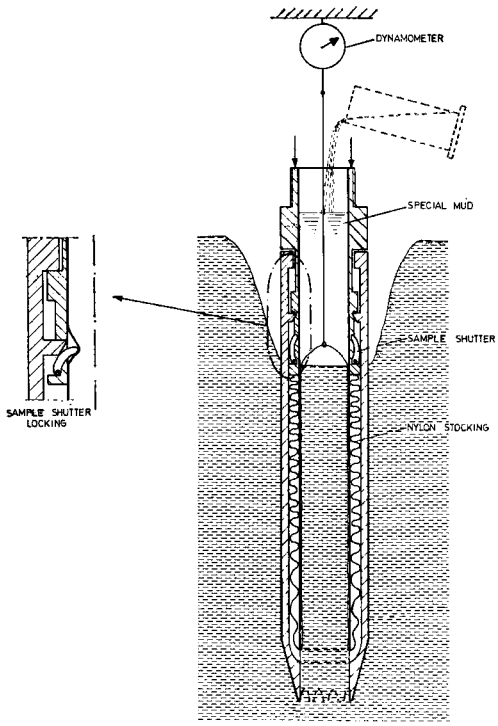


Fig. 3.38. Principle of Delft continuous 29 mm sampler.

Continuous-flight augers are most commonly used for continuous disturbed sampling. They are drilled into the ground continuously and withdrawn once the auger is full (Fig. 3.39). A special auger system developed in Sweden (Sökjer, 1961) combines sampling with in-situ shear strength measurement (Fig. 3.40). In Norway, an auger originally designed for cast-in-place concrete piles is now in service for soil sampling (Andresen, 1967). This auger, which has a hollow stem housing a relief valve (Fig. 3.41), has been used successfully to a depth of 22 m without hole collapse in soft clays with shear strengths of 30 to 40 kN/m^2.

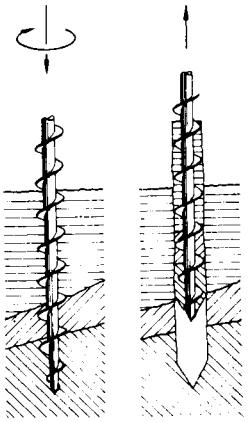


Fig. 3.39. Boring by continuous-flight auger.

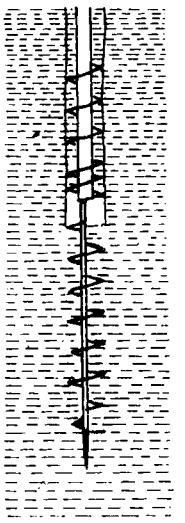


Fig. 3.40. Combined auger and shear strength device (Sökjer, 1961).

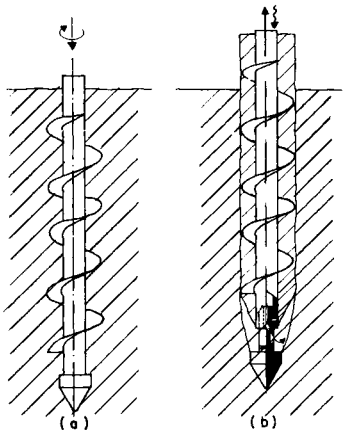


Fig. 3.41. Hollow stem auger with relief valve.

The Dutch post-hole auger (Fig. 3.42), which is operated by hand, is an efficient and economical way of obtaining disturbed samples up to several metres below the ground surface. Also manually operated is the *Geolog* probe (Fig. 3.43), used to obtain 16 mm samples down to 1.5 m, generally for agricultural purposes. The Anderson sampler (Fig. 3.44) is a form of simple piston sampler, with which a hole is formed by pushing the piston and sampler tube together into the ground before a single sample is taken; the hole can be advanced by continuous sampling if so desired.

3.3 VANE SHEAR TESTING

In the opinion of the Author, the vane test is the most suitable method for the determination of the undrained shear strength of soft clays. This test was first established as early as 1928–29 simultaneously in Sweden and Germany, and since its revival and further development in 1945 to 1948 in both Sweden and Britain, it has been used extensively in soft clays throughout the world.

3.3.1 Principle of the vane test

The vane itself normally consists of four thin rectangular metal plates attached at right angles to a central torque shaft to form a cross (Fig. 3.45). The vane is pushed vertically into the soil, and an increasing torque is applied to the shaft until failure of the soil occurs. The shear strength, s_u, of the clay is calculated from the torque, T, required to shear the clay over the cylindrical surface formed by the rotation of the vane.

On the assumption that the shear strength mobilized over the cylindrical surface is everywhere the same, Cadling & Odenstad (1950) derived the equation:

$$s_u = \frac{2T}{\pi D^2 \left(H + \frac{D}{3}\right)} \tag{3.4}$$

where H and D are the height and diameter of the vane. Flaate (1966a), however, assumed that the strength on the ends of the cylinder was mobilized in proportion to the rotation strain, from zero at the centre to s_u at the circumference, and this results in:

$$s_u = \frac{2T}{\pi D^2 \left(H + \frac{D}{4}\right)} \tag{3.5}$$

Fig. 3.42. Dutch post-hole auger *(Eijkelkamp B.V.)*.

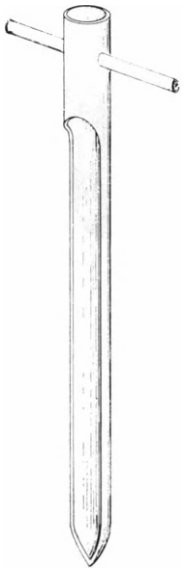


Fig. 3.43. The *Geolog* probe.

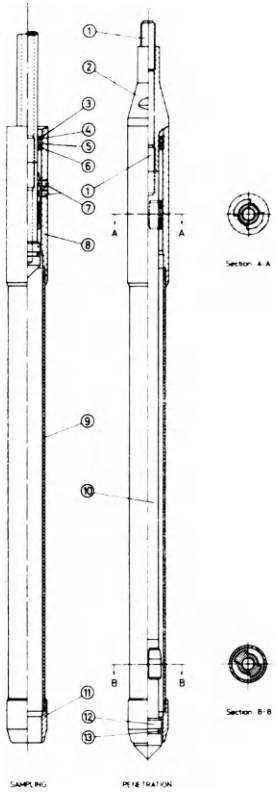


Fig. 3.44. Anderson single rod sampler *(Geonor A/S)*. (Penetration takes place with piston in lower position; piston is then retracted and locked in upper position before sampling).

For the 'standard' vane, $H=2D$, and Eqs. 3.4 & 3.5 become:

$$s_u = \frac{6}{7} \cdot \frac{T}{\pi D^3} \tag{3.6}$$

and

$$s_u = \frac{8}{9} \cdot \frac{T}{\pi D^3} \tag{3.7}$$

and the computed values of s_u will differ by only about 5%.

Vane tests may be carried out at the bottom of prebored holes *(borehole vane)* or by using a *vane borer* (Fig. 3.46) which penetrates into the ground to the required depth before a test is carried out. Vane borers are now more commonly used than borehole vanes.

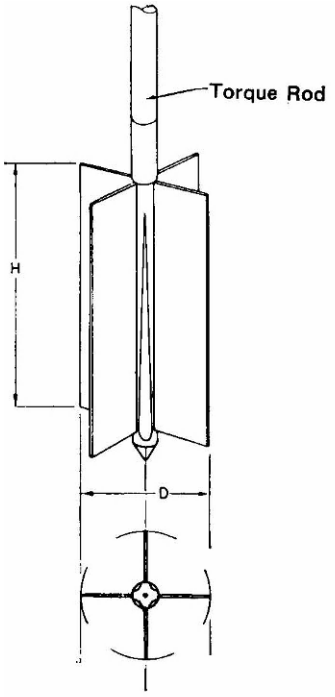


Fig. 3.45. The vane.

Fig. 3.46. *SGI* displacement vane borer.

3.3.2 Factors which influence vane tests

3.3.2.1 Vane dimensions

To determine accurately undrained shear strength, a vane must not be too small; 50 mm diameter and 100 mm high is considered the minimum size necessary. The SGI vane borer employs two standard vane sizes of 65×130 mm and 55×110 mm, and these do not always yield the same results.

In order to minimize disturbance to the soil when the vane is inserted, the thickness of the vanes and the area of the shaft should be as small as possible conducive with adequate strength. The area ratio of the vane (i.e. the cross-sectional area of the blades as a proportion of the circular plan area swept by the blades) should not exceed 0.1 (or 10%), which is the figure recommended by Hvorslev (1949) for thin-walled samplers. Wiesel (1973) has indicated, however, that it is necessary for the vane to be thick enough to have sufficient torsional rigidity to prevent varying deformations along the vane length which lead to brittle failure in quick clays.

3.3.2.2 Vane shape

Several investigators have measured the effects of vane shape on the measured shear strength of soft clays, and they have attempted to interpret the results to determine strength anisotropy. Aas (1965, 1967) and Richardson et al (1975) both used series of rectangular and triangular vanes to determine shear strengths on various planes in soft clays, but although useful data were obtained in both cases, uncertainties of interpretation remain.

There is no doubt that the measured shear strength is a function of vane shape, and it is important that this is standardized for measurements in different soft clay deposits to be comparable. The Author is of the opinion that rectangular vanes with proportions $H=2D$ are to be preferred to other shapes; such vanes measure largely the shear strength on vertical planes in the clay.

3.3.2.3 Vane insertion

Disturbance of the soil occurs by the process of drilling a hole for a vane test or by advancing a vane borer (Fig. 3.47). Hvorslev (1949) suggested that the soil is disturbed to a depth of three times the hole diameter below the bottom of the hole. Tests at the NGI, however, indicate that a vane should be advanced below the bottom of the hole to at least six times the hole diameter if a minimum of disturbance is to be present before the test commences. The disturbance caused by the advancement of the vane itself is minimized by careful control from the ground surface and by use of a vane of sufficiently low area ratio.

After each vane test, any clay sticking to the blades should be removed as far as possible before the next test proceeds; clay adhering to the vane effectively increases its area ratio. For a vane borer, a properly designed vane protection shoe ensures that the vane is cleaned each time it is retracted.

3.3.2.4 Rate of vane rotation

In all soil shear tests, the rate at which the stress or strain is applied has considerable influence on the test results, the measured shear strength decreasing with decreasing rate. Shown in Fig. 3.48 are the results of a series of vane tests carried out by Torstensson (1977), which are typical of those obtained by a number of investigators. For practical purposes, a rate of vane rotation of 0.2° per second is convenient, and this is often adopted as standard.

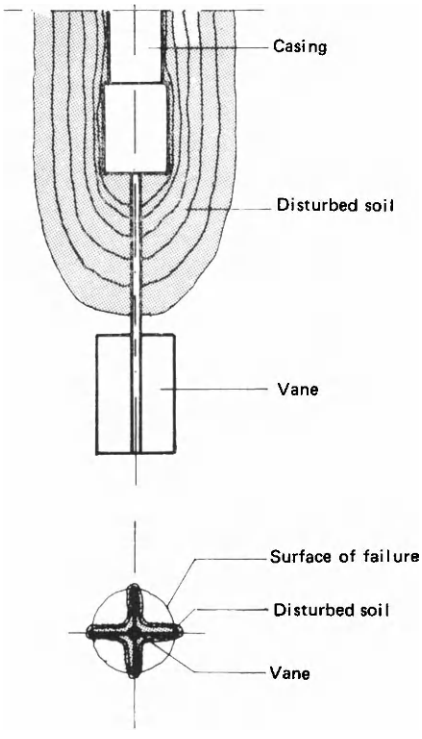


Fig. 3.47. Disturbance during penetration of a vane borer.

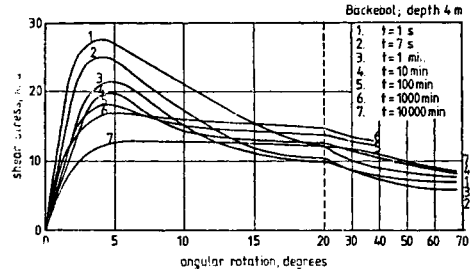


Fig. 3.48. Effect of testing rate on the 'stress-strain' curve for a vane test (Torstensson, 1977). (t = time to maximum torque).

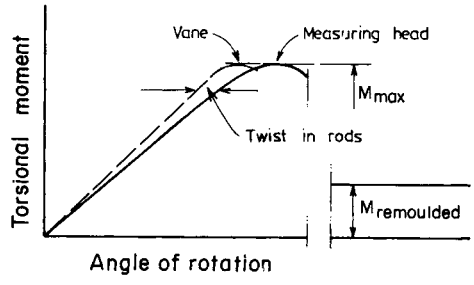


Fig. 3.49. Different rates of rotation of vane and measuring head caused by flexibility of connecting rods.

It should be remembered that it is sometimes difficult to achieve a constant rate of vane rotation when tests are carried out at great depths because of the flexibility in the rods and couplings transmitting the torque from the surface to the vane. It is possible to calibrate the system to make some correction to the rate of rotation of the torque head, but this will never be adequate, since the rotation of the vane (and therefore the twist of the rods) is controlled by the strength of the clay (Fig. 3.49).

Vane tests should, ideally, be carried out at such a rate that failure occurs after the same time of testing. The tentative Norwegian recommendations for

the vane test are that the rotation speed of the vane should be between 0.2° and 0.4° per second, and that failure should take place in 1 to 3 minutes. In practice, this is achieved by the use of a constant rate of rotation of 0.2° per second down to a depth of 15 m, beyond which the rate of rotation is doubled.

3.3.2.5 Friction

The torque applied to a vane inserted in the soil is usually measured at the ground surface. When a vane borer is used, the vane rods are housed in a tube or casing, and it is necessary for the mechanical friction to be reduced to a minimum in order to reduce errors in measurement. A calibration exercise must be carried out to correct for any friction.

Wiesel (1973) described a vane borer which measures torque and angular deformation just above the vane itself (Fig. 3.50), and a wireline vane borer with similar features (Fig. 3.51) was described by Doyle et al (1971).

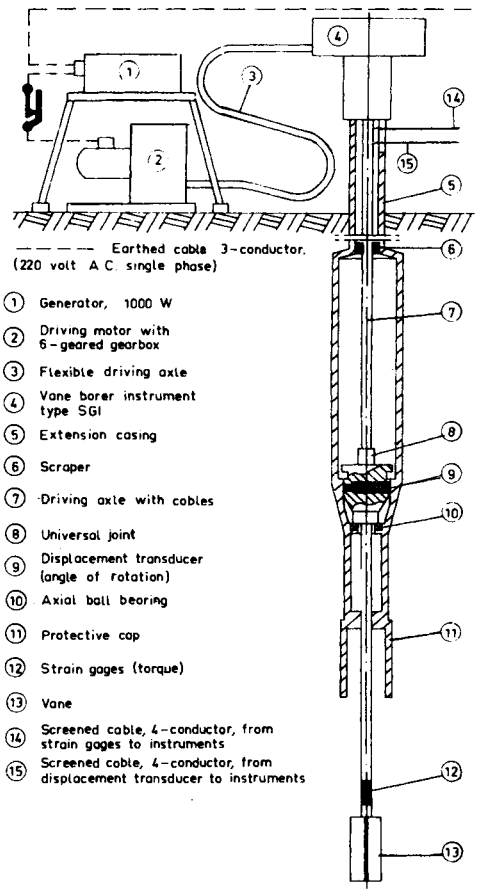


Fig. 3.50. Research vane borer described by Wiesel (1973).

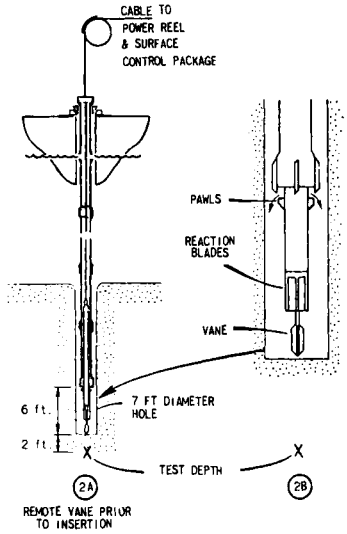


Fig. 3.51. Wireline vane borer described by Doyle et al (1971).

3.3.2.6 Thixotropic effects

It is well-known that the shear strength measured by a vane test often increases the longer the vane is inserted into the soil prior to the test. This effect, which first appears to have been reported by Flaate (1966b) (Fig. 3.52) is related to the dissipation of pore pressures induced by the vane insertion and to thixotropic strength regain of the clay. In order to obtain comparative strength data, it might be necessary in a particular clay to ensure that there is always a 'standard' delay time between vane insertion and testing.

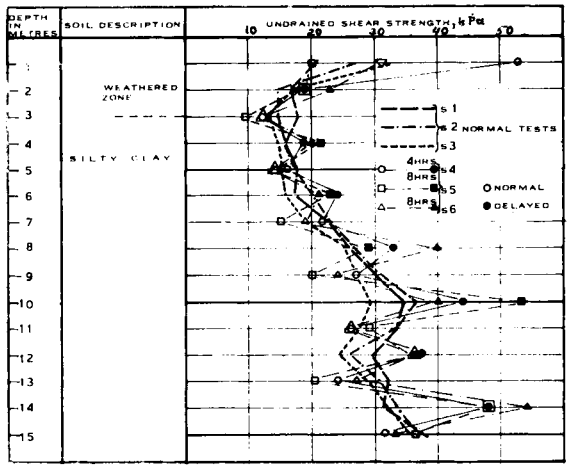


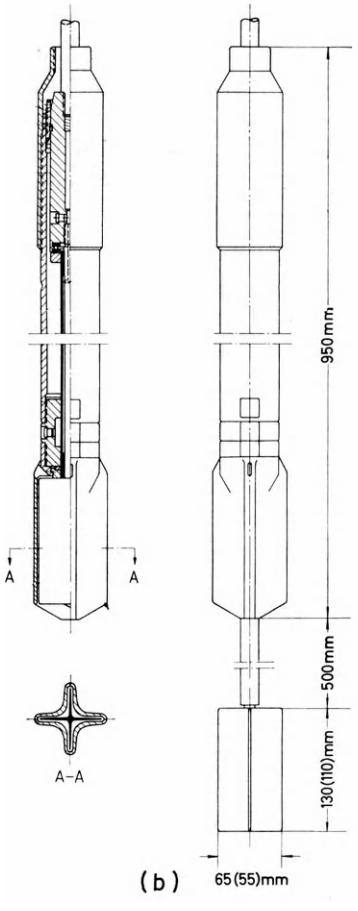
Fig. 3.52. Results of vane shear tests conducted by Flaate (1966b) at Seut, Norway, to investigate the effects of the time between vane insertion and testing.

3.3.3 Interpretation of vane test results

The stress-strain curves for vane tests in soft clays are generally of the form shown earlier in Fig. 3.48. The recording of the complete stress-strain relationship is often useful in identifying or classifying the clay deposit and enables a full interpretation of the test result. For this purpose, a strip chart recorder recommends itself but is seldom used. Commonly, the maximum torque (corresponding to the *peak* strength) only is measured, although the test is generally repeated after rotating the vane about 25 times to enable the *remoulded* strength to be measured. Where the *residual* strength is of interest, the vane test on the undisturbed clay is continued to a large strain until the measured torque drops to an ultimate constant value.

Guidance on the interpretation of results from vane shear tests has been given in many publications, recently in those by Wiesel (1973), Aas (1976), Helenelund (1977) and Torstensson (1977).

(a)



(b)

Fig. 3.53. SGI vane borer *(Geonor A/S)*: (a) torque head, (b) vane and protective shoe.

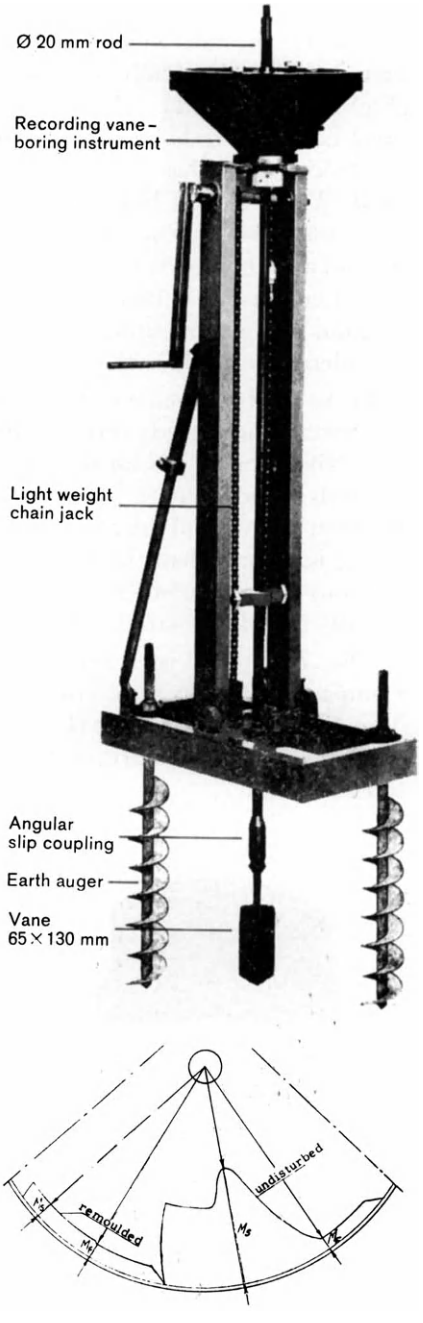


Fig. 3.54. Nilcon vane borer and a typical chart record.

3.3.4 Vane borers

Probably the most widely used vane borers are the SGI vane borer (Fig. 3.53), first described by Cadling & Odenstad (1950), and the Nilcon vane borer (Fig. 3.54) designed at Chalmer's University of Technology. The SGI equipment has inner rods housed in an outer tube to eliminate rod friction, while the Nilcon borer has only an extendable rod joining the vane to the torque head. With the SGI borer, the shear strength is measured directly from the maximum torque recorded; with the Nilcon equipment, the adhesion between the rod and the clay is first measured by activating the slip coupling, and this measured torque is deducted from the maximum value measured for the rotation of the vane and rods together. In Norway, the SGI vane borer is considered to produce the more reliable results.

In the SGI equipment, the vane is retracted into a protective shoe before the borer is advanced; this operation removes the clay sticking to the vane. The Nilcon borer has no such facility, and clay sticks to both the vane and the rods as penetration occurs. Further, at great depth, the adhesive forces between the rod and the clay result in a correction to the total recorded torque which is greater than the torque required to cause the vane alone to rotate. The one advantage of the Nilcon over the SGI equipment is that the Nilcon records the torque-strain relationship on a chart (Fig. 3.54).

The *Torvane* (Fig. 3.55), which was designed by the Slope Indicator Company, is meant to be used for shallow inspection purposes in test pits. The NGI inspection vane (Fig. 3.56) is another simple device which serves the same purpose (Andresen & Sollie, 1966), except that it can penetrate up to about 3 m.

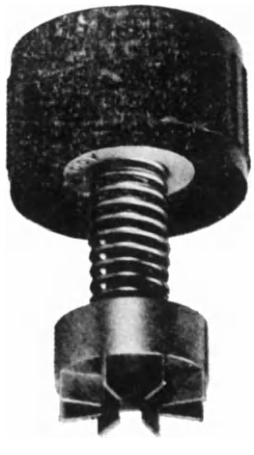


Fig. 3.55. The Torvane *(Slope Indicator Co.).*

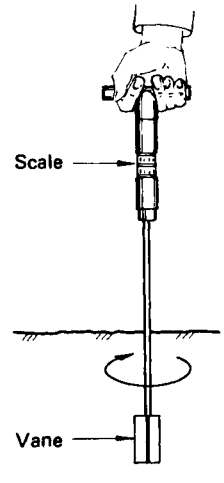


Fig. 3.56. Pocket vane borer *(Geonor A/S).*

3.4 PENETRATION TESTING

The European Symposium on Penetration Testing (1974) highlighted the interest in the application of penetration tests to all kinds of soils, including soft clays. This Symposium followed soon after the comprehensive book on penetration testing by Sanglerat (1972).

Penetration tests (or *soundings* as they are often called) are of the quasi-static and dynamic kinds. In soft clays, only quasi-static tests are of interest for the determination of shear strengths. If the soft clay is overlain by stiffer or coarser materials, however, dynamic or static-dynamic methods may be applied; in such cases the soft clay can only be 'identified' by the test, and shear strengths cannot be deduced. In this chapter, only quasi-static penetration tests will be dealt with.

3.4.1 Scandinavian penetration tests

3.4.1.1 Swedish weight sounding method

The Swedish weight sounding method has been the most commonly used penetration test in Scandinavian soft clays since it was developed by the Geotechnical Commission of the Swedish State Railways in 1914–1922. The method is also used in Japan but, according to Muromachi et al (1974), it is often employed as a supplement to the standard penetration test (SPT) or for the purpose of preliminary survey. The method is little used outside Scandinavia and Japan.

The Swedish weight penetrometer (Fig. 3.57) consists of a screw point attached to a 22 mm diameter rod which can be loaded at the ground surface by a series of weights (5, 10, 10, 25, 25 & 25 kg) to a maximum weight of 100 kg. Load is gradually added to or removed from the penetrometer to cause it to penetrate at a rate of about 20 mm/sec.; the rate should never exceed 50 mm/sec. The penetration depth and the load are recorded. When the penetrometer does not move under a weight of 100 kg, it is rotated by hand or by machine at the ground surface, and the number of half turns is recorded for each penetration of 200 mm.

Today, power units are used for the rotation of the rods, and for extracting them after a test is finished.

It should be realized that the Swedish weight sounding method does not provide 'exact' soil data. The penetration resistance is influenced by both the point resistance and the skin friction along the rods. In the Scandinavian sensitive clays, however, the skin friction is small. The method is simple, practical and economical, and it is considered to be ideal for Scandinavian conditions. All the later penetration test methods developed in Scandinavia evolved, more or less, from the weight sounding method.

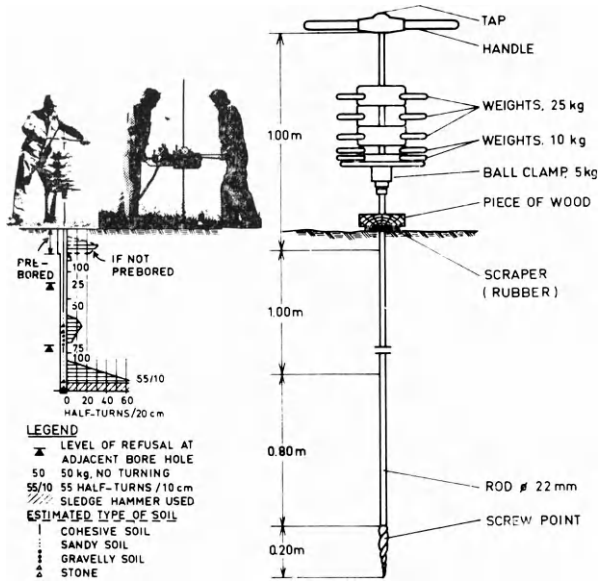


Fig. 3.57. Swedish weight sounding method (Dahlberg, 1974).

3.4.1.2 Other Scandinavian sounding methods

A motorized penetration test was developed in 1950 by the Swedish Geotechnical Institute (Dahlberg, 1974) which consists of a 60° cone, 25 or 40 mm diameter, which is pushed into the ground at a rate of approximately 50 mm/sec. (Fig. 3.58). As the cone itself penetrates without rotation, the rods are automatically rotated from the ground surface at the rate of one revolution per 12.5 mm penetration. In this way, the skin friction and point resistance can be measured separately, since the skin friction is evaluated from the measured torque and the point resistance is determined by subtracting the friction from the total penetration resistance (see Fig. 3.59). The cone resistance is recorded by a pen recorder (Fig. 3.60).

The SGI principle was used in a simplified form in the penetration test (Fig. 3.61) developed by the Norwegian Roads Authority and the Norwegian Geotechnical Institute (Andresen & Rygg, 1974).

The Nilcon equipment, recently described by Nilsson (1975), has been extensively used in Sweden during the last 20 years. The penetrometer (Fig. 3.62) is of square section of 10 cm^2 area and features a slip-coupling close to the cone which is used to determine the skin friction resistance along the rods. This is achieved by alternatively pushing the penetrometer and rods to measure the total penetration resistance, and then pulling the rods a distance of 50 to 100 mm (as permitted by the slip-coupling) to measure the skin friction. The friction force and total force are recorded on a strip chart (Fig. 3.63), and the difference between these gives the cone resistance. When

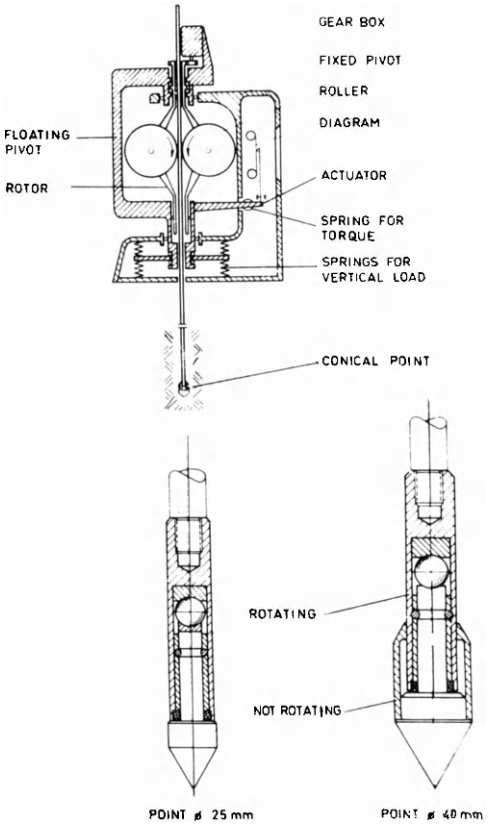


Fig. 3.58. SGI static sounding equipment (Dahlberg, 1974).

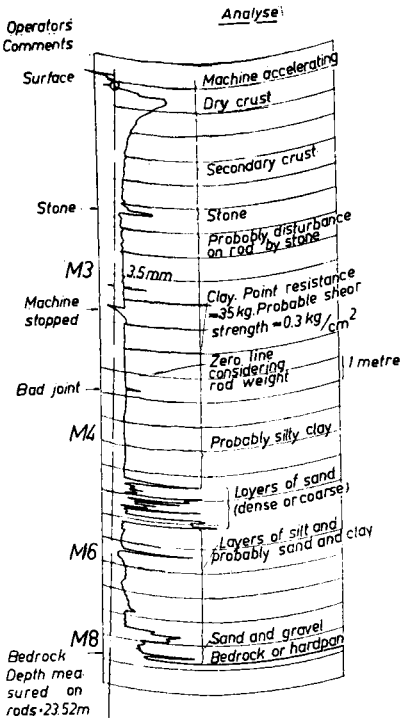


Fig. 3.60. Evaluation of test data from the SGI penetrometer (Broms, 1974).

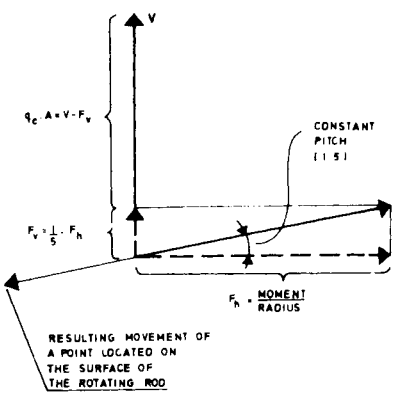


Fig. 3.59. Separation of skin friction and point resistance for the SGI penetrometer (Dahlberg, 1974).

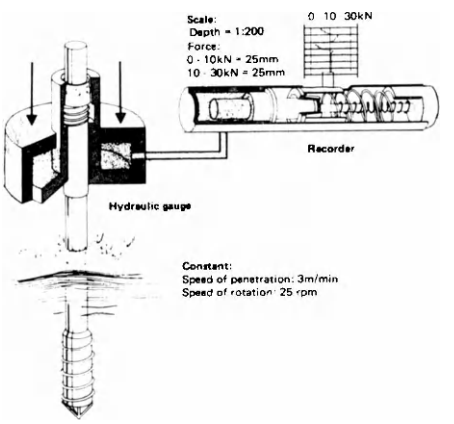


Fig. 3.61. NRA and SGI rotary sounding equipment (Andresen & Rygg, 1974).

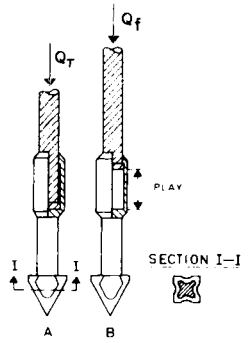


Fig. 3.62. Penetrometer used in Nilcon equipment (Nilsson, 1975).

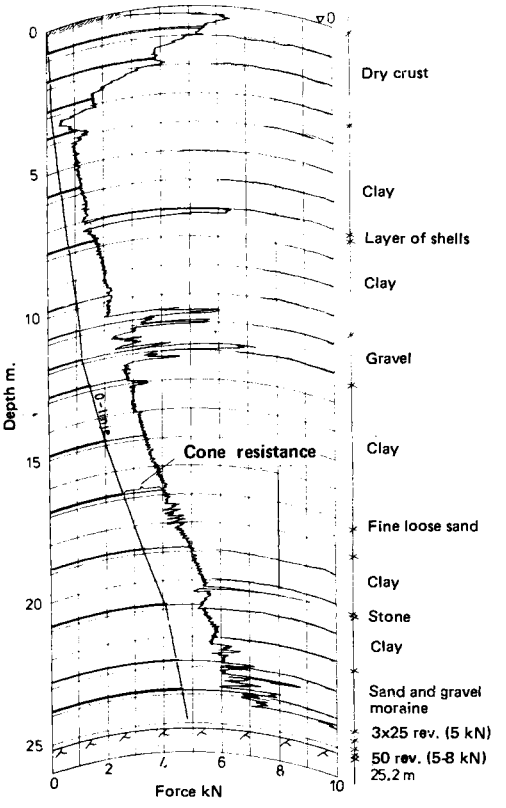


Fig. 3.63. Evaluation of test data from the Nilcon penetrometer.

the total penetration resistance exceeds the capacity of the equipment (10 to 20 kN), the point is rotated and readings taken in much the same way as with the Swedish weight sounding method.

3.4.2 Dutch cone test

The Dutch cone test was developed in The Netherlands in about 1930, and has been extensively used in The Netherlands, Belgium, France and (to a lesser extent) Germany. At ESOPT (1974), the use of this test method was mentioned in almost every national report. The test consists essentially of the continuous penetration of a 60° cone of 10 cm² area at a rate of 20 mm/sec. The penetration resistance is measured, and local friction is also often recorded. The application of the test to soft clays was for many years limited mainly to the determination of the existance and extent of soft clay layers, but more use of the method has been made recently for the indirect measurement of shear strength.

The undrained shear strength, s_u, of a clay is normally calculated directly from the cone resistance, q_c, by means of the relationship:

$$q_c = Ns_u + \gamma z \tag{3.8}$$

where γ is the bulk density of the soil, z is the depth, and N is the *cone factor* for the particular clay. The value of N is a function of the type of clay and the strength test against which the Dutch cone results are calibrated (Amar et al, 1975; Lunne et al, 1976).

The measurement of local friction, f_s, by means of a friction sleeve (as described below) enables fairly accurate soil identification. For this purpose, the friction ratio, R_f, or the friction index, I_f, is commonly used; these are defined as:

$$R_f = \frac{f_s}{q_c} \times 100\% \tag{3.9}$$

$$I_f = \frac{q_c}{f_s} \tag{3.10}$$

The many criteria employed for the identification of soils on the basis of R_f and I_f have been summarized by Sanglerat (1972).

The mechanical Dutch cone (Fig. 3.64) consists of a cone attached to a rod which is housed in an outer tube up to the ground surface (Vermeiden, 1948). The system is mechanically discontinuous (which is a slight disadvantage) in that the cone and inner rod are pushed a short distance (usually 70 mm) and stopped before the outer tube follows. The outer tube eliminates the friction that would otherwise exist between the rods and the clay, so that the cone resistance is measured directly. Mechanical friction occurs between the rods and the tube, however, and must be allowed for in the interpretation of the test data. It was for reasons of mechanical friction that NGI was discouraged from adapting the Dutch cone system for continuous penetration and measurement (Andresen, 1969).

An improved version of the original Dutch cone is the mantle cone, and this was later developed further to incorporate a friction sleeve (or adhesion jacket) to enable the skin friction between the sleeve and the clay to be measured at any depth (Begemann, 1953; Heijnen, 1974) (Fig. 3.65). The latter type of cone involves penetration of the cone alone, then the cone plus the jacket and, finally, the outer tubes. The point adhesion is deduced from the difference between the resistance of the cone alone and the resistance of the cone plus the sleeve.

3.4.3 Electric penetrometers

Cone penetrometers are now available which record the penetration resistance electrically. The main advantage of this type of instrument is that

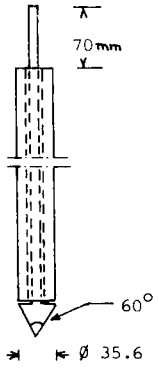


Fig. 3.64. Original Dutch cone (Vermeiden, 1948).

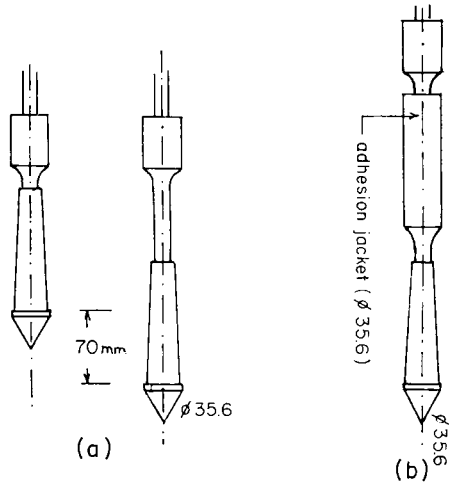


Fig. 3.65. Present form of Dutch cone: (a) mantle cone, (b) adhesion jacket cone (Begemann, 1953).

continuous measurement and recording are possible (de Ruiter, 1971). In addition, a greater accuracy of measurement can be achieved for the cone resistance. In Fig. 3.66 is shown a cross-section of the electric cone developed by the Country Roads Board, Victoria (CRB), Australia (Holden, 1974a) which is fairly typical of the electric cone assemblies now being used in many places.

The low cone resistances (1 to 2 kN) encountered in soft clays are difficult to measure accurately, and this is true also for electric penetrometers. The resistance must be determined to an accuracy of 20 to 100 N on a 10 cm^2 cone. At NGI, an electric cone has been designed on the basis of that described by Holden (1974a). The sensing element for measuring cone resistance is a concentric cylinder load cell with strain gauges. New types of high quality steel make it possible to work with stresses of up to 1800 MPa, with correspondingly high strains, but there are practical limitations on the minimum wall thickness that can be employed. NGI has achieved a sensitivity of ± 10 N and a linearity and hysteresis better than $\pm 10\%$ in the measuring range 100 to 490 N on a penetrometer of 120 kN capacity. The accuracy of a strain gauge penetrometer depends largely on very careful temperature compensation and calibration, and even with the best compensation, small temperature gradients within the sensing element affect the readings. A thermal gradient of only 0.25°C between the terminals of the strain gauge can correspond to a 30N load on the cone.

Geotek AB (Jonell, 1975) and Borros AB (personal communication) have developed wireless transmission systems for their cone penetrometers to eliminate electric cables from the penetrometer to the ground surface. It is

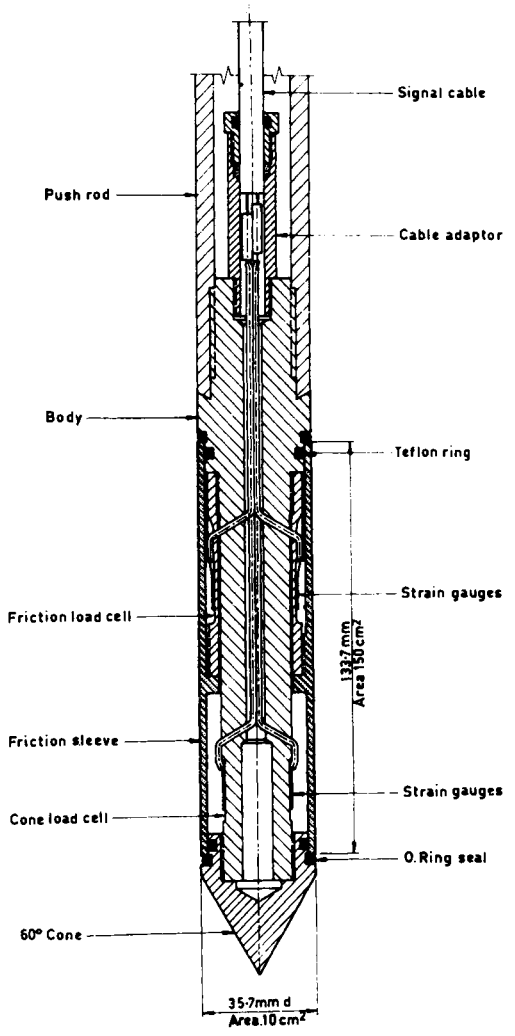


Fig. 3.66. Electric penetrometer of the Australian Country Roads Board (Holden, 1974a).

the Author's opinion, however, that cables cause no practical hardships when pre-threaded through the push-rods and stored in the manner described by de Ruiter (1971).

3.4.4 Other cone penetration tests

A number of other simple quasi-static cone penetration tests are in use throughout the world for the indirect determination of shear strength or bearing capacity. Worth particular mention is the light, portable penetrometer used extensively in soft clays in Japan (Muromachi et al, 1974). This equip-

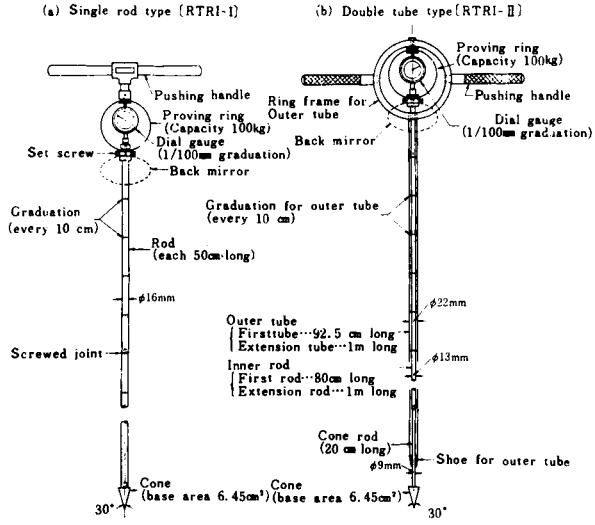


Fig. 3.67. Portable cone penetrometer used in Japan (Muromachi et al, 1974).

ment, which is used with or without an outer tube (Fig. 3.67), is patterned on that first developed at the Waterways Experiment Station in the USA.

3.4.5 Special cone penetrometers

In recent years, developments in instrumentation have led to cone penetrometers being devised for special purposes. In particular, equipment now exists for the measurement of pore pressure and cone resistance simultaneously (Senneset, 1974; Kallstenius, 1975; Schmertmann, 1975). Janbu & Senneset (1974) have suggested that pore pressure data are more useful for soil identification purposes than are local friction measurements, and that such data are invaluable for the interpretation of penetration tests in clay. Continuous measurements of pore pressure are possible with the *piezometer probes* developed by Torstensson (1975) (Fig. 3.68) and Wissa et al (1975) (see Fig. 3.78 and Section 3.8).

The *corrosion probe* (Rosenqvist, 1956) was developed at the NGI to determine the corrosion susceptibility of steel to soils, especially soft clays. This probe permits a rapid in-situ determination of the cathodic depolarization and the specific soil resistance, which are the two most important factors which govern the corrosion rate. Söderblom (1969) modified the NGI probe and used it to determine the salinity of Swedish quick clays; the same probe has also been used to locate failure surfaces after slides (Broms, 1974).

3.4.6 Standardization of penetration tests

Under the auspices of the International Society for Soil Mechanics and Foundation Engineering (ISSMFE), attempts were made between 1957 and

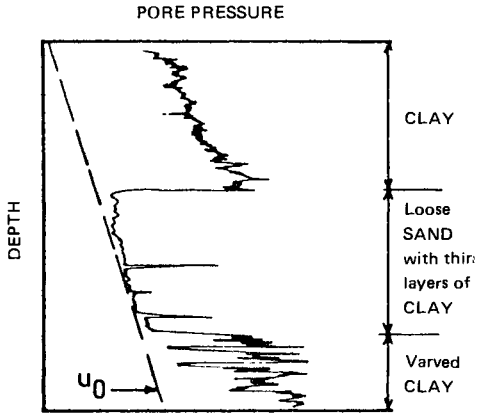


Fig. 3.68. Typical measurements obtained with Torstensson's (1975) pore pressure probe.

1977 to obtain international agreement on the standardization of penetration tests. These lengthy efforts culminated in the production of a document entitled *Report of the Sub-committee on the Penetration Test for Use in Europe* (ISSMFE, 1977), which recommended the standard equipment and methods to be employed for four commonly used penetration tests. These four tests were given the titles:

(1) Cone Penetration Test (CPT),
(2) Dynamic Probing Test (DPA and DPB),
(3) Standard Penetration Test (SPT), and
(4) Weight Sounding Test (WST).

It was recommended that these four be used as reference standards against which all other penetration tests were compared or calibrated. Of these four standard tests, only the CPT and WST are relevant to soft clays.

The CPT standard adopted a continuously penetrating 60° cone of 10 cm^2 area for which the cone resistance (and local friction if required) is measured and recorded electrically. The basic principle of the test and the equipment recommended are similar to those employed by de Ruiter (1971) in his electric version of the Dutch cone test. The report of the ISSMFE (1977) also presented details of a large number of static and quasi-static penetrometers used throughout the world and indicated how they differed from the recommended CPT.

The WST recommendations made by the ISSMFE followed closely the procedure already adopted in Scandinavia for the Swedish weight sounding test (see Section 3.5.1.1)

3.5 OTHER STRENGTH TESTS

3.5.1 The iskymeter

In Fig. 3.69 is shown the *iskymeter* developed at SGI (Kallstenius, 1961). This is a sounding device consisting of foldable blades on the end of a tube which is driven into the ground. When the folded blades reach the desired depth, the iskymeter is pulled out by means of a wire rope and a winch. The pull-out resistance is continuously recorded on a drum recorder, and the undrained shear strength is deduced from these measurements.

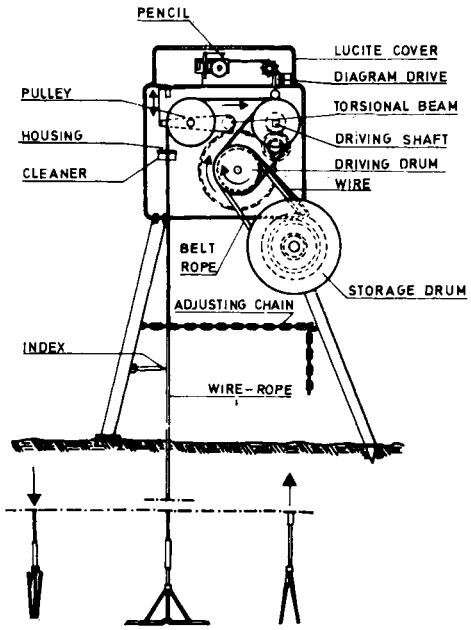


Fig. 3.69. SGI iskymeter (Kallstenius, 1961).

The iskymeter has not been used very extensively, although Muramachi et al (1974) reported that it is still occasionally employed in Japan.

3.5.2 Borehole shear device

The borehole shear device was developed by Handy & Fox (1967) for the purpose of in-situ strength measurements at any depth in a borehole. A modified version of this, used by the Kansas Highway Commission, was described by Wineland (1975). The shear head of the Kansas equipment (Fig. 3.70) consists of two curved shear plates which are forced against the borehole sides by a piston operated by a pressure regulator attached to a gas cylinder. The shear head is then pulled vertically upwards by means of a

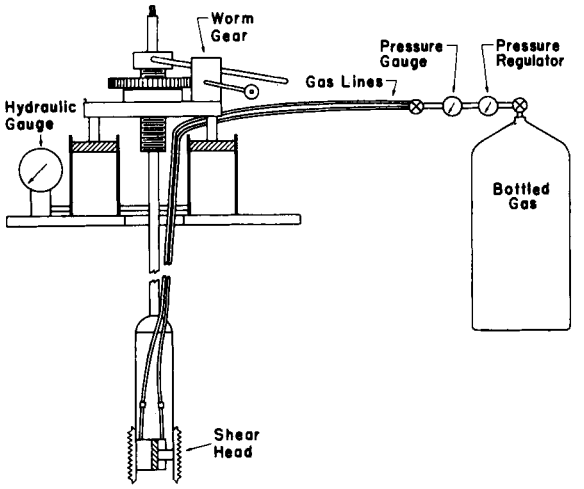


Fig. 3.70. Kansas borehole shear device (Wineland, 1975).

screw arrangement applied to the threaded upper end of the rod attached to the shear head. The force required to pull the shear head is measured on a hydraulic gauge attached to two pressure cylinders. The shear strength of the soil is calculated from the maximum force measured as a function of the normal stress applied to the borehole sides.

3.5.3 Large-scale field tests

The technical literature contains details of a number of large-scale field tests carried out to calibrate commonly used methods of in-situ strength measurement to verify design assumptions, or to facilitate better predictions of the performance of finished structures. An example of this category of test is the series of huge in-situ direct shear tests carried out by NGI (DiBiagio & Aas, 1967) to verify field vane results. The principle employed (Fig. 3.71) was that of a self-boring shear box. A 1 m diameter core of undisturbed clay was formed as the equipment bored into the ground (Fig. 3.72). This core could then be sheared in-situ at any depth to obtain the shear strength on a horizontal plane. The equipment was later modified to enable shear tests to be carried out at 45° inclinations.

3.5.4 Stress-strain-strength devices

Section 3.7 deals with the in-situ measurement of the stress-strain characteristics of a soil. Such measurements generally involve the use of sophisticated equipment which is also capable of measuring the shear strength of the soil. The several types of pressuremeter all have this capability, either directly or indirectly. Reference should be made to Section 3.7 for a description of these.

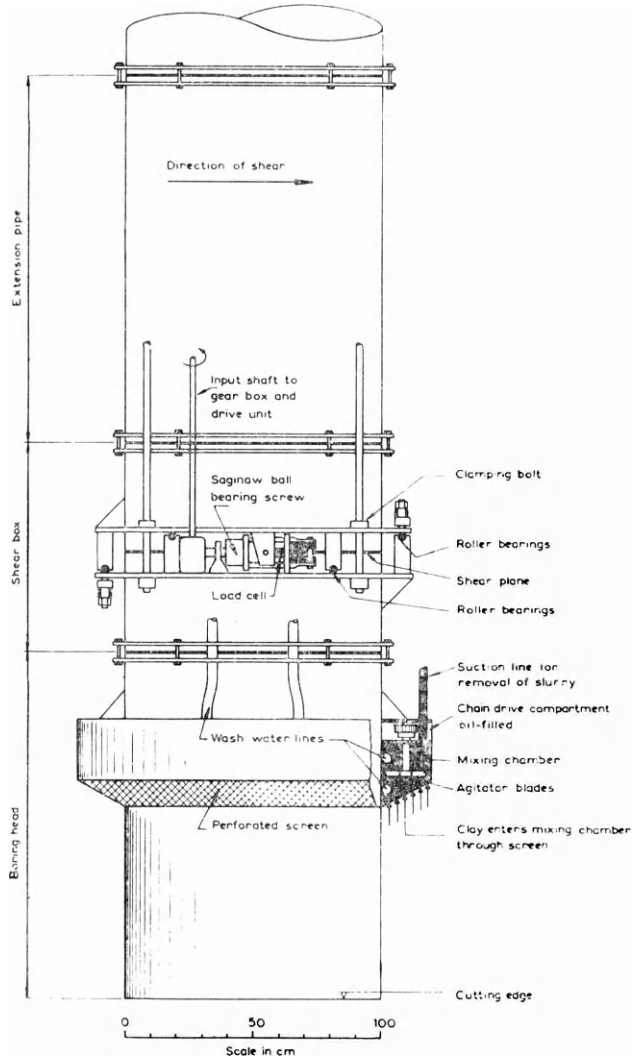


Fig. 3.71. NGI self-boring direct shear apparatus (DiBiagio & Aas, 1967).

3.6 MEASUREMENT OF STRESS-STRAIN CHARACTERISTICS

The in-situ measurement of the stress-strain-strength characteristics of soft clays and the correlations of these with stress history have recently become the subject of renewed interest and research. Although laboratory tests on undisturbed samples are thought to provide fairly reliable estimates of most of the parameters required for foundation design, the coefficient of earth

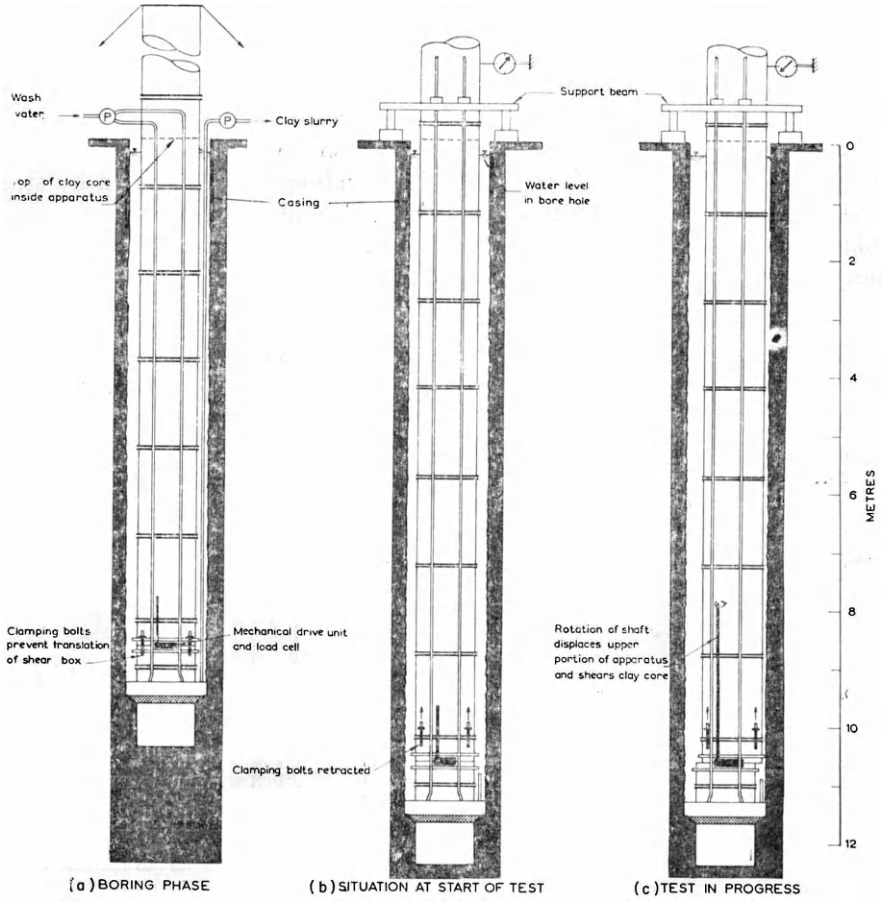


Fig. 3.72. Operation of NGI self-boring shear apparatus (DiBiagio & Aas, 1967).

pressure 'at rest', K_o, the shear modulus and the preconsolidation pressure are often difficult and laborious to obtain in the laboratory.

The in-situ determination of the stress-strain characteristics of soils calls for rather sophisticated field testing techniques. Field measurements of total horizontal stress in soft clays and the derived values of K_o can be obtained by self-boring pressuremeter tests, by direct measurements of lateral earth pressures by total stress cells, and by hydraulic fracturing tests with hydraulic piezometers. Pressuremeter tests also yield values of the shear modulus. Estimates of vertical preconsolidation pressure can be obtained from field vane measurements of shear strength versus depth, especially in clays where laboratory measurements of stress history have already been correlated to field vane strength profiles.

3.6.1 The Ménard pressuremeter

First proposed in Germany in 1933, the pressuremeter test was subsequently developed by Ménard (1956) to measure the in-situ strength-deformation characteristics of soils and rock. The Ménard pressuremeter device (Fig. 3.73) is widely used in France for the design of both shallow and deep foundations. It consists of a cylindrical probe, made of rubber and protected by steel 'scales', which is inserted into a predrilled hole and pressurized by gas. The volume of fluid injected into the cell under increasing pressure increments is measured to obtain the radial stress-strain characteristics.

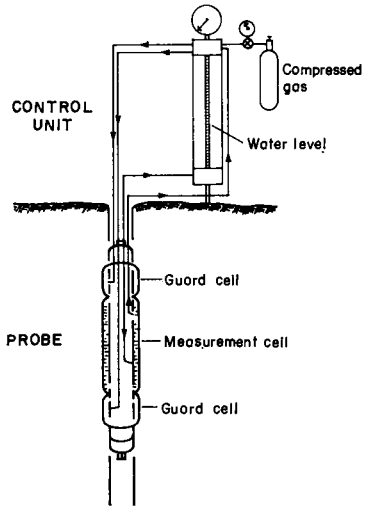


Fig. 3.73. Ménard pressuremeter (Ladd et al, 1977).

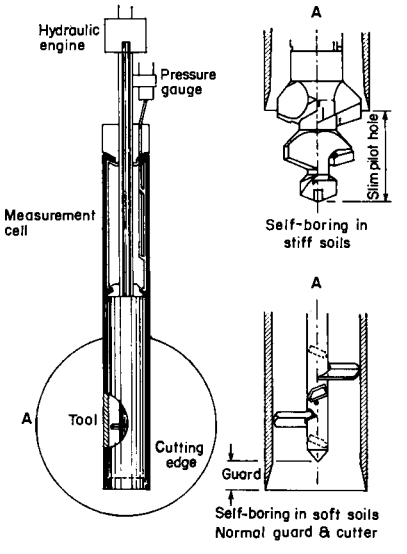


Fig. 3.74. French self-boring pressuremeter (Ladd et al, 1977).

The Ménard pressuremeter has so far found only limited use outside of France, but increased interest in the method might have resulted from the publication of the book by Baguelin et al (1978). There remain the difficulties associated with drilling disturbance and end effects at large strains, and various other procedural problems. As a result, essentially empirical design procedures based on French experience have been adopted.

The Ménard pressuremeter is not suitable for measuring the total horizontal stress 'at rest' in soft clays.

3.6.2 Self-boring pressuremeters

Self-boring pressuremeter devices were developed independently in France (Baguelin et al, 1972), and in England (Wroth & Hughes, 1973). Based on the

same measurement concept as the Ménard pressuremeter, these self-boring devices both feature a small rotary boring tool at their tip (Fig. 3.74) and the use of circulating water to carry the soil cuttings as a slurry up an inner tube. The advantage of this type of equipment is that the pressuremeter is inserted with far less disturbance than is caused by predrilling in soft clays. The latest models incorporate load cells to make independent measurements of horizontal stress, and transducers to monitor pore pressure. Highly-trained personnel are required for their operation, however, because the installation and test procedures are complicated.

In theory, the self-boring pressuremeter can yield in-situ total horizontal stress, pore pressure (if measured), shear modulus and undrained shear strength of the soil. On the assumption that the guard cells minimize end effects, the test data are analyzed on the basis of the radial expansion of a cylindrical cavity. Analyses by Baguelin et al (1972), Ladanyi (1972) and Palmer (1972) enabled derivation of the complete undrained stress-strain curve of the soil from the test results. The solution assumes the soil to have a unique stress-strain relationship.

The self-boring pressuremeter, although still mainly a research tool, offers the capability of in-situ investigation of the strength-deformation properties of soils in greater detail than previously possible, although further critical appraisal is required of the data obtained. Baguelin et al (1978) and Ladd et al (1977) have discussed, to a limited extent, the results of self-boring pressuremeter tests and have reviewed the assumptions and major problems of the method. Of the uncertainties of the test, the amount of disturbance that occurs during installation and its effect on the derived stress-strain curve needs careful examination.

The self-boring technique for obtaining K_o in soft soils appears promising, although it has not yet been investigated or evaluated sufficiently. Wroth & Hughes (1973, 1974) have presented reasonably good K_o data. Baguelin et al (1974), however, have reported that small changes in the installation technique in stiff soils cause large variations in the measured horizontal stress; in soft clays, their results were less erratic, the horizontal stress averaging approximately 20% lower than the value obtained by hydraulic fracturing (see Section 3.7.4). Roy et al (1975) showed that disturbance effects during insertion of the pressuremeter in a sensitive Canadian clay make the interpretation of the test results very difficult. Tavenas et al (1975) presented K_o measurements from self-boring pressuremeter tests which were 40% lower than their 'best estimate' of the true value for a soft clay, and Massarsch et al (1975) reported results which were $\pm 15\%$ of the 'true' value.

3.6.3 Total stress cells

Total stress cells consist of wide, thin (10 mm or less), spade-like probes equipped with a total pressure sensor which are inserted vertically into the

soil. They are so designed to minimize disturbance effects and to model as closely as possible the deformation modulus of the soil while still providing a satisfactory response. After insertion, the total horizontal stress is measured directly, after dissipation of the excess pore pressures caused by insertion (Fig. 3.75). The cell used is often a modified Glötzl earth pressure cell, which consists of a 3 mm thick pressure cell which is expanded slightly (0.3 μm) by the application of controlled pneumatic pressure increments. The pressure required for a small induced expansion is taken as the total horizontal stress. This fairly new technique is restricted to soft soils.

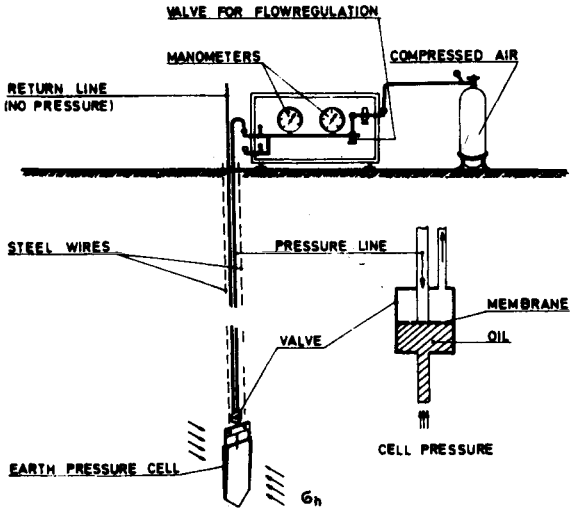


Fig. 3.75. Total stress cell equipment (Massarsch et al, 1975).

Massarsch et al (1975) and Tavenas et al (1975) compared earth pressure cell measurements with results from hydraulic fracture tests, and concluded that the former gave more consistent (though sometimes scattered) results and 'reasonable' K_0 values. They cautioned, however, against the placement of undue reliance on the measured values.

NGI constructed a large self-boring device in 1969 to measure the in-situ lateral pressures in soft clays. This consisted of a steel pile (Fig. 3.76) instrumented with total stress cells which penetrated under its own weight as a cutting chain between thin steel plates at the pile's tip excavated the clay which was removed by flushing water.

3.6.4 Hydraulic fracturing

Hydraulic fracturing consists in the initiation of a crack in the soil adjacent to a piezometer by the gradual increase in the water pressure in the piezometer.

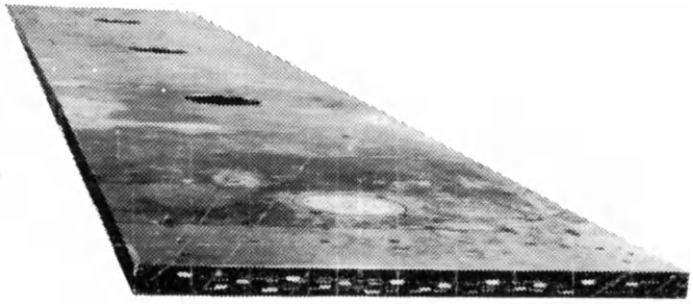


Fig. 3.76. NGI self-boring instrumented sheet pile.

When the fracturing occurs, a sharp increase in the flow rate from the piezometer takes place. The seepage flow is then monitored as the pressure gradually decreases; this is comparable to a falling-head permeability test. An abrupt reduction in flow rate occurs when the crack closes, and the pressure at which this happens is then assumed to be equal to the minor principal total stress. When K_0 is less than unity (soft soils), the crack is vertical, and the closure pressure corresponds to the total horizontal stress.

Originally used for the determination of in-situ stresses in rock, the hydraulic fracturing method was first applied to clays by Bjerrum & Andersen (1972), who used the equipment shown in Fig. 3.77; they also developed the interpretation technique now widely used by others. Their measurements at six sites yielded K_0 values almost the same as or slightly higher than K_0 values measured in the laboratory which were believed to be reliable. Massarsch et al

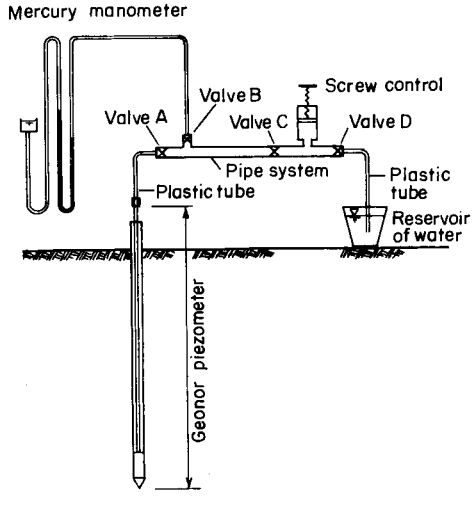


Fig. 3.77. Hydraulic fracturing equipment (Bjerrum & Andersen, 1972).

(1975), however, reported generally very erratic K_o data from hydraulic fracturing tests in five soft deposits. Although Tavenas et al (1975) obtained reasonable results in a sensitive clay, they attributed these to a fortuitous compensation of disturbance effects and testing errors rather than to the reliability of the test technique. Massarsch & Broms (1976) have further questioned the hydraulic fracturing method.

The reliability of hydraulic fracturing tests appears to vary with soil type, testing technique, and even the method of data interpretation (Ladd et al 1977). Small differences in measured horizontal total stress can lead to significant variations in the coefficient K_o. Massarsch et al (1975) reviewed thoroughly the various sources of error inherent in the technique.

3.6.5 Comparison of methods for stress-strain characteristics

The Ménard pressuremeter, generally interpreted by means of empirical formulae, has found little application in clays because of important disturbance effects. Neither is the device suitable for measuring the in-situ horizontal stress.

In-situ techniques that enable determination of the coefficient of earth pressure 'at rest', K_o, are fairly new. The self-boring pressuremeter and total stress cells require advanced technology and are more time-consuming than the hydraulic fracturing method, but they should yield more consistent values. Recent comparative studies of the three in-situ techniques have led to conflicting results and divergent opinions as to their respective reliabilities. Laboratory test procedures on undisturbed specimens that can provide reliable estimates of K_o often prove complex and very costly. It is desirable, therefore, that the in-situ horizontal stress and K_o are predicted from already established laboratory correlations between K_o and stress history as a function of plasticity index; for example, those proposed by Brooker & Ireland (1965) and Ladd et al (1977). The use of these relationships should yield results at least as accurate as the presently available in-situ techniques and their interpretation for most cohesive soils. Tavenas et al (1975) have suggested that the values of K_o based on such correlations are likely to be more accurate than the results of any in-situ technique, and that these values are probably of sufficient accuracy for the solution of most practical problems. In those cases where the empirical formulae do not apply, it appears that in-situ tests are not likely to yield any better results.

Self-boring pressuremeters offer great potential, especially for the establishment of complete in-situ stress-strain curves for clay soils. A great deal more work is needed, however, to perfect the test technique and to gain experience in different soil types (Ladd et al, 1977). Because the test is costly and requires more technical expertise than other in-situ tests, the profession should be aware of the full capabilities and limitations of the test before utilizing it in practice.

3.7 PERMEABILITY TESTING

The accurate determination of the permeability of clays is very difficult, mainly because of the extremely low values of permeability which clays exhibit. Laboratory tests rarely give the same values as in-situ measurements, field permeabilities generally being much higher than laboratory values. This is partly explainable by the fact that small test specimens are used for laboratory tests and that water flow takes place vertically; the macro-structure of the clay might not be properly represented by the test specimen, and the horizontal permeability might be higher than the vertical value. For in-situ permeability measurements, the main cause of concern is the disturbance of the soil which occurs when the measuring device is inserted.

An in-situ permeability test in soft clay is usually performed by means of a hydraulic piezometer, usually installed by pushing without a borehole, on which a falling-head permeability test is carried out. The water pressure applied to the piezometer from the ground surface must not be so high as to cause hydraulic fracturing (see Section 3.7.4). The analysis of the test data is usually based on the equation developed by Hvorslev (1951) for flow from a piezometer.

Piezometer probes have been developed independently by Torstensson (1975) and Wissa et al (1975), the latter being illustrated in Fig. 3.78. These instruments afford a means of estimating permeability continuously with depth by the application of the analytical solution for the rate of excess pore pressure dissipation.

Because of the disturbance which results when a piezometer is pushed into a soft clay, a self-boring permeameter (Fig. 3.79) has been developed by Jezequel & Mieussens (1975). This equipment is similar to the self-boring pressuremeter (see Section 3.7.2) and consists of a cutting cylinder with an internally mounted rotary chopping tool; the soil debris is flushed to the surface. While the boring is taking place, the de-airing cylinder is fixed in a protective position over the permeameter filter element by means of two inflated ward cells. When the unit reaches a depth at which a permeability measurement is required, the ward cells are deflated, and the permeameter descends as further drilling occurs, until the filter element is brought into direct contact with the soil. The ward cells are then inflated once more in order to prevent leakage of water up the drilled hole.

3.8 GEOPHYSICAL LOGGING

Geophysical borehole logging methods have some useful applications to soft clays, but these have not yet been fully explored. More than a dozen different logging methods are in use in the petroleum industry, of which the electrical (resistivity), seismic (wave velocity and amplitude) and radiation (natural gamma, gamma and neutron rays) methods are of some use in soft

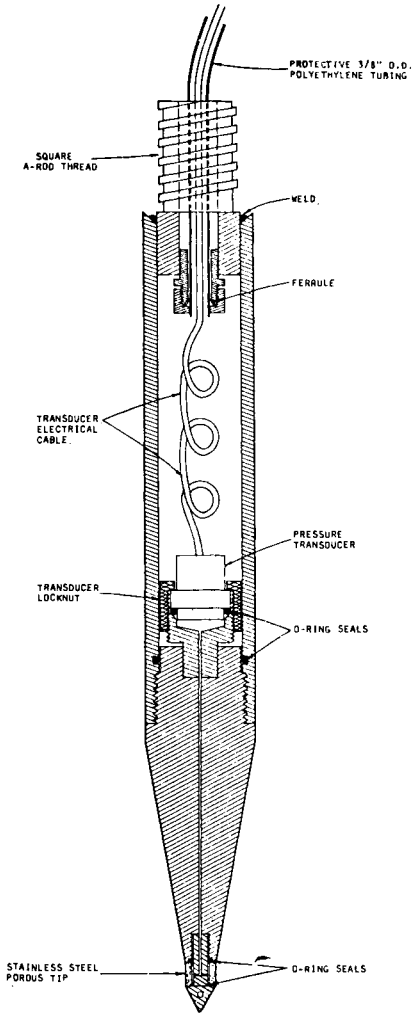


Fig. 3.78. Piezometer probe (Wissa et al, 1975).

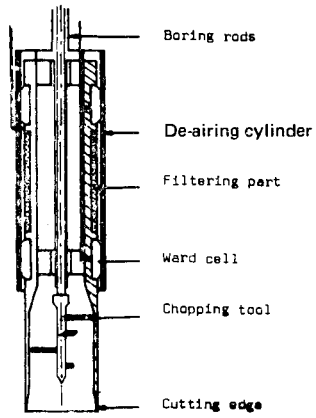


Fig. 3.79. Self-boring permeameter (Jezequel & Micussens, 1975)

clays (Guyod, 1964). All these methods, which are usually applied in uncased boreholes (Fig. 3.80), are limited mainly to the determination of the soil profile, and it is necessary for the interpretations to be checked against samples.

Resistivity measurements identify differences in soil porosity and pore salinity. The natural gamma radiation from clay minerals differs from that from coarser materials. Gamma rays applied to the soil disperse at different rates in soils of differing porosities, and this enables estimation of the in-situ density. The scatter of neutron rays is a function of the amount of water in a soil body, and this enables a determination of water content. Experience has

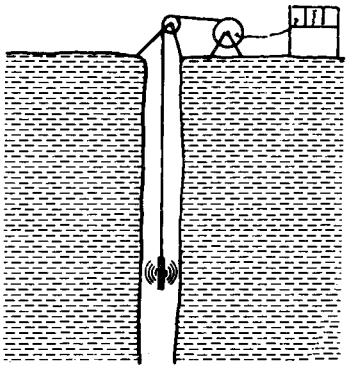


Fig. 3.80. Geophysical borehole logging.

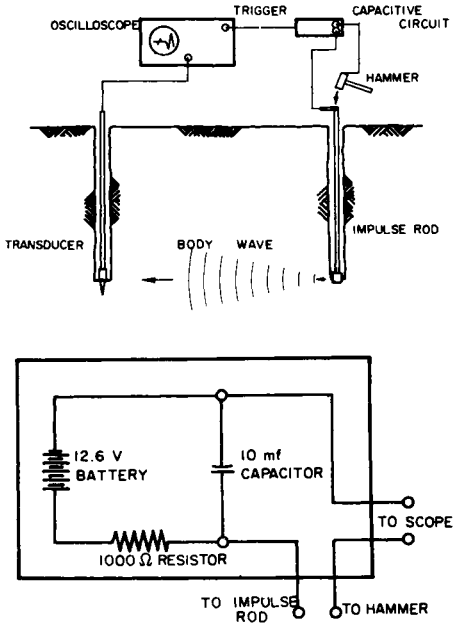


Fig. 3.81. Cross-hole seismic method (Ballard & McLean, 1975).

shown that in-situ porosities can also well be determined by acoustic measurements in holes containing water or drilling mud.

Of the geophysical methods mentioned above, none measures the in-situ soil modulus, which is often required for engineering design. The cross-hole seismic method (Ballard & McLean, 1975), the principle of which is illustrated in Fig. 3.81, appears to offer a means of doing this reasonably well. The more recently devised in-situ impulse test (Fig. 3.82) measures the dynamic shear modulus of soil and its relationship with shear strain at strain levels which correspond to the range of heavy earthquake motions (Miller et al, 1975).

3.9 STANDARDIZATION OF TEST METHODS

The standardization of test equipment and test procedures clearly facilitates the direct meaningful comparison of test data, and it is highly desirable that the tests commonly used in soft clays should be standardized. Standardization, however, should not be permitted to inhibit the development of new test methods nor the improvement of existing ones.

National standardization of test methods is increasing, but the international standardization of soil tests is fraught with difficulties. A considerable step forward was made recently with the publication by the International Society for Soil Mechanics and Foundation Engineering (1977) of the recommended standards for penetration tests (see Section 3.5.6).

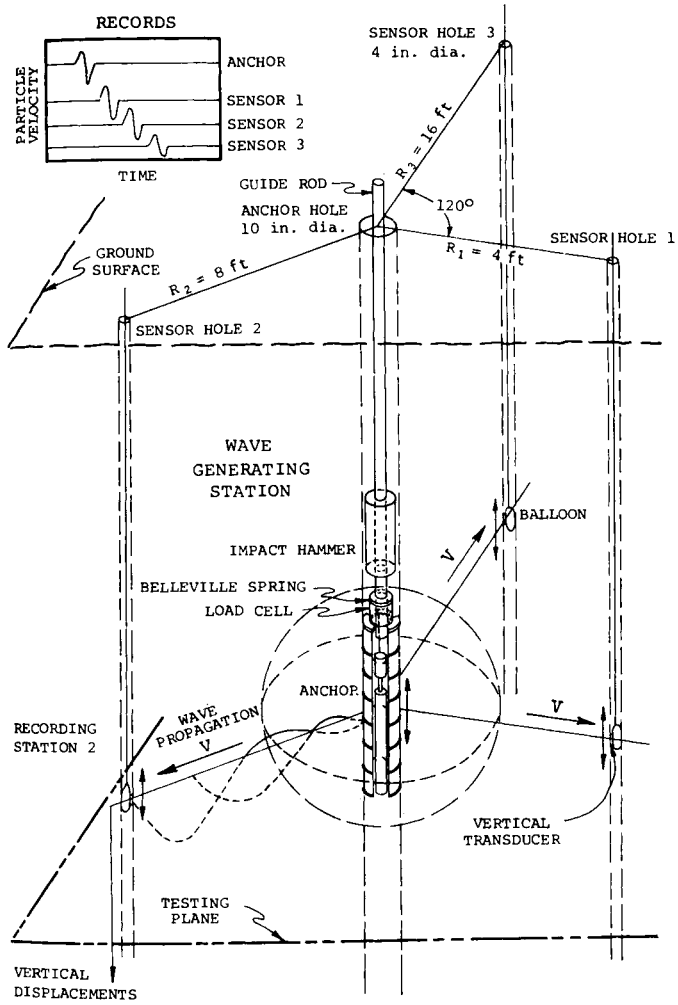


Fig. 3.82. In-situ impulse test (Miller et al, 1975).

Some well-known and commonly used standards which are applicable to the in-situ testing of soft clays are:

(1) American Society for Testing and Materials (ASTM).

Standard Method for Field Vane Shear Test in Cohesive Soil. ASTM Designation D 2573–72.

Standard Method for Soil Investigation and Sampling by Auger Borings. ASTM Designation D 1452–65.

Standard Method for Penetration Test and Split-barrel Sampling of Soils ASTM Designation D 1586–67.

Standard Method for Thin-walled Tube Sampling of Soils. ASTM Designation D 1587–74.

(2) Australian Standards Association.

Site Investigation Code. Australian Standard AS 1726–1975.

Determination of the Penetration Resistance of a Soil. Australian Standard AS 1289. F 5.1–1977.

(3) British Standards Institution.

Methods of Test for Soils for Civil Engineering Purposes. British Standard 1377, 1975.

British Code of Practice on Site Investigation. CP 2001, 1957.

(4) Bulgarian State Standards.

Building Foundations and Equipment.

Static Penetrometers for Testing Building Ground (in Bulgarian). Bulgarian Standard 8002–70, 1970.

(5) Canadian Standards Association.

Code for Split-barrel Sampling of Soils. Canadian Standard A 119.1, 1960.

(6) German Standards Association.

Baugrund; Erkundung durch Schürfe and Bohrungen sowie Entnahme von Proben; Aufschlüsse im Boden. DIN 4021, Blatt 1, 1971.

Baugrund; Ramm-und Drucksondiergeräte. DIN 4094, Blatt 1 & 2, 1974.

(7) Japanese Society of Soil Mechanics and Foundation Engineering.

Soil Sampling Manual, 1963.

ACKNOWLEDGEMENTS

The Author acknowledges the valuable assistance received from his colleagues at the Norwegian Geotechnical Institute while he was preparing the manuscript for this chapter. He is particularly indebted to a number of these. Mr Ove Eide read and discussed with the Author the preliminary manuscript prepared for the International Symposium on Soft Clay in Bangkok in July 1977. Dr Suzanne Lacasse wrote Section 3.6 on the measurement of the stress-strain characteristics of soft clays, and Mr Per Kolstad prepared the draft for Section 3.3.3 on sample disturbance. Mrs Amy and Mr Steinar Sollie assisted the Author with his written English.

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Chapter 4

SHEAR STRESS-STRAIN PROPERTIES OF SOFT CLAY

by

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R.H.G. Parry has been Lecturer in Engineering at the University of Cambridge and a Fellow of Pembroke College since 1967. He graduated in Civil Engineering from Melbourne University and continued there to obtain a Master's degree by research. Through the receipt of a Shell Postgraduate Scholarship, he obtained a PhD degree from Imperial College, London, in 1956. After a brief spell with Soil Mechanics Limited in England, he returned to Australia in 1957 to join the CSIRO. From 1960 to 1967, he established and ran Foundation Engineering (Aust) Pty Ltd, and was responsible for projects in Australia, New Guinea and Sarawak.

Since joining Cambridge, Dr Parry has been engaged in research on a number of broad topics, including the strength and deformation of soft clay, and pile behaviour. He has undertaken a wide range of consulting assignments in Britain and overseas, and in recent years he has been concerned with the design of foundations for a number of offshore oil production platforms.

C.P. Wroth has been Professor of Engineering Science and Head of the Department of Engineering Science at the University of Oxford, and a Fellow of Brasnose College, since 1979. He graduated from the University of Cambridge in 1954, having studied both mathematics and engineering. After a year's schoolteaching, he returned to Cambridge to carry out research under the late Professor Roscoe, obtaining a PhD degree in 1958. He then joined G. Maunsell & Partners to gain general experience in civil engineering. In 1961, he was appointed to a Lectureship in Engineering at Cambridge, becoming Reader in Soil Mechanics in 1975. He was a Fellow of Churchill College from 1963 to 1979.

Professor Wroth's main research activities have been the stress-strain behaviour of soils and the development of constitutive relationships for computations of geotechnical boundary value problems. This work led to co-authorship with Professor A.N. Schofield of the well-known book *Critical State Soil Mechanics*. Recently, he has been particularly concerned with the development of a self-boring pressuremeter.

Chapter 4

SHEAR STRESS-STRAIN PROPERTIES OF SOFT CLAY

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4.1 INTRODUCTION

Historically, there have been two distinct classes of calculation in soil mechanics—stability analyses and settlement computations. In the simplest form of stability analyses in saturated clays, the only soil parameter used is the undrained *shear strength* (apart from the unit weight of the soil). As the subject of *soil mechanics* has developed, so has our understanding of undrained shear strength and the recognition that it is not a unique property of a soil, but that it depends on a number of factors such as the consolidation history, the effective stress changes imposed on the soil, and the strain rate. In any such calculation, a failure mechanism is postulated (which needs to be kinematically acceptable), but no further statement or calculation about deformations or strains can be made. In complete contrast, conventional settlement computations are only concerned with deformations and strains, and no account is taken of failure or limiting stresses. The only soil parameters required are those necessary to specify the *deformation characteristics,* such as a pseudo-elastic modulus from undrained triaxial tests or a compressibility modulus from consolidation tests. Unfortunately, but for good reason, these two types of design calculation have been completely divorced from each other; it has been traditional to consider *soil strength* and *soil stiffness* as totally unrelated quantities.

Three developments have now made it possible to carry out complete deformation analyses. These are: (1) the advent of large electronic computers, (2) the development of numerical methods (such as the finite element method), and (3) the establishment of constitutive relationships which provide a realistic description of soil behaviour. These constitutive relationships, or stress-strain laws, embrace information about both soil strength *and* stiffness.

For the past twenty years, the Soil Mechanics Group at Cambridge has been pursuing a long-term programme of research (initiated by the late Professor Roscoe) into the stress-strain behaviour of soils. This work has led to the development of a family of complete mathematical models based on the

critical state concept, which provide a rational framework for: (1) qualitative interpretation of soil behaviour, and (2) complete solutions to boundary value problems. In particular, this approach leads to a rational basis for linking strength with consolidation, and stability with settlement. It is for these reasons that the Authors feel strongly that it is no longer valid or helpful to consider just the shear strength of soft clays (or any soil for that matter), and to put this property into a separate category. The stage has been reached in the development of the subject of geotechnical engineering where the interrelationship between strength, stiffness, consolidation and distortion must be appreciated and exploited. This chapter has been written in this spirit, with the title interpreted in the widest manner.

With the background set regarding the properties to be considered, the next question to be considered is the range of soil covered by the description *soft clay,* and this is discussed in Section 4.2. The distinction between a soft clay and a stiff clay has traditionally been one of undrained shear strength, but an equally satisfactory and valid distinction would be one of degree of overconsolidation. This alternative approach shifts the emphasis from strength to deformation characteristics, distinguishing between the ductile response of normally consolidated soils and the brittle response of heavily overconsolidated clays.

The engineering problems of soft clay of most concern to the design engineer are commonly those during and immediately after construction. This is simply because of the fact that distortion of soft clays under undrained conditions causes excess pore pressure to be generated. Usually, these will be positive (except where the total stresses have been substantially reduced as a result of excavation), and the subsequent consolidation of the clay will increase its strength and stiffness. The most critical case is thus one of undrained loading, which is the central topic of this chapter. The question of settlement is not considered in this chapter, but consideration is given to the response of soft clay in conditions of consolidation and other drained loading in order to establish the concept and existence of *yield* and of *yield loci.*

In order to present these ideas and the experimental evidence in support of them, it is necessary to concentrate on results of high quality tests conducted in the conventional triaxial apparatus, from which the effective stress path for the soil can be derived, and to leave aside the results of vane shear tests and the unsatisfactory empirical correlations that have been proposed for their interpretation.

4.2 NATURAL STATE OF SOFT CLAYS

4.2.1 Formation

The soft clays and silts discussed here are recent alluvial deposits, probably formed within the last 10,000 years. The resulting ground surface will be flat and featureless, except where dissected by rivers or other erosional channels.

In most cases, there will have been no material eroded from the surface except from such channels. Thus, in a geological sense, the bulk of the material may be regarded as normally consolidated but, in fact, in its natural state, this soil will usually exhibit the characteristics of a lightly overconsolidated clay.

A number of factors may give rise to some degree of overconsolidation in the soil, but the most important ones are probably:

(1) changes in static ground water level, and

(2) secondary or delayed consolidation.

Strong overconsolidation effects may be introduced by weathering or desiccation due to evaporation or extraction of moisture by plant roots, but these influences are mostly limited to a thin surface crust a metre or so thick, whereas the main interest of this contribution is below the surface crust. Salts in the pore fluid may also cause a form of apparent overconsolidation through the creation of bonds between particles. The behaviour described in this chapter, however, does not pre-suppose the existence of such bonding.

4.2.2 Influence of ground water changes

A simple cycle of events producing overconsolidation in a clay deposit has been described by Parry (1970) and is shown in Fig. 4.1. During deposition under water, the soil at a point P in Fig. 4.1a will follow curve *1* on the e against σ'_v plot in Fig. 4.1b. Some time after deposition ceases, the water will be drawn down to the top surface of the soil, but this drawdown does not produce any change in effective stress at P. However, the physiographic or climatic factors producing this drawdown may result in the water-table being drawn down below the surface of the soil to a depth z_m. Providing static ground water conditions are reached with the level at z_m, the soil at point P will follow curve *2* in Fig. 4.1b, and the vertical effective stress will attain a maximum value σ'_{vm}. If the ground water level now rises to z_o, the soil will follow curve *3*, and the vertical effective stress at point P becomes σ'_{vo}. The overconsolidation ratio *(OCR)* is then:

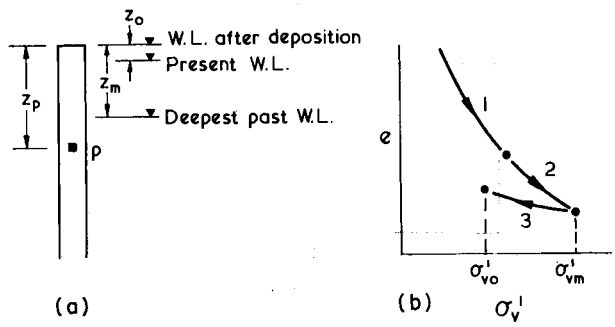

Fig. 4.1. Overconsolidation caused by ground water movements (Parry, 1970).

$$OCR = \sigma'_{vm}/\sigma'_{vo} \tag{4.1}$$

Real soil will have a much more complex history than this, but the important points are the maximum past water-table depth z_m and the existing depth z_o.

The value of the greatest effective vertical stress, σ'_{vm}, at any point such as P (below a depth z_m) in Fig. 4.1a is equal to the total overburden pressure minus the head of water at P, that is:

$$\sigma'_{vm} = \gamma z_p - \gamma_w (z_p - z_m) \tag{4.2}$$

The present value of the effective vertical stress σ'_{vo} is given by:

$$\sigma'_{vo} = \gamma z_p - \gamma_w (z_p - z_o) \tag{4.3}$$

Thus, the overconsolidation ratio is given by:

$$OCR = \frac{\sigma'_{vm}}{\sigma'_{vo}} = \frac{\gamma' z_p + \gamma_w z_m}{\gamma' z_p + \gamma_w z_o} \tag{4.4}$$

where γ is the bulk unit weight, assumed here to be constant, γ' is the buoyant unit weight, and γ_w is the unit weight of water. The soil is assumed to be fully saturated.

4.2.3 Delayed consolidation

The quasi-preconsolidation effect introduced by secondary or delayed consolidation has been discussed by a number of workers, e.g. Leonards & Altschaeffl (1964), Raymond (1966) and Bjerrum (1967). Figure 4.2 is repro-

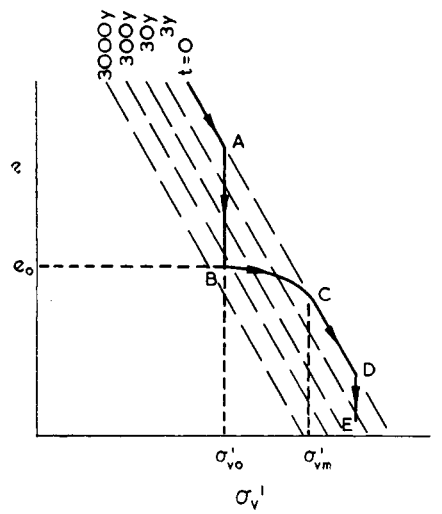

Fig. 4.2. Apparent overconsolidation caused by delayed consolidation (Bjerrum, 1967).

duced from the paper by Bjerrum and shows a system of consolidation curves representing different periods of consolidation. Thus, under laboratory conditions of thin oedometer specimens, the primary consolidation is comparatively fast, being complete in a day or so, and corresponds closely to the 'instant' (t=O) curve. If, however, the applied stress is held constant for a long period of time, further consolidation takes place at this constant pressure, σ'_{vo}, and the state of the sample moves from A to B, crossing the delayed consolidation lines.

The same behaviour occurs after sedimentation in the field, and the soil will attain a state such as (e_o, σ'_{vo}) at point B after a period of say 1000 years. On loading in the laboratory, this soil will show very little consolidation until the applied pressure reaches σ'_{vm} at point C, where the curve breaks to join the instant consolidation line. In the terminology of plasticity, this break point indicates yielding in the soil. The apparent overconsolidation ratio is given by $\sigma'_{vm}/\sigma'_{vo}$.

4.3 COMPARISON OF OVERCONSOLIDATED CONDITIONS

It has been shown above that the *OCR* (or apparent *OCR*) arising from ground water changes and delayed consolidation produces a soil which, on loading, shows on a plot of e against log σ'_v a break point or yield point. If the soil condition resulting from these two influences is identical in all respects, it is irrelevant which is the true cause regarding the calculation of pore pressures under additional surface loading. In fact, it is not known at the moment if the soil conditions are the same or not, but some observations may be made as below.

4.3.1 Magnitudes of overconsolidation ratio

The magnitudes of *OCR* produced by the two effects of ground water changes and delayed consolidation seem to be much the same (Fig. 4.3); Bjerrum (1973) gave values ranging from 1.25 to 1.8 for a number of soils. With reference to Eq. 4.4, if z_m=2 m, z_o=0.5 m and γ=18 kN/m^3 (typical values) then, at a depth z_p=2 m:

$$OCR = \frac{8 \times 2 + 10 \times 2}{8 \times 2 + 10 \times 0.5} = 1.7$$

which is similar to the higher magnitudes given for delayed consolidation. Below z_p=2 m, the values will be less than 1.7, and above this depth values will exceed 1.7. This distribution of values with depth is discussed below.

4.3.2 Variation of OCR with depth

The variation of *OCR* with depth is given by Eq. 4.4 for ground water movement. This applies below the past maximum depth of the water-table,

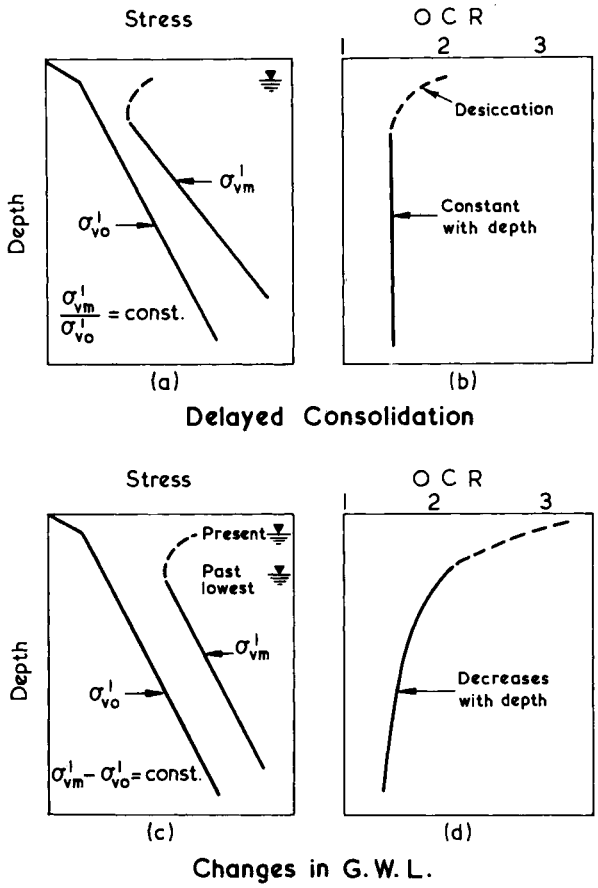

Fig. 4.3. Variations of σ'_{vm} and *OCR* with depth from delayed consolidation and ground water movements.

z_m, if it is assumed that the effects of desiccation have not extended below this depth, which probably corresponds closely with the level of minimum undrained shear strength (Parry, 1970). Above depth z_m, the effects of desiccation have probably been present, producing the variation in *OCR* shown in Fig. 4.3d, i.e. higher values than given by Eq. 4.4. In contrast to the decrease in *OCR* with depth occasioned by changes in ground water level, the apparent *OCR* due to delayed consolidation can be expected to be approximately constant with depth, although this may depend to some extent on the age of the deposit and its rate of formation.

Expressed in another way, the value $(\sigma'_{vm} - \sigma'_{vo})$ for ground water change will be constant below z_m, and will increase in the desiccated zone above z_m, whereas this value should increase below the desiccated zone if caused by delayed consolidation, as shown in Fig. 4.3a. The scatter of experimental data,

however, usually masks these differences. Cases where $(\sigma'_{vm}-\sigma'_{vo})$ is virtually constant with depth and cases where $(\sigma'_{vm}-\sigma'_{vo})$ increases with depth have been published, and it seems likely that in most deposits of soft clay both factors have contributed to produce a light degree of overconsolidation.

4.3.3 Value of K_0

An important difference between the two primary causes of true or apparent overconsolidation may be in the ratio, K_0, of horizontal stress, σ'_{ho}, to vertical stress, σ'_{vo}, although insufficient is known at present about delayed consolidation to be sure how great the difference may be.

Values of K_0 due to stress change have been measured on clays in the laboratory, and it is known that for normally consolidated soils (*instant* consolidation) the value of K_0 (K_{nc}) corresponds quite closely to:

$$K_{nc} = 1 - \sin \varphi' \tag{4.5}$$

which is a simplification of an equation suggested by Jàky (1948). The values increase with increasing overconsolidation.

It has been shown (Wroth, 1975) that for a lightly overconsolidated ($OCR<5$) clay the value of K_0 can be reasonably approximated by assuming isotropic elastic behaviour for the soil. The expression obtained for K_0 is:

$$K_0 = OCR \cdot K_{nc} - \frac{\upsilon'}{1-\upsilon'}(OCR-1) \tag{4.6}$$

where υ' is Poisson's ratio in terms of effective stress. Figure 4.4 shows experimental values of υ' plotted against plasticity index for a number of different soils (Wroth, 1975).

A comparison of computed and laboratory measured values of K_0 during one-dimensional swelling from a normally consolidated state is shown for Boston Blue Clay in Fig. 4.5 (Wroth, 1975) and for kaolin (Nadarajah, 1973) in Fig. 4.6. For example, a soil which has an angle of friction $\varphi'=20°$ and a plasticity index of 40%, would give $K_0=0.66$ for $OCR=1$ from Eq. 4.5. Substitution of this value into Eq. 4.6, together with $\upsilon'=0.35$ from Fig. 4.4, gives:

$$K_0 = 0.54 + 0.12\ OCR$$

and, hence, yields the values:

OCR	K_0
1	0.66
2	0.78
3.8	1.0
5	1.14

Thus, for the soft lightly overconsolidated soils considered here, values of K_0 in the field, for which the OCR will be commonly between 1.0 and 2.5, may be expected to range from about 0.6, for almost normally consolidated

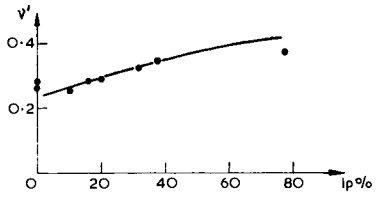

Fig. 4.4. Variation of effective stress Poisson's ratio with plasticity index (Wroth, 1975).

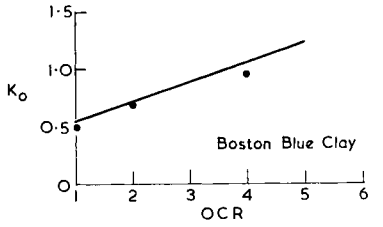

Fig. 4.5. Theoretical and observed relationships between K_0 and OCR for Boston Blue Clay (Wroth, 1975).

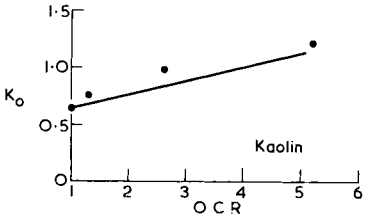

Fig 4.6 Theoretical and observed relationships between K_0 and OCR for kaolin.

soil with low plasticity, to nearly unity, for soils of high plasticity with OCR up to 2.5. Cylic loading, such as alternating ground water level, will of course influence these values, but the range is still probably correct. In the absence of contrary evidence, it must be assumed that delayed consolidation will not lead to values of K_0 in excess of those for normally consolidated soils, and these values will usually be between 0.5 and 0.7.

Values of K_0 in soft clays have been published by a number of workers using such methods as hydraulic fracture, total pressure cells and the pressuremeter, but the accuracy of some of these is in question. However, measured values generally range from 0.5 to 1.2 (Berre & Bjerrum, 1973; Wroth and Hughes, 1973; Massarsch et al, 1975; Tavenas et al, 1975) and are commonly around 0.8 to 1.0.

4.4 YIELD

4.4.1 Definition of yield

Soils, in common with a number of engineering materials, exhibit a stress-deformation response as shown in Fig. 4.7. Unloading at a point such as B will produce a curve BC, with the closed stress cycle A to B, B to C causing a permanent, or plastic, deformation AC. On reloading from C, a path similar to BC is retraced, but with a small hysteresis loop, until a point D, close to B, is reached. At D, an abrupt change in the stress path is observed, subsequent to which the extended virgin loading curve is followed. The point D indicates yield of the material for the reloading curve CDE; it marks a major change in

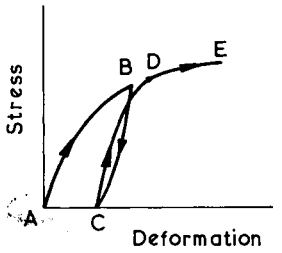

Fig. 4.7. Typical stress-deformation response of a work-hardening plastic material.

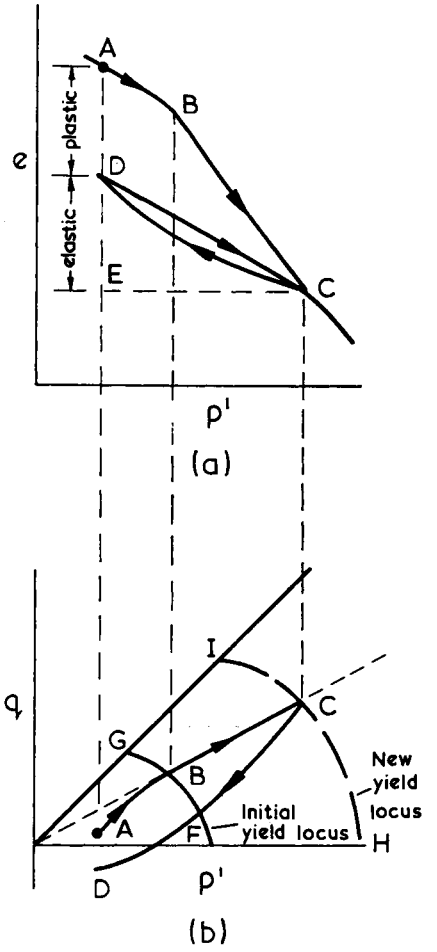

Fig. 4.8. Consolidation curves, stress paths and yield in one-dimensional consolidation.

behaviour in that the local tangent stiffness (or modulus) is greatly reduced and further plastic strains are induced. The importance of yield will become evident later in this chapter. It should be emphasised that *yield* is not the same as *failure:* depending on the precise stress history, a specimen may indicate a succession of yield states before (or after) it fails in the sense of maximum shear stress or maximum stress ratio.

4.4.2 Yield in one-dimensional consolidation

When a soft clay or silt is consolidated under conditions of zero lateral strain, the initial compressibility (AB in Fig. 4.8a) is fairly small and largely reversible, until the soil reaches a yield condition at B corresponding to the preconsolidation pressure. This portion AB of the behaviour may be regarded as non-linear elastic, with a variable elastic modulus depending on the mean effective stress, p'. After the yield point, greater compressibility is experienced

and, to a large extent, the deformation is irreversible. In this phase (*BC* in Fig. 4.8a) the soil is undergoing plastic strain-hardening during which a new yield condition is being continually created. During the process of normal consolidation from *B* to *C*, the ratio of the principal effective stresses, K_o, is constant, so that the corresponding path is a straight line in the plot* of deviator stress, $q=(\sigma_1' - \sigma_3')$, against mean effective stress, $p' = \frac{1}{3}(\sigma_1' + \sigma_2' + \sigma_3')$, in Fig. 4.8b.

If at a point such as *C* the soil is unloaded one-dimensionally, it follows curve *CD* in Figs. 4.8a and 4.8b, and the state of the soil moves inside the new yield locus represented by *HCI*. Elastic volumetric strains are experienced in this phase with essentially the same elastic properties as on first loading, *AB*. Thus, in Fig. 4.8a, *AD* represents plastic, irreversible volumetric strain, whereas *DE* represents elastic and reversible volumetric strain. On reloading from *D*, *C* is the new yield point.

It is convenient at this point to look at the consolidation behaviour of soft, lightly overconsolidated natural clays in some detail. In order to study possible effects of sample orientations within the clay deposit, the behaviour of both vertical and horizontal samples will be examined.

In Fig. 4.9, settlements or voids ratios have been plotted against consolidation pressure (log scale) for soft soils from three sites: Fulford (Parry & Nadarajah, 1974), Canvey Island (George & Parry, 1973; Parry & Nadarajah, 1974) and Mucking Flats (Wesley, 1975). In Fig. 4.10, the values of the coefficient of volume compressibility, m_v, have been plotted against consolidation pressure. All three soils show very similar behaviour, viz:

(1) For stress increments in the pre-yield or elastic phase just in excess of the field stresses, the compressibility, m_v, in the horizontal direction is almost double that in the vertical direction.

(2) The maximum compressibility is higher in the vertical direction than in the horizontal direction and is reached at a stress slightly higher than the vertical preconsolidation pressure. The maximum compressibility in the horizontal direction is reached at a stress approximating to the vertical preconsolidation pressure.

(3) A yield or preconsolidation pressure is more readily distinguished from the vertical specimen than from the horizontal specimen.

(4) As the consolidation pressure is increased beyond yield, the compression curves for the vertical and horizontal specimens approach each other and eventually become identical. The two curves merge at a consolidation pressure equal to about 2.5 to 3 times the preconsolidation pressure;

* When the behaviour of a material is assumed to be isotropic (which is the case here) it is preferable to use parameters of stress and strain that are derived from invariants. The arguments in favour of this and the particular choice of p' and q are presented by Schofield & Wroth (1968). It should be noted that the definitions of p' and q differ from those in use at the Massachusetts Institute of Technology.

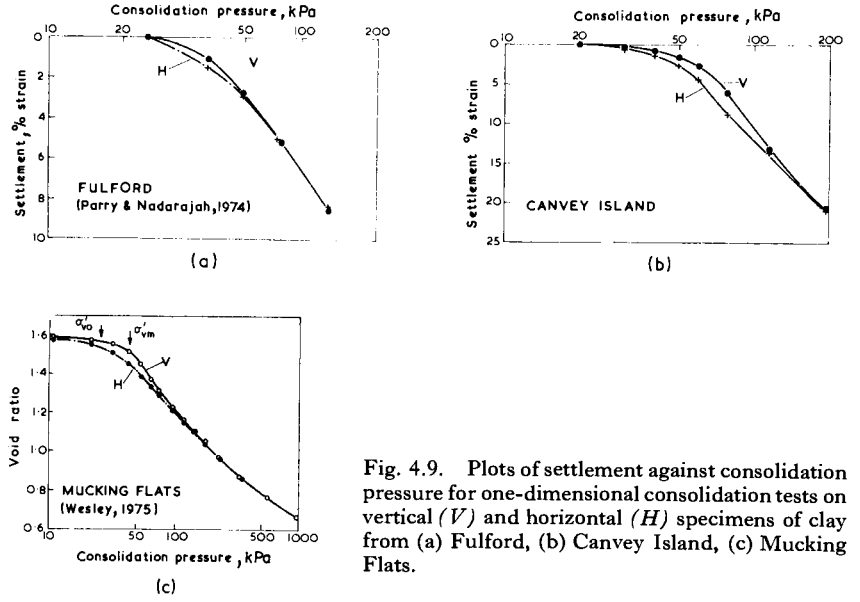

Fig. 4.9. Plots of settlement against consolidation pressure for one-dimensional consolidation tests on vertical *(V)* and horizontal *(H)* specimens of clay from (a) Fulford, (b) Canvey Island, (c) Mucking Flats.

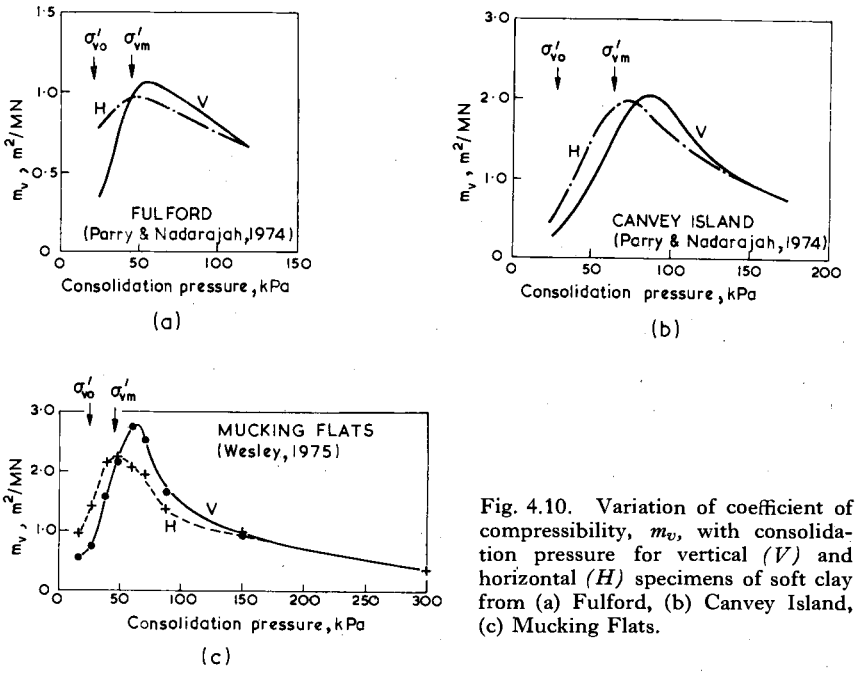

Fig. 4.10. Variation of coefficient of compressibility, m_v, with consolidation pressure for vertical *(V)* and horizontal *(H)* specimens of soft clay from (a) Fulford, (b) Canvey Island, (c) Mucking Flats.

that is, at this pressure the effects of initial structural anisotropy have been overcome by the applied pressure. This corresponds with previous findings at Cambridge in tests on laboratory prepared specimens of kaolin. These are initially prepared as a slurry, poured into a mould, and then subjected to a one-dimensional consolidation pressure sufficient to give the sample adequate strength to be self-supporting. It has been found (Balasubramaniam, 1969) from radiographic observations on embedded lead shot that an isotropic consolidation pressure of about three times the initial one-dimensional pressure is required before the initial anisotropy is overcome and the soil behaves isotropically.

4.4.3 Yield and failure for other stress paths

Yielding may be experienced not only during one-dimensional consolidation but also for other applied stress paths. A lightly overconsolidated soil at point A in Fig. 4.11 can be regarded as occupying a position within an elastic zone bounded by the failure line $q=Mp'$ and the yield locus shown. Any volumetric strains that occur for stress changes within the elastic zone are small and reversible.

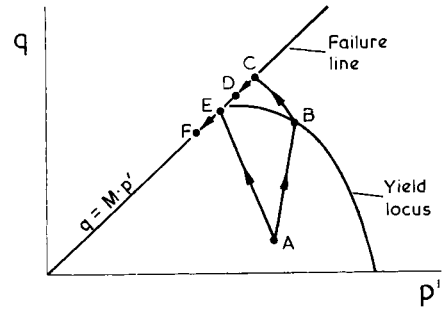

Fig. 4.11. Possible stress paths, yield locus and failure line for a soft clay.

A soil element following an effective stress path such as AEF will theoretically exhibit elastic volumetric strains without yield until it reaches failure at E, after which it may soften to F. An element following path $ABCD$ yields at B before reaching failure, the sequence of behaviour being: $A \rightarrow B$ elastic volumetric strains, B yield, $B \rightarrow C$ plastic (possibly with work-hardening), C failure, and $C \rightarrow D$ softening (which may occur after failure).

4.4.4 Evidence for yield locus from drained tests

The break point or yield condition for lightly overconsolidated clay in one-dimensional consolidation is well known, but it is not so well recognised for other types of loading. Evidence of yield for other types of loading will now be presented.

A series of tests on laboratory prepared, lightly overconsolidated specimens of kaolin have been reported by Parry & Nadarajah (1973, 1974). All samples were consolidated one-dimensionally to an *OCR* of 1.6 at state *P* in Fig. 4.12. The four types of triaxial test carried out were:

(1) drained compression test *PQR*, keeping σ_1' constant and reducing σ_3',
(2) drained compression test *PST*, keeping σ_3' constant and increasing σ_1',
(3) reconsolidation along the one-dimensional swelling line *PUVW*, and
(4) deviator stress removed and the sample isotropically consolidated *(PYZN)*

where σ_1' is the axial effective stress and σ_3' the radial effective stress. In Fig. 4.13, the volumetric strain, ε_v, is plotted against q or p' for the four tests, and yield points are located by simple straight-line extrapolations of portions of the curves with different slopes. The resulting volumetric yield locus *ZUSQ* is shown in Fig. 4.12.

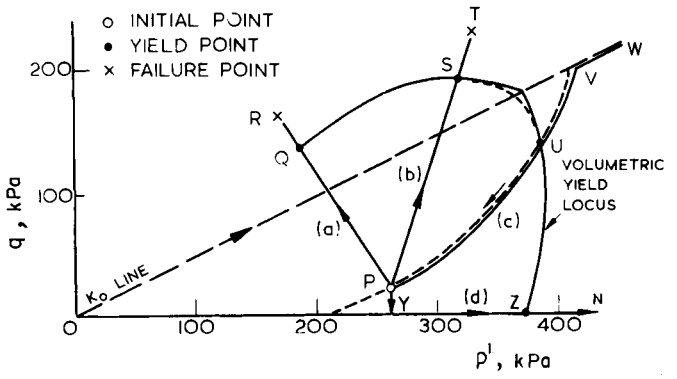

Fig. 4.12. Stress paths, volumetric yield locus and failure points for drained tests on kaolin.

Mitchell (1970) conducted a series of drained triaxial tests on undisturbed specimens of Leda clay taken from a single block sample in which the cell pressure and axial load changes were applied simultaneously to give a constant ratio $\eta = q/p'$. The resulting plots for vertically trimmed specimens of void ratio against p', with straight-line approximations, are shown in Fig. 4.14. Yield points are taken as the intersections of the straight lines and are reproduced in Fig. 4.15 in a q vs p' plot, giving a distinct yield locus.

Further evidence for the existence of the volumetric yield locus has been presented for an undisturbed soft silty clay (Pender et al, 1975). Specimens from one block sample were submitted to various stress paths under drained conditions. A typical plot of q against ε_v is shown in Fig. 4.16, and the yield point is indicated. The resulting yield locus is shown in Fig. 4.17 as obtained

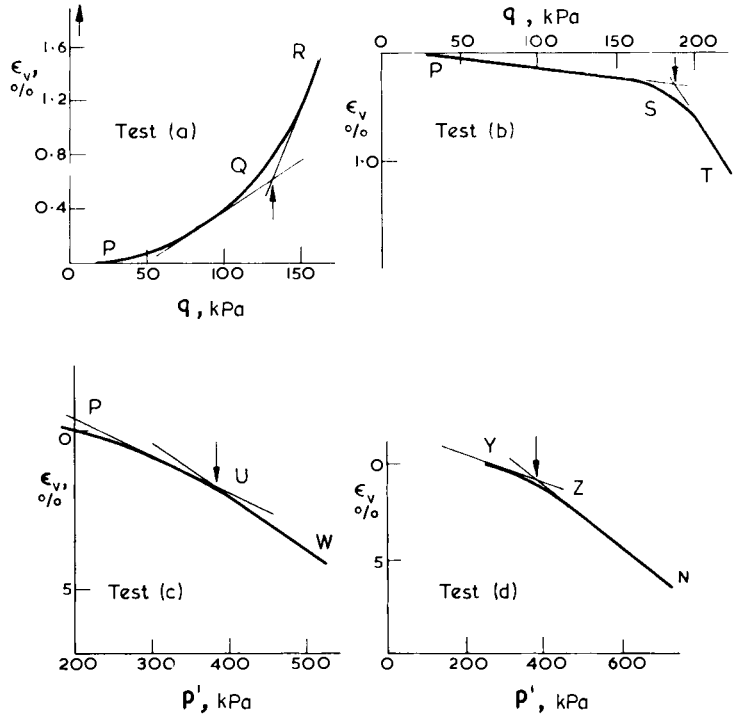

Fig. 4.13. Yield points in drained tests on kaolin.

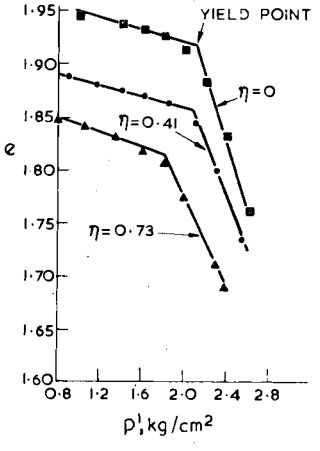

Fig. 4.14. Yield points in drained tests on Leda Clay (Mitchell, 1970).

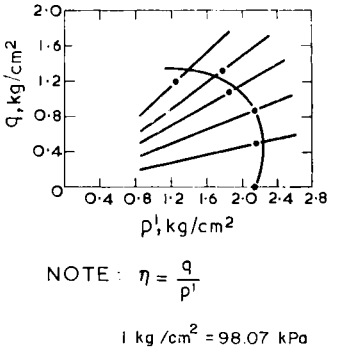

Fig. 4.15. Volumetric yield locus for Leda Clay (Mitchell, 1970).

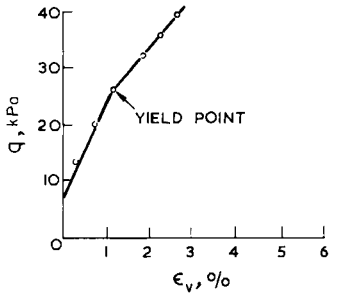

Fig. 4.16. Typical yield point in drained tests on Canvey Island clay (Pender et al, 1975).

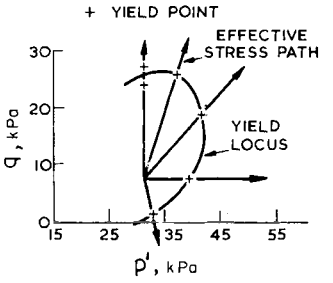

Fig. 4.17. Yield locus for Canvey Island clay (Pender et al, 1975).

from the various stress paths. This was used in making an analysis of the field behaviour, which is further discussed in a later section.

It will be noted that there is similarity in the shapes of the yield loci in Figs. 4,12, 4.15 and 4.17.

4.4.5 Evidence for yield locus from undrained tests

Excess pore pressures which develop during undrained tests are governed by the volumetric strain characteristics of the soil. Thus, a yield condition would be indicated by a sharp increase in pore pressure. Surprisingly little work appears to have been done on this aspect of soil behaviour, but some confirmation of this expected behaviour can be seen in results for undrained triaxial compression tests on lightly overconsolidated laboratory prepared kaolin (Parry & Nadarajah, 1974).

The effective stress paths for three different tests are shown in Fig. 4.18. Sample *A* was consolidated under zero axial strain conditions (by applying a predetermined ratio $\sigma_3'/\sigma_1'=K_0$) to $\sigma_1'=550$ kPa, then allowed to swell under zero lateral strain conditions to an *OCR* of 1.25. Samples *B* and *C* were isotropically consolidated to 413 kPa and 550 kPa respectively, then allowed to swell isotropically to give an *OCR* of 1.3 in each case.

The pore pressure response is plotted against change in deviator stress, $q-q_0$, in Fig. 4.19 for samples *A* and *B* (the response for *C* was geometrically similar to *B*). A sharp increase in pore pressure response can be seen for sample *A* at $q-q_0=160$ kPa, indicating a distinct yield condition. Sample *B* does not show such a distinct change but the pore pressure response does increase markedly with increasing $q-q_0$, particularly for $q-q_0$ greater than 85 kPa, which is probably a reasonable approximation to the yield point. The broken line gives the theoretical response for an isotropic elastic material, as discussed in a later section. It can be seen that the early response for both tests is fairly close to the elastic response.

The approximate yield points are indicated on the stress paths in Fig. 4.18. The broken lines show the theoretical isotropic elastic response and, again, it

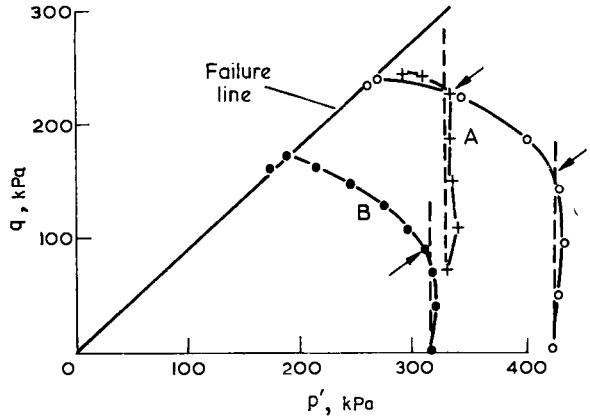

Fig. 4.18. Stress paths from undrained tests on kaolin.

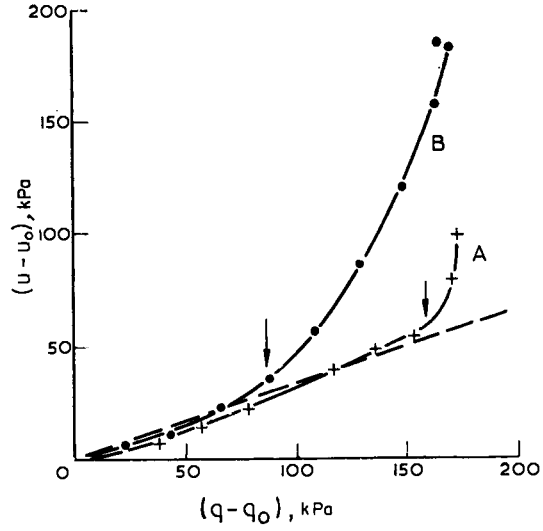

Fig. 4.19. Pore pressure changes for undrained tests on kaolin indicating probable yield points.

can be seen that the actual response early in each test corresponds approximately to elastic behaviour.

4.5 GENERAL IDEALISED UNDRAINED BEHAVIOUR

4.5.1 The three phases

In Fig. 4.20a, effective stress and total stress paths are shown for the three phases envisaged in the deformation of an element of soft clay under vertical loading (such as an embankment or oil tank) and undrained conditions. The

initial pore pressure is u_0. Because the horizontal intercept between the total stress and effective stress paths is the pore pressure, u, the pore pressure response with applied pressure is of the type given in Fig. 4.20b. The three basic phases are:

(1) An elastic phase, in which pore pressures are substantially recoverable when loading is reversed.

(2) A plastic phase, in which some work hardening may occur.

(3) Either: (a) A state of contained failure for soil with undrained stress-strain behaviour, as shown in Fig. 4.20c. In this phase, additional total stress may be applied to the soil providing it is isotropic and q is not increased.
or: (b) A state of contained failure for soil with undrained stress-strain behaviour, as shown in Fig. 4.20d. In this phase, additional total stress may be applied to the soil providing q is reduced in accordance with the additional strains undergone by the soil.

The pattern of behaviour is essentially developed from critical state concepts, with the failure line for normally and lightly overconsolidated clays coinciding

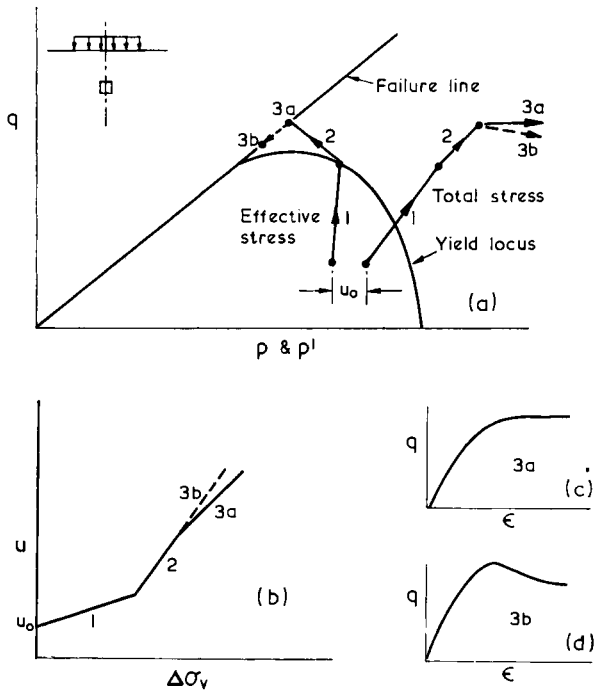

Fig. 4.20. (a) Idealised field stress paths, (b) pore pressure response; (c) & (d) stress-strain curves.

with the critical state line. The shape of the yield locus is similar to that for the *Cam-clay model.* This model was proposed on the basis of triaxial test data with the behaviour in the elastic phase being assumed to be isotropic. In considering natural soft clay deposits, the possibility of anisotropic behaviour must be recognised.

4.5.2 Phase 1 : elastic behaviour

4.5.2.1 Fundamentals

It will be assumed here that the soil is cross-anisotropic, with the vertical axis being the axis of radial symmetry. If, for convenience, the vertical direction is denoted by a suffix a and the horizontal directions by suffixes b and c, all these being principal directions of stress and strain, then:

(1) E'_a; $E'_b = E'_c$ are the Young's moduli in terms of effective stress,
(2) $E'_c/E'_a = n'$, and
(3) υ'_{bc} is the ratio of strain in direction c to an applied strain in direction b and vice versa, υ'_{ca} is the ratio of strain in direction a to an applied strain in direction c, and υ'_{ac} is the ratio of strain in direction c to an applied strain in direction a.

From superposition:

$$\frac{\upsilon'_{ca}}{\upsilon'_{ac}} = \frac{E'_c}{E'_a} = n'$$

and the linear strains, ε_a, ε_b and ε_c, are given by:

$$\varepsilon_a = \frac{\Delta\sigma'_a}{E'_a} - \frac{\upsilon'_{ca}}{E'_c}(\Delta\sigma'_b + \Delta\sigma'_c) \tag{4.7a}$$

$$\varepsilon_b = -\upsilon'_{ac}\frac{\Delta\sigma'_a}{E'_a} + \frac{\Delta\sigma'_b}{E'_c} - \upsilon'_{bc}\frac{\Delta\sigma'_c}{E'_c} \tag{4.7b}$$

$$\varepsilon_c = -\upsilon'_{ac}\frac{\Delta\sigma'_a}{E'_a} - \upsilon'_{bc}\frac{\Delta\sigma'_b}{E'_c} + \frac{\Delta\sigma'_c}{E'_c} \tag{4.7c}$$

Under undrained conditions, for a saturated soil it can be assumed that the volumetric strain, ε_v, is zero. Thus:

$$\varepsilon_v = \varepsilon_a + \varepsilon_b + \varepsilon_c = 0 \tag{4.8}$$

The above equations allow pore pressure changes and effective stress paths to be determined for different applied total stress conditions. Effective stress paths will be defined in terms of their gradients $\Delta q/\Delta p'$ and $\Delta t/\Delta s'$, while pore pressure changes will be in terms of the parameter A (Skempton, 1954), where:

$$\Delta u = B\{\Delta\sigma_c + A(\Delta\sigma_a - \Delta\sigma_c)\} \tag{4.9}$$

and B is assumed to be unity. Note that $t = (\sigma'_a - \sigma'_c)/2$ and $s' = (\sigma'_a + \sigma'_c)/2$. Both triaxial conditions and plane strain conditions will now be considered.

4.5.2.2 Triaxial conditions

Under triaxial conditions, if direction a is assumed to be vertical and to correspond to the axis of the cylindrical specimen:

$$\Delta\sigma'_b = \Delta\sigma'_c \tag{4.10}$$

From this relationship, together with $\varepsilon_v = 0$, it has been shown (Parry & Wroth, 1976) that:

$$\frac{\Delta q}{\Delta p'} = \frac{3}{2} \cdot \frac{2\upsilon'_{bc} + 4n'\upsilon'_{ac} - n' - 2}{\upsilon'_{bc} - n'\upsilon'_{ac} + n' - 1} \tag{4.11}$$

It can similarly be shown that:

$$\frac{\Delta t}{\Delta s'} = \frac{2\upsilon'_{bc} + 4n'\upsilon'_{ac} - n' - 2}{2\upsilon'_{bc} + n - 2} \tag{4.12}$$

and:

$$A = \frac{n'\,(1 - 2\upsilon'_{ac})}{n'\,(1 - 4\upsilon'_{ac}) + 2(1 - \upsilon'_{bc})} \tag{4.13}$$

Adoption of the assumtion made by Henkel (1971) that:

$$\upsilon'_{bc} = \tfrac{1}{2}\,(\upsilon'_{ac} + \upsilon'_{ca}) = \tfrac{1}{2}\,\upsilon'_{ac}\,(1 + n') \tag{4.14}$$

and with $\upsilon'_{bc} = 0.2$, gives values of $\Delta q/\Delta p'$, $\Delta t/\Delta s'$ and A shown in Table 4.1. The value of υ'_{bc} does not have a marked effect on $\Delta q/\Delta p'$, $\Delta t/\Delta s'$ and A. It will be noted that, for $n' = 1$, the familiar value of $A = \frac{1}{3}$ is obtained, and the effective stress path is a vertical straight line on a q—p' plot and a line with a gradient 3:1 on a t—s' plot.

Table 4.1. Values of stress ratio and parameter A for triaxial and plain strain conditions

n'	Triaxial conditions			Plane strain conditions		
	$\Delta q/\Delta p'$	$\Delta t/\Delta s'$	A	$\Delta q^*/\Delta p'$	$\Delta t/\Delta s'$	A
0.5	5.4	1.4	0.15	6.5	2.3	0.3
1.0	∞	3.0	0.33	∞	∞	0.5

4.5.2.3 Plane strain conditions

Under plane strain conditions, if direction a is assumed vertical and zero strain occurs in direction b:

$$\varepsilon_b = 0 \tag{4.15}$$

By the use of this relationship together with $\varepsilon_v = 0$, it has been shown (Parry & Wroth, 1976) that, with $q^* = (\sigma_a - \sigma_b)$:

$$\frac{\Delta q}{\Delta p'} = \frac{3\,(1 + n' - 2n'\upsilon'_{ac} - 2n'\upsilon'_{ac}\upsilon'_{bc} - n'^2\upsilon'^{\,2}_{ac} - \upsilon'^{\,2}_{bc})}{(1 - n' + n'\upsilon'_{ac} - n'\upsilon'_{bc} + n'\upsilon'_{ac}\upsilon'_{bc} - \upsilon'^{\,2}_{bc})} \tag{4.16}$$

It can similarly be shown that:

$$\frac{\Delta t}{\Delta s'} = \frac{1 + n' - 2n'\upsilon'_{ac} - 2n'\upsilon'_{ac}\upsilon'_{bc} - n'^2\upsilon'^{\,2}_{ac} - \upsilon'^{\,2}_{bc}}{1 - n' - n'^2\upsilon'^{\,2}_{ac} - \upsilon'^{\,2}_{bc}} \tag{4.17}$$

and:

$$A = \frac{n'\,(1 - n'\upsilon'^{\,2}_{ac} - \upsilon'_{ac} - \upsilon'_{ac}\upsilon'_{bc})}{1 + n' - 2n'\upsilon'_{ac} - 2n'\upsilon'_{ac}\upsilon'_{bc} - n'^2\upsilon'^{\,2}_{ac} - \upsilon'^{\,2}_{bc}} \tag{4.18}$$

Again, adoption of the assumptions $\upsilon'_{bc} = \frac{1}{2}\,\upsilon'_{ac}\,(1 + n')$ and $\upsilon'_{bc} = 0.2$, gives values of $\Delta q^*/\Delta p'$, $\Delta t/\Delta s'$ and A shown in Table 4.1. It will be noted that, for $n' = 1$, $A = 0.5$ and the effective stress path is vertical on both a $q-p'$ plot and a $t-s'$ plot.

4.5.2.4 Effective stress paths

For an element of clay at some depth which is responding elastically to some loading on the ground surface, the (local) increments of total vertical and horizontal stress will be proportional to one another. In general, $\Delta\sigma_h = l_1\Delta\sigma_v$, where l_1 is a constant, the value of which is governed by the geometry of the problem being considered. The resulting effective stress paths and plots of Δu against vertical applied stress $\Delta\sigma_v$ are shown in Fig. 4.21, where:

$$\Delta u = \Delta\sigma_v\,\{l_1 + A\,(1 - l_1)\} \tag{4.19}$$

In a simple undrained triaxial compression test with constant cell pressure, the relationship:

$$\Delta u = \tfrac{1}{3}\,\Delta q$$

should hold for isotropic elastic soil. This relationship is plotted as a broken line in Fig. 4.19, and it can be seen that it is a fair representation of the pore pressure change early in these tests. In fact, in the initial part of each test, the pore pressure response is less than the isotropic elastic value, possibly indicating that the soil is stiffer vertically than laterally in both samples, although one was consolidated isotropically.

The presence of decaying organic matter in many soft clay deposits may lead to small amounts of gas in the pore fluid and, thus, the degree of saturation, S, is less than unity. This would cause a decrease in the value of A and a tilt of the effective stress path, as shown in Fig. 4.22.

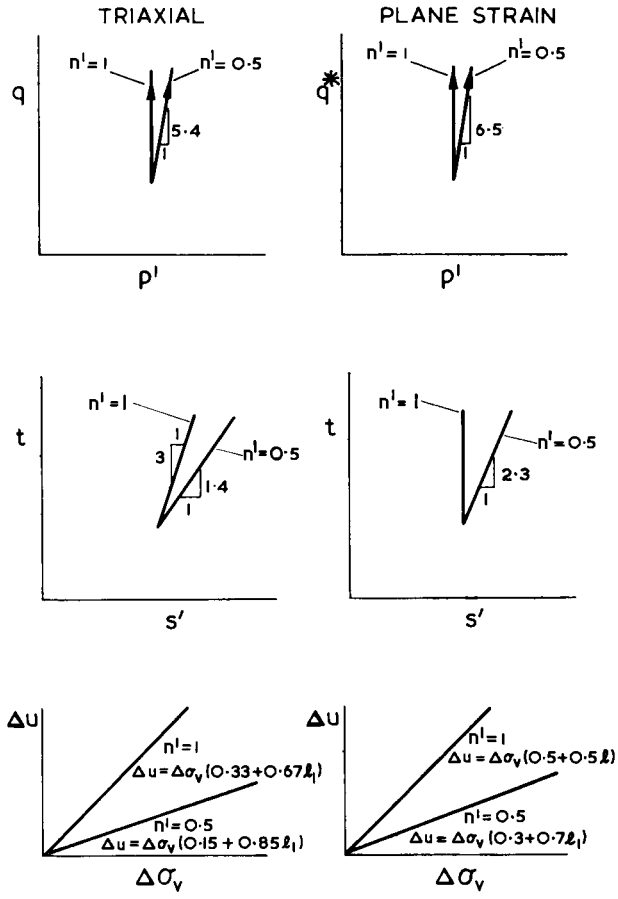

Fig. 4.21. Effective stress paths and pore pressure response for elastic behaviour.

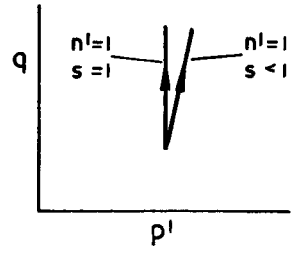

Fig. 4.22. Effect of gas in pore fluid on effective stress path.

4.5.3 Influence of sample orientation on elastic behaviour in triaxial compression and extension

It has been shown in the previous section that, if a soil behaves as an isotropic linear material, then the effective stress path on a q vs p' plot, such as that in Fig. 4.21, is a vertical line. This is true for both compression and extension. If the lateral stiffness differs from the vertical stiffness, an inclined line will result, such as that for $n'=0.5$ in Fig. 4.21.

Anisotropic behaviour in clay may arise from two sources:

(1) *structural anisotropy*, such that the linear strain in any direction produced by a principal effective stress increment in that direction will differ with the direction in the soil, and

(2) *stress anisotropy*, such that the linear strain produced in any direction by an increase in principal compressive effective stress in that direction will be different in magnitude to the linear strain produced by a corresponding decrease in principal compressive effective stress in that direction.

In an undrained triaxial compression test on soft clay, both forms of anisotropy will influence the behaviour because an increase in axial total stress will produce a positive increment of pore pressure. As a result, there is an increase in axial effective stress but a decrease in radial effective stress.

The stress paths considered here will be those for tests on cylindrical triaxial test specimens with an initial isotropic effective stress; this is usually the condition for undisturbed clay samples before testing. The conclusions, however, will be valid even where initial stress conditions are not isotropic, so long as the soil behaves in an elastic manner.

If it is assumed for the moment that in any particular direction the moduli are the same for increasing and decreasing stress, then the effective stress paths for $n'<1$ and $n'>1$ are straight lines lying on either side of the vertical ($n'=1$) line on a $q-p'$ plot. These paths are continuous into compression ($q>0$) and extension ($q<0$) space. It is useful to look at actual stress paths, and those for $n'=0.5$ and $n'=2.0$ are shown in Fig. 4.23a as NLS and QLU respectively for $\upsilon'_{bc}=0.2$. The gradients of these lines are $NLS=5.4$ and $QLI=-7.5$. The total stress path for conventional triaxial compression and extension tests has a gradient of 3 and is represented by MLR. This total stress path is based on the assumption that the initial pore pressure is zero.

If a triaxial specimen is removed from a soil deposit so that its axis corresponds to the vertical direction in the field, and the lateral stiffness in the field is half the vertical stiffness, then undrained compression and extension tests will give effective stress paths LN and LS respectively. A horizontal specimen, however, would not give stress paths LQ or LU, as two of the radial directions mutually orthogonal in the specimen will have stiffnesses corresponding to the vertical and horizontal field values. This will lead to a steeper stress path.

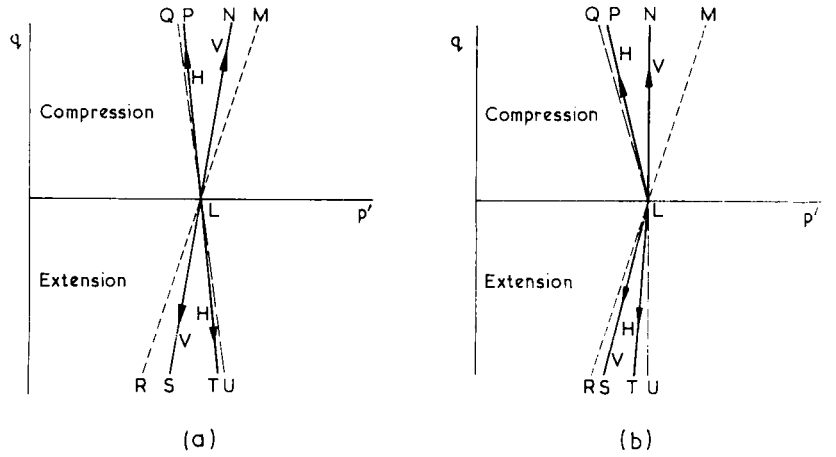

Fig. 4.23. Theoretical stress paths in an elastic phase for compression and extension on vertical *(V)* and horizontal *(H)* specimens assuming (a) vertical modulus in compression twice the horizontal value, and equal moduli for increasing and decreasing stress, (b) vertical modulus in compression twice the horizontal value, and modulus for decreasing stress twice that for increasing stress.

For the purposes of illustration, it will be assumed here that the two radial values of stiffness can be averaged to give $n'=1.5$; this is not strictly correct but will give a stress path very close to the true theoretical one. The resulting stress path is *PLT* with a gradient of -9.9. Thus, the relevant effective stress paths are *NLS* for a vertical specimen and *PLT* for a horizontal specimen.

Now consider the case where, in any particular direction, the modulus for increasing principal effective stress is different to that for decreasing principal effective stress. If the modulus for decreasing effective stress is greater than for increasing effective stress, this will have the effect of rotating the effective stress paths in compression space anticlockwise, and in extension space clockwise. The opposite will occur if the modulus for decreasing effective stress is less than for increasing effective stress.

Again it is instructive to take a numerical example, and it will be assumed that, in any direction, the modulus for decreasing effective stress is twice that for increasing effective stress. If $n'=0.5$ for increasing effective stress both axially and radially, then $n'=1$ for undrained axial compression, and $n'=0.25$ for undrained axial extension. If $n'=2.0$ for increasing effective stresses both axially and radially, then $n'=4$ for undrained axial compression, and $n'=1$ for undrained axial extension. The resulting stress paths are *NLS* and *QLU* in Fig. 4.23b, where the lettering of corresponding stress paths is retained from Fig. 4.23a. It can be seen, as stated above, that the effective stress paths have rotated anticlockwise in the compression space and clockwise in the extension space, so that *NL* and *LU* are now vertical, *LQ* has a gradient of -3.5 and *LS* has a gradient of $+4.0$.

The stress paths LN and LS correspond to those for undrained triaxial compression and extension tests respectively on a sample taken vertically in a soil deposit where $n'=0.5$ for effective stresses increasing in both directions. However, LQ and LU will not correspond exactly to the stress paths for horizontal specimens because of lack of symmetry of radial stiffness. If 'average' stiffness is used in the radial direction to obtain some indication of actual stress paths, the values of n' will be $n'=3$ in compression, and $n'=0.75$ in extension. The stress paths for these values of n' are LP and LT in Fig. 4.23b, with gradients of -4.2 and $+14.3$ respectively. Thus, the stress path LT is now just fractionally off vertical. The relevant stress paths, then, are: LN for compression of a vertical sample, LS for extension of a vertical sample, LP for compression of a horizontal sample, and LT for extension of a horizontal sample.

There is little published evidence of undrained triaxial compression and extension tests with pore pressure measurement on natural soft clay samples, taken vertically and horizontally. One reason for this may be the need to test at low effective stress levels consistent with field stresses. A notable exception is the work of Wesley (1975) on soft clay from Mucking Flats adjacent to the River Thames. The water-table at this site is about 1 m below ground level, and other relevant data here are for the soil between depths of 3.18 m and 3.48 m, for which $w_L=56\%$, $w_P=25\%$, $w_n=52\%$, the sensitivity from vane tests is about 5, and the measured overconsolidation ratio is about 1.5. The clay fraction is about 30%.

Undrained triaxial compression and extension tests were carried out by Wesley on horizontal and vertical undisturbed samples first consolidated to the assumed mean effective stress in the field, equal to 20.7 kPa. It has already been seen in a previous section that one-dimensional consolidation tests on vertical and horizontal specimens gave, for small stress changes above field values, a horizontal compressibility almost twice the vertical compressibility. The stress paths for the undrained tests are shown on a q/p' plot in Fig. 4.24, and it can be seen that, with the exception of the extension tests on the vertical sample, all stress paths are almost linear until failure is approached.

There is a remarkable correspondence between the inter-relation of all four stress paths in Fig. 4.24 and the predicted paths in Fig. 4.23b, even to the extent that the slopes of the predicted paths are very similar to the experimental ones. This suggests, at least for this clay, that the horizontal modulus is about half the vertical modulus for increasing effective stresses, and that the modulus for decreasing effective stress is about twice that for increasing effective stress.

Stresses in soft clay under field loading, such as embankments or storage tanks, are often calculated on the basis of the soil being an isotropic, elastic medium. This practice has usually been followed for convenience rather than with any conviction that it was correct. In fact, it receives strong support from Fig. 4.23b, at least for an extensive zone under the loaded area, where there

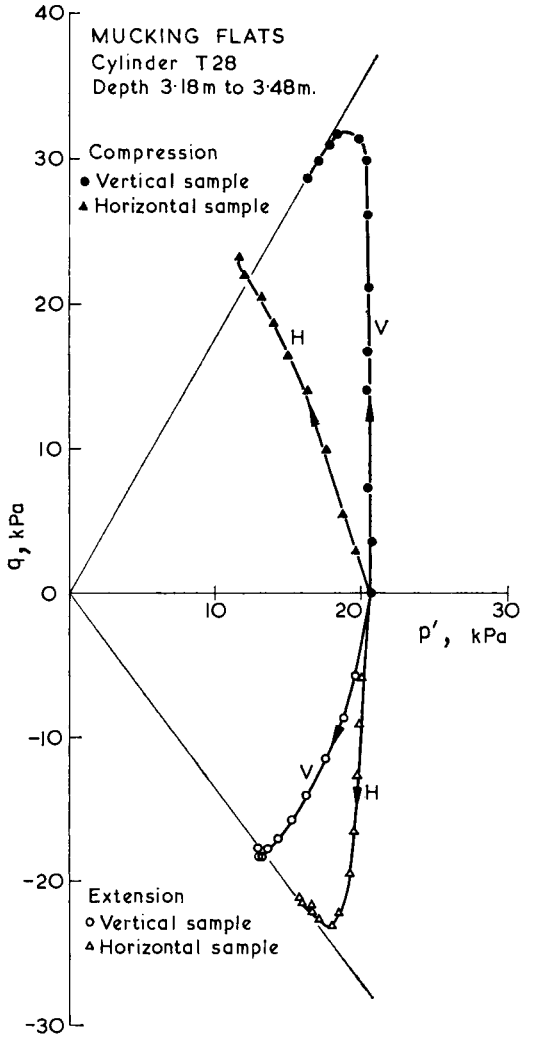

Fig. 4.24. Observed stress paths in compression and extension for vertical *(V)* and horizontal *(H)* specimens.

will be an increase in vertical effective stress and, probably, a decrease in horizontal effective stress due to the increase of pore pressure. The stress path in this case corresponds to that for vertical compression in Fig. 4.23b and is thus a vertical line, which is the stress path for an isotropic elastic soil. Towards the edge of the loaded area, and outside it, the conditions may be quite different. As the applied increments of principal stress rotate towards the horizontal, there will be a tendency for the horizontal effective stresses to increase, and for the vertical effective stresses to decrease, particularly outside

the loaded area. This would tend to give a horizontal stiffness much lower than the vertical stiffness. In an actual case, there would be a progressive change from within the zone under the loaded area to a zone outside the loaded area where the stress conditions would change from the soil acting isotropically to the soil acting with a vertical stiffness much greater than the horizontal stiffness.

4.5.4 Phase 2: plastic deformation

The pore pressure response during the plastic phase depends on the form of the yield locus. A simple picture of the behaviour can be found by assuming the soil to follow a straight line on the $q-p'$ plot, with a gradient of $-m$ as shown in Fig. 4.25, and by again considering separately the triaxial and plane strain conditions. (Note that the value of m for a given soil will not be the same in triaxial and plane strain conditions.)

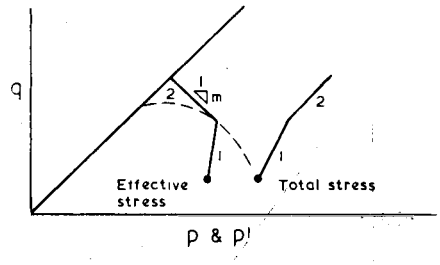

Fig. 4.25. Idealised stress paths during plastic deformation.

4.5.4.1 Triaxial conditions

Although the element of clay now responds plastically, it is assumed that the total stress increments remain proportional, i.e. $\Delta\sigma_h = l_2\Delta\sigma_v$. However, the constant l_2 will not be exactly the same as the constant l_1 adopted for a purely elastic response in Section 4.5.1.3. Hence:

$$\Delta u = \Delta p - \Delta p' = \tfrac{1}{3}(\Delta\sigma_v + 2\Delta\sigma_h) - \left(-\frac{\Delta q}{m}\right)$$

and:

$$\Delta u = \Delta\sigma_v \left[\tfrac{1}{3}(1 + 2l_2) + \frac{1 - l_2}{m}\right] \tag{4.20}$$

Therefore:

$$A = \frac{1}{3} + \frac{1}{m} \tag{4.21}$$

If $m=1.5$, then $A=1$ regardless of the value of l_2.

4.5.4.2 Plane strain conditions

In this case, the horizontal stress increment in the plane strain direction is some value between the major principal stress increment, $\Delta\sigma_v$, and the minor principal stress increment, $\Delta\sigma_h$. For the purpose of this exercise, its value will be assumed to be $\frac{1}{2}(\Delta\sigma_v+\Delta\sigma_h)$ (after Hill, 1950, p. 129). Thus:

$$\Delta u = \Delta p - \Delta p' = \tfrac{1}{2}(\Delta\sigma_v + \Delta\sigma_h) - \left(\frac{-\Delta q}{m}\right)$$

$$= \Delta\sigma_v \left[\tfrac{1}{2}(1 + l_2) + \frac{1 - l_2}{m}\right] \tag{4.22}$$

or

$$A = \tfrac{1}{2} + \frac{1}{m} \tag{4.23}$$

If $m=2$, $A=1$ regardless of the value of l_2.

The pore pressure responses as functions of σ_v are shown in Fig. 4.26a for triaxial conditions and in Fig. 4.26b for plane strain conditions.

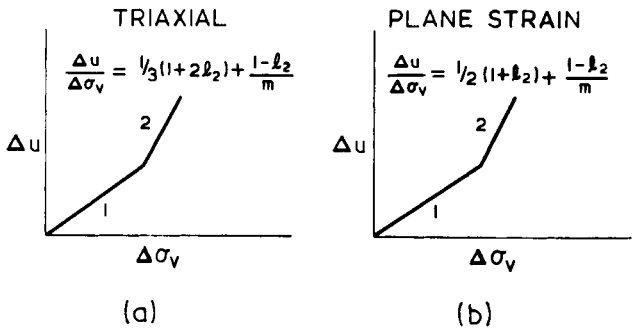

Fig. 4.26. Expected pore pressure response during plastic deformation.

4.5.5 Phase 3a: contained failure, no softening

During this phase, an element of soil can continue to take increasing total stress, providing no increase in deviator stress, q, occurs and the element is restrained by adjacent soil. Thus, as shown in Fig. 4.27, effective stresses remain constant, but p increases with no increase in q. Therefore:

$$\Delta q = 0 \text{ and, hence, } \Delta\sigma_v = \Delta\sigma_h$$

and:

$$\Delta u = \Delta p - \Delta p' = \Delta\sigma_v = \Delta\sigma_h \tag{4.24}$$

In Phase 3a, the line of Δu vs. $\Delta\sigma_v$ has a gradient of 1.0, as shown in Fig. 4.28, and would be collinear with the Phase 2 response if $m=1.5$ for triaxial

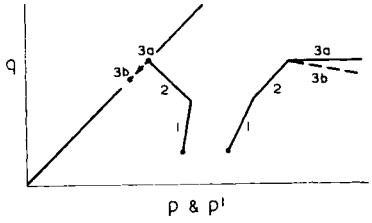

Fig. 4.27. Idealised stress paths during failure.

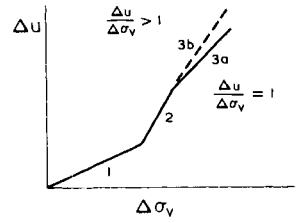

Fig. 4.28. Expected pore pressure response during failure.

conditions and $m=2$ for plane strain. In many field cases, it is likely that the Phase 2 and 3 response would be indistinguishable.

4.5.6 Phase 3b: contained failure, with softening

In this case, an element of soil can continue to take increasing total stress providing the deviator stress, q, decreases commensurately with the resulting shear strain.

By definition:

$$\Delta u = \Delta p - \Delta p'$$

$$\Delta p = \tfrac{1}{3}\,(\Delta\sigma_v + 2\Delta\sigma_h)$$

and with the assumption:

$$\Delta q/\Delta p' = g$$

then:

$$\Delta u = \Delta\sigma_v\left(\frac{1}{3} - \frac{1}{g}\right) + \Delta\sigma_h\left(\frac{2}{3} + \frac{1}{g}\right) \tag{4.25}$$

If $\Delta\sigma_v = \Delta\sigma_h$, then $\Delta u = \Delta\sigma_v$, which is case 3a above. If $\Delta\sigma_v < \Delta\sigma_h$, then $\Delta u > \Delta\sigma_v$, as shown in Fig. 4.28.

4.6 UNDRAINED SHEAR STRENGTH

The undrained shear strength of a soft clay deposit is not a unique property of the material but depends on many factors such as: (1) the orientation of the planes of maximum shear stress (on account of the anisotropy of the clay), (2) the rate of application of the loading, (3) the size of the specimen of clay involved in the test, and (4) the type of test and apparatus used for the measurement. All these factors are interrelated, and it is not possible to isolate any one to discover how it alone affects the undrained shear strength of a soft clay. In this section, discussion is concentrated on the influence of sample orientation and anisotropy on the measurement of undrained shear strength in conventional triaxial tests. Some brief statements on the other factors are subsequently offered.

4.6.1 Influence of sample orientation on undrained triaxial shear strength

4.6.1.1 Anisotropy effects

If triaxial specimens are cut from a natural soil deposit with the specimen axes ranging from vertical to horizontal, the undrained strengths of these specimens vary with their inclinations. Two distinct types of variation have been observed. The first type of variation has been reported by Lo (1965), De Lory & Lai (1971) and Wesley (1975), and consists of a progressive decrease in strength as the inclination rotates from vertical to horizontal. The second type, reported by Duncan & Seed (1966), Mitchell (1967) and Parry & Nadarajah (1974), shows a lower strength for inclined specimens than for vertical or horizontal specimens, with the last two giving similar strengths. In some cases, a marginally higher strength is observed for specimens cut vertically than for those cut horizontally. The influence of in-situ stress and anisotropy of fabric on the determination of shear strength characteristics has been discussed by Bhaskaran (1975).

Two possible causes may be advanced for this variation in laboratory measured strength with specimen inclination. These are: (1) anisotropy in the soil, leading to a different effective stress path during testing and consequently, a different strength, and (2) preferred orientation of particles, leading to different shear resistances depending on the relative inclination of the failure plane and on the particle orientation.

4.6.1.2 Influence of anisotropy on stress paths

The influence of anisotropy on effective stress paths in undrained triaxial tests has been discussed in an earlier section. Predicted paths within the elastic phase are shown in Fig. 4.23b together with some corresponding published stress paths in Fig. 4.24. A simple projection of these stress paths to meet the failure envelope would clearly give, for compression tests, a lower strength for the horizontal specimen than for the vertical specimen. In fact, as the effective stress paths approach the failure envelope, they will often deviate from a straight line, as seen for the vertical specimen in Fig. 4.24, which experiences a peak value of q before the path joins the failure envelope and follows it as the specimen softens. From critical state theory, it would be expected that the two stress paths would tend to converge after failure. In some cases, as discussed above, samples will yield and undergo plastic deformation before failing, and consequently, they will not follow a simple linear effective stress path to failure. Notwithstanding these deviations from a simple linear stress path, soil deposits which exhibit the type of anisotropic behaviour discussed in Sections 4.5.1 and 4.5.2 are likely to show higher undrained compression strengths for vertical specimens than for horizontal ones.

An inclined specimen should show a progressive change in stress path from that for a vertical specimen to that for a horizontal specimen and, consequently, a progressive change in strength. This is shown very well in Fig. 4.29,

reproduced from the work of Wesley (1975) for Mucking Flats clay. This soil was taken from the same depth as that represented in Fig. 4.24 but from a different cylinder; the paths show rather less linearity than in Fig. 4.24 but their interrelation is exactly the same; it should be noted that this is a t—s' plot reproduced directly from Wesley's thesis. Isotropic elastic behaviour would not give a vertical straight line for triaxial compression, but one inclined at 3:1 (Table 4.1), as shown by the broken line in Fig. 4.29. It can again be seen that the stress path for the undrained compression test on the vertical specimens corresponds closely to the isotropic elastic stress path.

It is clear that the failure values for deviator stress, represented by the intersection of the stress paths with the failure envelopes, must give a progressive decrease in undrained strength from vertical to horizontal in undrained compression tests.

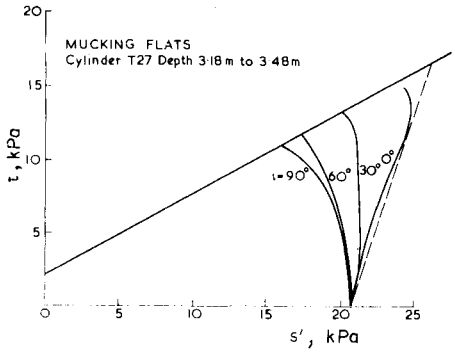

Fig. 4.29. Stress paths from vertical ($i=0°$), horizontal ($i=90°$) and inclined specimens of clay from Mucking Flats (Wesley, 1975).

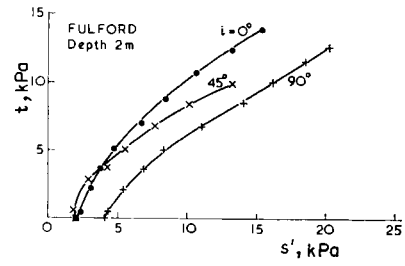

Fig. 4.30. Stress paths from vertical, horizontal and inclined specimens of Fulford clay (Parry & Nadarajah, 1974).

4.6.1.3 Possible influence of particle orientation

The second general category of soft clays is that for which the undrained shear strength in compression is less for inclined specimens than either vertical or horizontal specimens. One published example is Fulford clay (Parry & Nadarajah, 1974), and it is of interest to look at the effective stress paths for this soil. They are reproduced on a t—s' plot in Fig. 4.30 and it is immediately seen that these stress paths are completely different from those in Fig. 4.29; indeed, the anisotropic behaviour for this soil appears to be more complex than for the Mucking Flats soil.

Where the strength of inclined specimens is less than for both the vertical and horizontal specimens, it seems likely that reduced resistance on planes corresponding closely to the longer particle dimensions must be having an influence. This appears at first to be contradicted to some extent by the fact that the measured values of effective stress parameters, c' and φ', do not seem to be very much affected by the orientation of the failure plane (e.g. Lo, 1965;

Parry & Nadarajah, 1974), at least in soils of low or moderate sensitivity. This may not be a real contradiction, however, if looked at closely, for the two reasons:

(1) Measurement of the values of the parameters c' and φ' requires consolidation in the laboratory, usually isotropic and usually to stresses markedly higher than the field stresses. It has already been pointed out in Section 4.4.1 that initial anisotropy can be overcome progressively by consolidation under a different stress regime from that in the field, with the initial anisotropy disappearing completely when the applied major effective stress is about three times the preconsolidation pressure in the field.

(2) At the low effective stress levels in these soils in their natural state and as sampled, it is the value of c' and not φ' which dictates strength, and quite small variations in values of c' can have a measurable influence on undrained strength. In undrained compression tests on Fulford clay (Parry & Nadarajah, 1974), the value of c' in fact varied from 3 kPa for horizontal samples to 8 kPa for vertical specimens. The source of the c' component in strength is obscure and, in some cases, may be incorrectly deduced from a linear extrapolation of an effective stress envelope to give a zero effective stress ordinate. In Fulford clay, however, a number of measurements were made which indicated the presence of tensile resistant bonds between the particles; these could be the source of the cohesive strength in this soil, and they may perhaps be anisotropic. This anisotropy of cohesive strength has been shown very clearly for two quick clays by Lo & Morin (1972); consolidated undrained and drained tests on these clays gave effective stress failure envelopes which showed high strengths in the low stress range, these strengths differing with different specimen inclinations.

4.6.2 Rate of testing

Strain rate effects in laboratory measurements of shear strength have been the subject of many publications (e.g. Bishop & Henkel, 1962; Richardson & Whitman, 1963; Eide, 1967; Berre & Bjerrum, 1973; Wesley, 1975). They will not be discussed further here other than to point out the following:

(1) Undrained triaxial tests carried out rapidly may show a distinct peak in the stress-strain curve, whereas slower tests may not.

(2) Remoulded clays usually show a drop of 5% in undrained triaxial shear strength for every tenfold decrease in testing rate.

(3) Undisturbed clays tend to show a much greater drop in shear strength with decreasing strain rate than remoulded clays; they may show a drop of as much as 15% for every tenfold decrease in testing rate.

(4) An axial strain rate of 0.5% per minute, or slower, should be adopted for undrained triaxial tests on specimens 38 mm diameter and 76 mm long.

In the field, there is an additional complication caused by a variation in the rate of the test. This is because of the fact that the drainage of the 'specimen' of clay involved in the field test is not controlled (as in a laboratory test), so that varying amounts of consolidation will occur with different strain rates and, consequently, differing values of shear strength will be measured. This effect is intimately related to the question of size of specimen, and it is briefly considered in the next section.

4.6.3 Specimen size

The size of 'specimen' involved in a field test is of considerable importance for two reasons. The first of these, as mentioned above, is related to the degree of consolidation that may occur during a test. If a small zone of soil is under test, significant dissipation of excess pore pressure may occur even if the test is conducted quite rapidly.

In general, if a_0 is a representative dimension of the in-situ test apparatus then, by dimensional analysis, the time factor, T, governing the rate of dissipation of excess pore pressure will be some function of $c_{vc}t/a_0^2$ from elementary consolidation theory, where t is the time that has elapsed and c_{vc} is the coefficient of consolidation of the soil in question. Hence, for a given soil and a given time of test, the time factor is inversely proportional to the square of the dimension a_0 (e.g. if the scale of the test is halved the degree of consolidation after a given time t is increased by a factor of four). Conversely, for a given apparatus, the time factor is directly proportional to the rate of testing, and hence the degree of consolidation is directly affected by the rate of testing. This is an entirely separate effect from that of strain rate *per se,* and one that is absent from laboratory undrained tests in which any drainage from the specimen is prevented.

The second effect of the size of a specimen is related to how representative it is of the ground that is to be loaded. Any inhomogeneity in the form of anisotropy or fabric within the clay or thin silt seams will have a major effect on its shear properties and will introduce an important scale effect. This aspect of soil behaviour was discussed in detail by Rowe (1972).

4.6.4 Type of test

Clearly, in the light of the above, the type of test and choice of apparatus will have a big influence on the measurement of the undrained shear strength of soft clay. In the laboratory, the most common test is likely to be the conventional triaxial compression test, although there may be occasions when tests in a plane strain apparatus or in direct simple shear are called for.

In the field, the options will include the vane shear test, the cone penetration test and the pressuremeter test. The vane (Skempton, 1948) is the most widely used field test to measure in-situ the undrained shear strength of soft clay. It has been found in some soft clays, however, that undrained shear strengths obtained by the field vane are higher than those deduced from a back-analysis

of field failures (e.g. see Bjerrum, 1972; Parry, 1972; Pilot, 1972). The discrepancy probably arises from strain rate effects or anisotropy of shear strength or, perhaps, from both. Bjerrum (1972) considered strain rate to be dominant, but there is some controversy about the effect of strain rate. Cadling & Odenstad (1950) found a 20% increase in undrained shear strength in a soft clay for a tenfold increase in rate of rotation of the vane from 0.1° per second to 1° per second, whereas Aas (1967) found no significant increase for the same variation in rate.

The vane test can be used to measure the undrained shear strength in different directions by changing the geometry of the vane, as discussed by Aas (1967) and Menzies & Mailey (1976).

In-situ testing of soft clay is the topic of Chapter 3, and further discussion is inappropriate here.

4.7 PORE PRESSURES IN SOFT GROUND UNDER SURFACE LOADING

In the previous three sections, the behaviour of soft clay has been examined in terms of yield, undrained effective stress paths and undrained shear strength. In this section, the relevance of these concepts to field situations is examined and an assessment made of their usefulness in the prediction of field performance or in the interpretation and back-analysis of full-scale field data.

The theoretical ideas of pore pressure development are applied here to well-documented field cases. The first case is that of axisymmetric loading of two circular fills at Åsrum, in Norway reported by Höeg et al (1969). The other cases are of an axisymmetric loading at Canvey Island, England reported by Pender et al (1975) and of a plane strain case of a long road embankment at Boston reported by D'Appolonia et al (1971).

4.7.1 Application to field case of axisymmetric loading at Åsrum

4.7.1.1 General description

In order to study the problems of likely settlements of buildings on the quick clays in the area around Oslo, the Norwegian Geotechnical Institute carried out two field tests on a site at Åsrum. Each test consisted of a circular fill placed on the existing ground surface, with careful measurements being made of excess pore pressures generated in the underlying clay and of settlements of the fill. Full details were given by Höeg et al (1969).

Profiles of the soil at the two neighbouring sites are given in Fig. 4.31a, and the in-situ vertical stresses are shown in Fig. 4.31b (both diagrams are reproduced from the paper by Höeg et al). The upper 1 to 2 m consists of a fairly stiff fissured crust, below which the very quick and soft clay extends to bedrock. The natural water content of the clay ranges from about 55% to 70%, compared with a range of liquid limit of about 35% to 50%. The undrained shear strength was measured by in-situ vane tests and unconfined compression

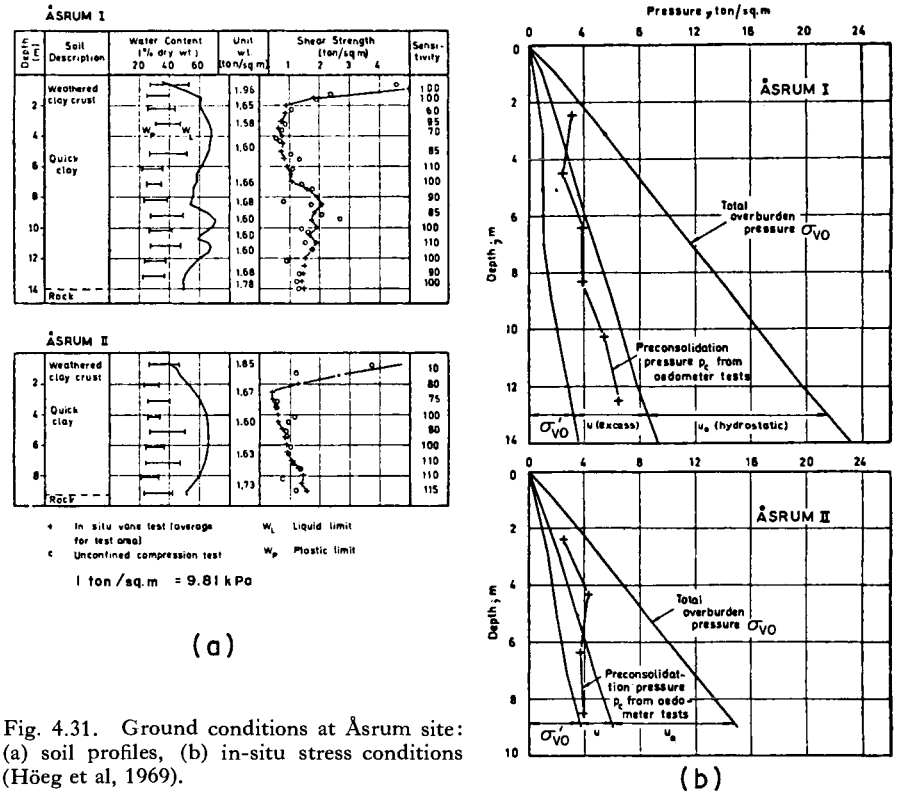

Fig. 4.31. Ground conditions at Åsrum site: (a) soil profiles, (b) in-situ stress conditions (Höeg et al, 1969).

tests on undisturbed samples. Beneath the surface crust, the strength is as low as 5 kPa and increases with depth.

The observed values of the excess pore pressures recorded by the piezometers at depths of 3 m and 5 m beneath the two fills are shown in Figs. 4.32a to 4.32d. It is at once apparent that the responses of the piezometers near the centre-line show two well-defined phases, with a sharp break between these two. These responses will now be interpreted in the light of the theories developed in Section 4.5.

4.7.1.2 Åsrum I: piezometer A at 3 m depth on centre-line

The positions of the various piezometers are indicated in Fig. 4.33a. In this section, the response of piezometer A at a depth of 3 m on the centre-line of the fill is examined in detail.

Since the piezometer is on the centre-line, conditions of axial symmetry applied throughout the test. Although the diameter of the fill reduces with height, for the purposes of the calculations the surface loading produced by the fill is assumed to be a uniform circular vertical load of intensity $\Delta\sigma=\gamma h$ and of average radius $a=6.25$ m, as shown in Fig. 4.33b.

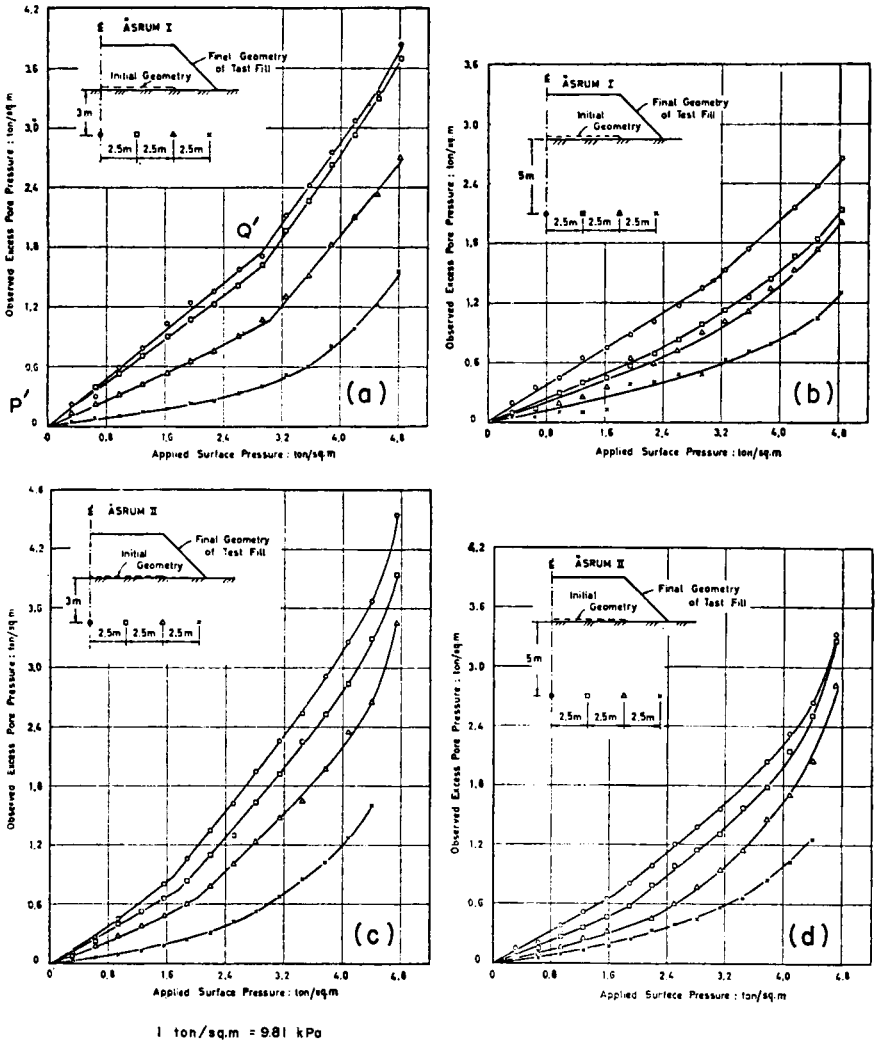

Fig. 4.32. Pore pressure responses at Åsrum I and Åsrum II sites: (a) depth 3 m, Åsrum I, (b) depth 5 m, Åsrum I, (c) depth 3 m, Åsrum II and, (d) depth 5 m, Åsrum II (Höeg et al, 1969).

For the initial analysis of the behaviour of the soft clay, it is assumed that the elastic response is *isotropic*. From the elastic stress distributions tabulated by Poulos & Davis (1974) for a uniform flexible circular load of radius a on an elastic half-space, the relevant values for piezometer A are:

$$r/a = 0,\ z/a = 0.48,\ \Delta\sigma_v/\Delta\sigma = 0.919,\ \Delta\sigma_h/\Delta\sigma = 0.391.$$

Hence, the ratio of the increments of total stress is:

$$l_1 = \frac{\Delta\sigma_h}{\Delta\sigma_v} = \frac{0.391}{0.919} = 0.425$$

and the factor $(1+2l_1)/3=0.617$. From Eq. 4.19, the perfectly elastic response of piezometer A would be (Fig. 4.33c):

$$\frac{\Delta u}{\Delta\sigma_v} = \tfrac{1}{3}(1+2l_1) = 0.617 \tag{4.26}$$

In terms of the observed surface load, $\Delta\sigma$, (rather than the unknown local increment of total vertical stress, $\Delta\sigma_v$) the response is given by (Fig. 4.33d):

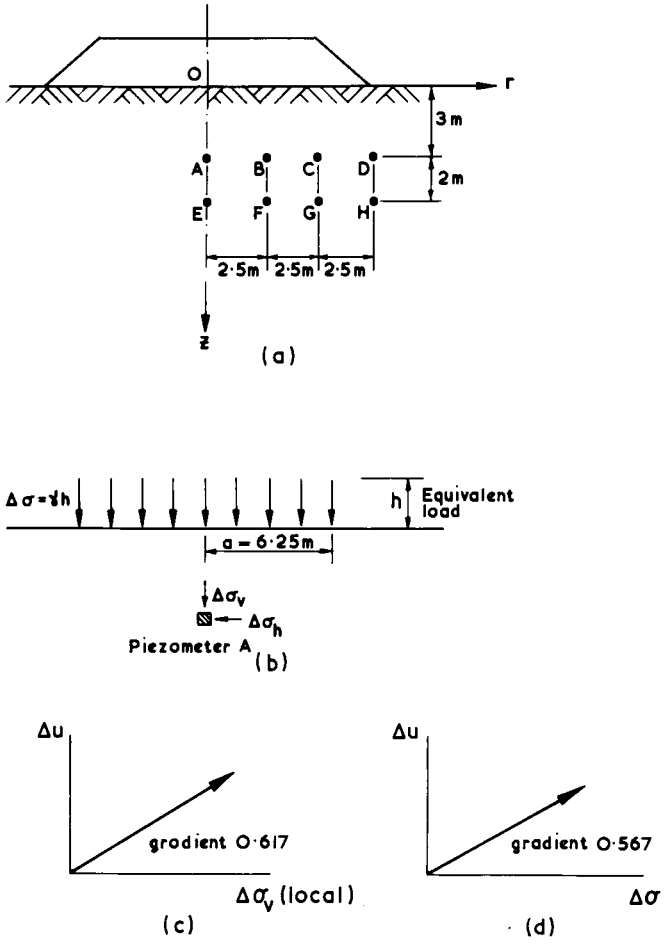

Fig. 4.33. Predicted pore pressure response at Åsrum I site: (a) locations of piezometers, (b) idealised equivalent loading, (c) elastic pore pressure response for piezometer A related to change in total vertical stress at piezometer tip, (d) elastic pore pressure response for piezometer A related to surface loading.

$$\frac{\Delta u}{\Delta \sigma} = \frac{\Delta u}{\Delta \sigma_v} \cdot \frac{\Delta \sigma_v}{\Delta \sigma} = 0.617 \times 0.919 = 0.567 \tag{4.27}$$

This gradient almost exactly matches that of the first linear portion, $P'Q'$, of the relevant plot in Fig. 4.32a. (It should be noted that the scales in Fig. 4.32 for Δu and $\Delta \sigma$ are unfortunately not the same). The point Q' corresponds to the change in behaviour from an elastic response to either a plastic response or a contained failure. This occurs at an increment of surface load of $\Delta\sigma = 2.84$ t/m^2 (27.9 kPa), for which the excess pore pressure generated at piezometer A based on an elastic response would be, from Eq. 4.27, $\Delta u = 0.567 \times \Delta\sigma = 1.61$ t/m^2. This corresponds closely to the value observed for point Q' in Fig. 4.32a.

If the clay behaves perfectly elastically in the first phase, then the excess pore pressure is obtained directly from the increment of mean total principal stress, Δp, thus:

$$\Delta p = \tfrac{1}{3}(\Delta\sigma_v + 2\Delta\sigma_h)$$

$$\Delta p' = \tfrac{1}{3}\{(\Delta\sigma_v - \Delta u) + 2(\Delta\sigma_h - \Delta u)\}$$

and

$$\Delta p - \Delta p' = \Delta u$$

In an isotropic elastic soil under undrained conditions (i.e. no volume change), $\Delta p' = 0$, and the stress path on a $q-p'$ plot is a vertical straight line (see Table 4.1). Thus, if $\Delta p' = 0$, then $\Delta u = \Delta p$. Hence, the result of Eq. 4.27 could be obtained directly from an appropriate curve for $\Delta p/\Delta\sigma$ and, thereby, avoid the need to calculate the stress increments $\Delta\sigma_v$ and $\Delta\sigma_h$. The evaluation of the latter, however, has the two advantages that: (1) it allows estimates to be made of the total and effective stress paths and, hence, enables a fuller understanding of the behaviour of the clay, and (2) it allows an anisotropic elastic response of the clay to be used if necessary, i.e. the use of the expressions given in Eq. 4.19 and Table 4.1.

At the stage represented by Q', the clay locally around piezometer A yields. At yield, $\Delta\sigma = 2.84$ t/m^2. From elastic theory:

$$\Delta\sigma_v = 2.61,\ \Delta\sigma_h = 1.11,\ \Delta p' = 0$$

$$\Delta q = 1.50,\ \Delta p = 1.61 \text{ (all units in t/m}^2\text{)}.$$

The total and effective stress paths for the stages PQ and $P'Q'$ can now be plotted if the initial in-situ stress states are known. Unfortunately, the problem of the in-situ lateral stress is a difficult one, and the best that can be done is to estimate this from all the limited information available.

From the results of the consolidation tests and the profile of stresses in Fig. 4.31b, for the depth $z = 3$ m, $\sigma'_{vo} = 1$ t/m^2, $u_o = 4.5$ t/m^2 and the overconsolidation ratio is 3. By means of Eq. 4.6 for the estimation of K_o for lightly overconsolidated soils, and on the assumption that $K_{nc} = 0.65$ and $\upsilon' = 0.28$ (for a soil with plasticity index of 16%), then:

$$K_o = 3 \times 0.65 - \frac{0.28 \times 2}{0.72} = 1.26$$

This estimate for K_o gives $\sigma'_{ho} = 1.26$ t/m², $\sigma'_{vo} = 1.00$ t/m², $q_o = -0.26$ t/m², $p'_o = 1.17$ t/m² and $p_o = 5.67$ t/m². The total and effective stress paths, $P_A Q_A R_A$ and $P'_A Q'_A$, for an element of soil at point A based on these estimated in-situ stresses are plotted in Fig. 4.34. From the position of the point Q'_A, and from the in-situ vane shear strengths (plotted in Fig. 4.31a) of about 0.8 t/m² corresponding to $q_f = 1.6$ t/m²), it is concluded that the clay has probably reached *failure* at point Q'_A. This means that for the soil at this depth of 3 m, there is no second phase of plastic yielding and that the behaviour goes directly from elastic to contained failure.

If this suggestion is correct, then the second linear phase of pore pressure response in Fig. 4.32a should have a gradient $\Delta u/\Delta\sigma = \Delta\sigma_v/\Delta\sigma$, on the assumption that no post-peak softening occurs (see Section 4.5.4). Once the clay has yielded or failed, the assumption of an elastic stress distribution throughout the elastic half-space is no longer valid, but most of the soil some distance from the region of contained failure is still behaving elastically; inside the failing region, the total stress distribution must alter to some degree to accommodate the plastic strains of the soil. There is limited evidence to show that the increments of total vertical stress, $\Delta\sigma_v$, remain as though they were given by elastic theory, and the increments of total horizontal stress, $\Delta\sigma_h$, are larger than the corresponding elastic values. If it is assumed, for the sake of argument, that the elastic stress distribution for $\Delta\sigma_v$ is valid, then for the Phase 3a in Fig. 4.20, the expected response is $\Delta u/\Delta\sigma = \Delta\sigma_v/\Delta\sigma = 0.919$. The observed value is 1.03, so that the above assumptions are in reasonable agreement with the field data, and certainly do not conflict with them. It seems likely, in fact, that some post-peak softening occurred, as shown by Phase 3b in Fig. 4.20.

4.7.1.3 Åsrum I: piezometer E at 5 m depth on centre-line

With the same assumptions for piezometer E as for piezometer A, the relevant values are:

$$r/a = 0,\ z/a = 0.8,\ \Delta\sigma_v/\Delta\sigma = 0.756,\ \Delta\sigma_h/\Delta\sigma = 0.184$$

Therefore:

$$l_1 = \frac{0.184}{0.756} = 0.243$$

$$\frac{\Delta u}{\Delta\sigma_v} = \tfrac{1}{3}(1 + 2l_1) = 0.496$$

and:

$$\frac{\Delta u}{\Delta\sigma} = 0.496 \times 0.756 = 0.375$$

This gradient should be compared with that of 0.45 for the observed data of Fig. 4.32b.

At yield, $\Delta\sigma = 3.1$ t/m^2, and from elastic theory:

$$\Delta\sigma_v = 2.34,\ \Delta\sigma_h = 0.57,\ \Delta p' = 0$$
$$\Delta q = 1.77,\ \Delta p = 1.16 \text{ (all units in t/m}^2\text{)}.$$

An estimate is now made of the initial in-situ stress state at piezometer E, on the same basis as for piezometer A in the last section. From Fig. 4.32, for $z = 5$ m, $\sigma'_{vo} = 1$ t/m^2, $u_o = 7.7$ t/m^2 and $OCR = 3$. As before, K_o is taken as 1.26, so that $\sigma'_{ho} = 1.26$ t/m^2, $q_o = -0.26$ t/m^2, $p'_o = 1.17$ t/m^2 and $p_o = 8.87$ t/m^2. The total and effective stress paths, $P_E Q_E R_E$ and $P'_E Q'_E$, for an element of soil at E are plotted in Fig. 4.34; the effective stress path starts from the same point as for element A (by chance) and only differs from it by virtue of a slightly larger value of Δq to cause yield.

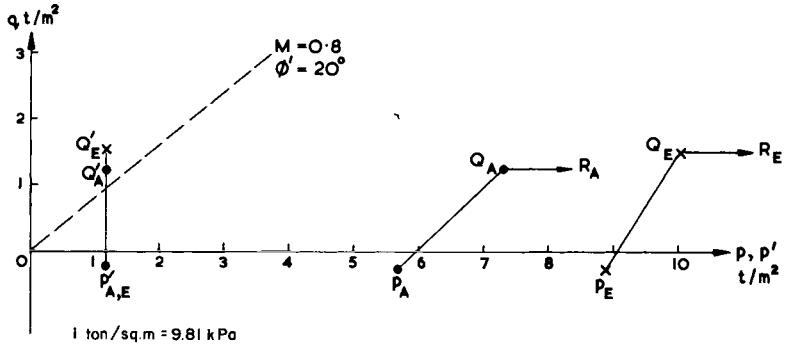

Fig. 4.34. Predicted total stress and effective stress paths for piezometers A and E at Åsrum I site.

The same argument as for element A is invoked to suggest that element E has reached failure at Q'_E, and that the behaviour of the soil changes directly from elastic to contained failure without an intermediate stage of plastic yielding. On this basis, the gradient of the second stage would be expected to be $\Delta u/\Delta\sigma = 0.756$; this compares with a measured value of 0.687 from Fig. 4.32 b.

4.7.1.4 Åsrum I: piezometers not on centre-line

For the piezometers B, C, & D at a depth of 3 m and F, G & H at a depth of 5 m, which are not on the centre-line of the fill, conditions of axial symmetry no longer apply. The simple expressions derived in Section 4.5 are not applicable, and the situation is much more complicated because of the rotation of the principal axes of stress and stress increment. However, if the soil behaves in an isotropic elastic manner while undergoing no volume change, then

$\Delta p' \equiv 0$, and the excess pore pressure is given (as before) by the increment of mean total principal stress, Δp. From the charts and functions given by Poulos & Davis (1974), the ratios $\Delta p/\Delta\sigma$ have been calculated for the six piezometers, and these are compared in Table 4.2 with the observed gradients of $\Delta u/\Delta\sigma$ taken directly from the first phases of the responses plotted in Figs. 4.32a and 4.32b. There is reasonably good agreement between the two sets of values, which supports the interpretation of the results in terms of isotropic elasticity.

Table 4.2. Comparison between first phase of observed excess pore pressures and those computed from elastic theory

Piezometer	z/a	r/a	Computed $\Delta p/\Delta\sigma$	Observed $\Delta u/\Delta\sigma$
A	0.48	0	0.567	0.600
B	0.48	0.4	0.532	0.546
C	0.48	0.8	0.396	0.343
D	0.48	1.2	0.186	0.105
E	0.8	0	0.375	0.450
F	0.8	0.4	0.347	0.315
G	0.8	0.8	0.266	0.276
H	0.8	1.2	0.162	0.150

The consequences of the departure from the simple case of axial symmetry is illustrated for the case of piezometer *G* in Fig. 4.35. From Poulos & Davis (1974), it is possible to calculate the increments of stress (shown in perspective in Fig. 4.35a and in elevation in Fig. 4.35b) from elastic theory in terms of the applied (circular) surface load, $\Delta\sigma$. They are $\Delta\sigma_z/\Delta\sigma = 0.504$, $\Delta\sigma_r/\Delta\sigma = 0.185$, $\Delta\sigma_\theta/\Delta\sigma = 0.109$ and $\Delta\tau_{zr}/\Delta\sigma = 0.204$. The Mohr's circle of stress for the (r, z) plane is shown in Fig. 4.35c, and the *principal increments* of stress can readily be calculated to be $\Delta\sigma_1/\Delta\sigma = 0.603$, $\Delta\sigma_2/\Delta\sigma = \sigma\Delta_\theta/\Delta\sigma = 0.109$ and $\Delta\sigma_3/\Delta\sigma = 0.086$. The principal axes of the stress increments are as shown in Fig. 4.35d and do not coincide with those of stress (the principal directions of which depend on the ratio of $\Delta\sigma$ to the initiali n-situ stresses at point *G*).

After yield has occurred, which is assumed to coincide with the onset of contained failure, the local distribution of stresses in the vicinity of *G* can no longer be elastic. It is suggested, for want of any experimental evidence, that the distribution of the major principal stress increment, $\Delta\sigma_1$, remains largely unaffected, and that $\Delta u = \Delta\sigma_1$. If this hypothesis is valid, then the expected gradient in Fig. 4.32a for the second phase for piezometer *G* would be $\Delta u/\Delta\sigma = (\Delta u/\Delta\sigma_1) \times (\Delta\sigma_1/\Delta\sigma) = 0.603$. This should be compared with an observed value of about 0.5.

4.7.2 Application to field case of axisymmetric loading at Canvey Island

As part of a detailed site investigation for a major oil refinery on a deposit of soft clay at Canvey Island in England, two small circular trial embankments

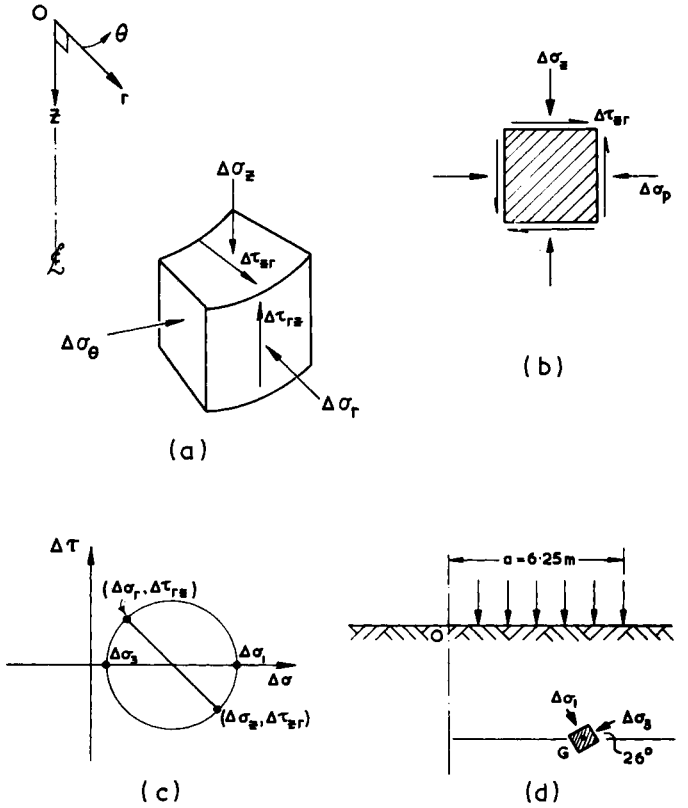

Fig. 4.35. Applied stresses at piezometer G at Åsrum I site: (a) perspective view, (b) elevation showing total stress increments, (c) Mohr circle of total stress increments, (d) directions of principal total stress increments from elastic theory.

were constructed to simulate the behaviour of the oil tanks. The performance of the embankment was monitored by observations of the settlement of the embankment, and of excess pore pressures recorded by piezometers placed in the ground beneath.

A detailed description of the site and instrumentation is given by George & Parry (1973). The pore pressure responses have been interpreted by Pender et al (1975) in the light of the theories advanced earlier in this chapter. The main findings only will be presented here.

Undisturbed samples of the soft clay were subjected to stress-controlled drained triaxial tests with a variety of stress paths in order to establish the yield locus. The results are shown in Figs. 4.16 and 4.17.

The locations of four piezometers, $P1$, $P5$, $P7$ and $P10$ are given in Fig. 4.36. Observed pore pressure changes are plotted in Fig. 4.37 against changes in vertical total stress, $\Delta\sigma_v$, at tip level, calculated by finite element analysis using a bilinear model. The response of each piezometer shows the expected

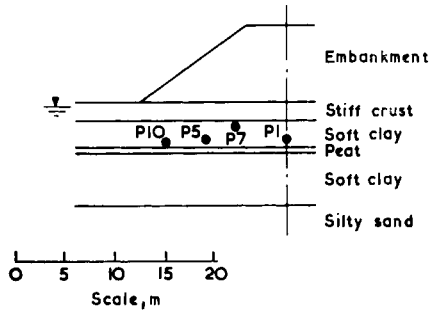

Fig. 4.36. Soil profile and piezometer locations for Canvey Island loading test (George & Parry, 1973).

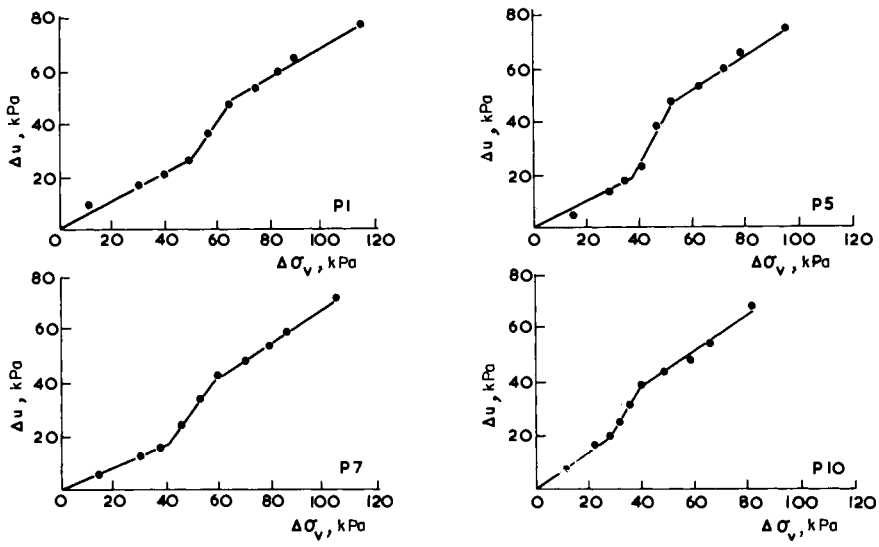

Fig. 4.37. Pore pressure responses for Canvey Island piezometers *P*1, *P*5, *P*7 & *P*10 plotted against calculated values of total vertical stress (Pender et al, 1975).

pattern of three linear phases, with well-defined points of change between the phases. For the first phase of the pore pressure response, the resulting value of the ratio $\Delta u/\Delta p$ was 0.5 to 0.6, whereas that predicted from isotropic elasticity would be 1.0. However, this discrepancy may be for any or all of the following reasons:

(1) The soil may have contained gas in the pore water because of the organic matter present in a recent alluvial deposit.
(2) The soil may have behaved anisotropically.
(3) The excess pore pressures will have dissipated during the period of construction of the embankment.

(4) The finite element computations are only approximate and are affected by the choice of boundary conditions and distribution of soil parameters within the mesh of elements.

4.7.3 Application to field case of plane strain loading near Boston

A well-documented case history for the plane strain situation has been reported by D'Appolonia et al (1971). The paper reports the evaluation of excess pore pressures measured under a long road embankment constructed near Boston, Massachusetts, as part of the Interstate Highway system.

A cross-section of the embankment is shown in Fig. 4.38a and piezometer locations in Fig. 4.38b. Full details of the properties of the ground are given in the paper by D'Appolonia et al. A selection of the observed values of excess pore pressure is shown in Fig. 4.39, where the results are plotted against the elevation of the embankment.

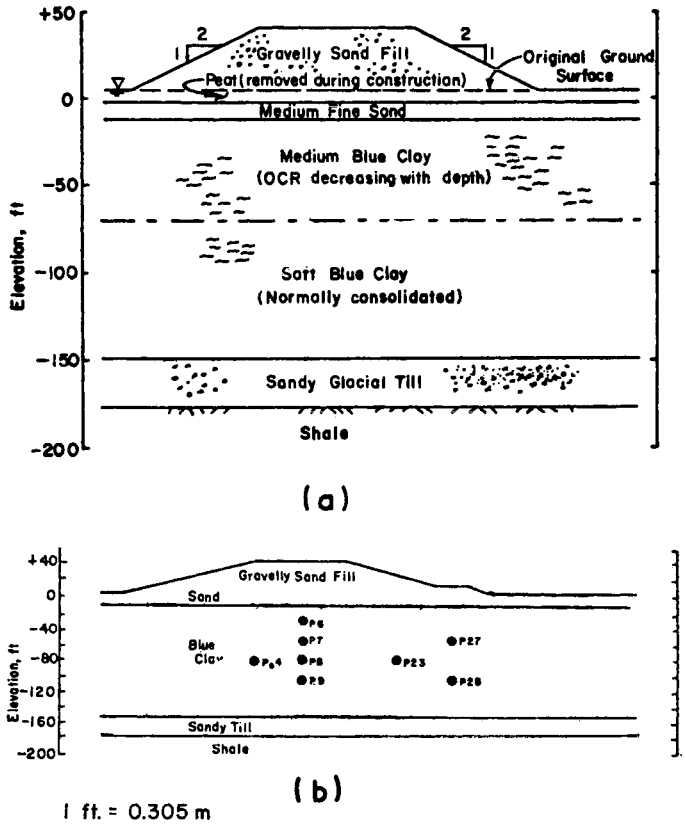

Fig. 4.38. Boston test embankment: (a) soil profile, (b) cross-section showing locations of piezometers (D'Appolonia et al, 1971).

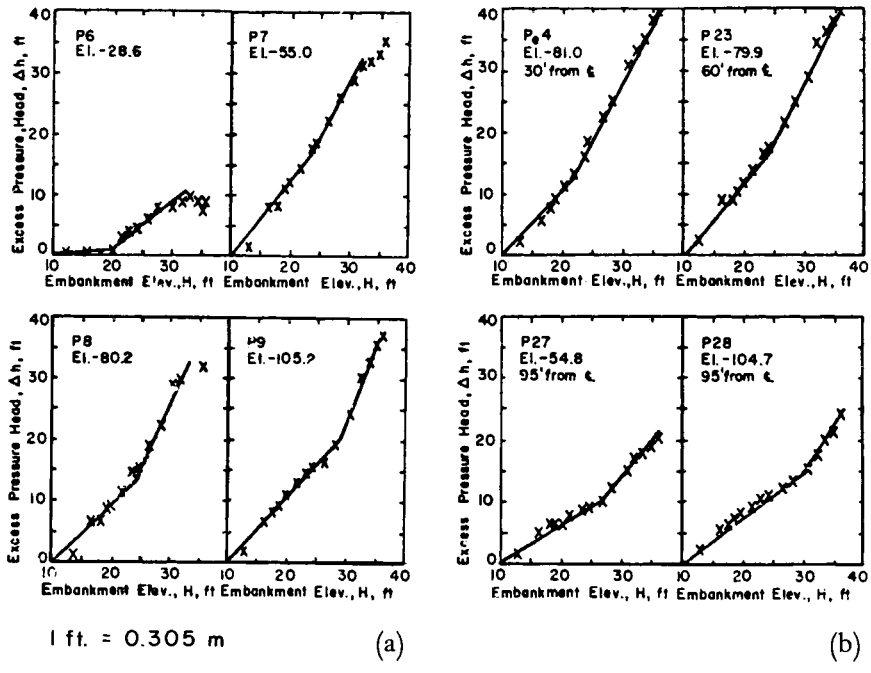

Fig. 4.39. Piezometer responses under Boston test embankment as a function of embankment elevation: (a) piezometers under centre-line, (b) piezometers off centre-line (D'Appolonia et al, 1971).

All the piezometer readings show two distinct responses. The end of the elastic phase is clearly defined in each case, as the local element of soil (around the piezometer) yields plastically or fails after behaving elastically. It was pointed out in Section 4.5.4 that, in some cases, the pore pressure responses in Phases 2 and 3 (i.e. plastic yielding and failure) would be difficult to distinguish. It can be seen that some of the responses in Fig. 4.39 could be three-phased, although a third phase is not clearly distinguishable. It is possible, then, that after completion of the elastic phase, the soil did progress through a plastic phase to contained failure without any distinct change in pore pressure response.

D'Appolonia et al (1971) made great efforts to interpret these results, and they considered four different distributions of increments of total stresses. They also considered various relationships between changes of total stress and of pore pressure. They concluded that, for the pre-yield elastic phase, the best prediction of pore pressure is given by three-dimensional elastic theory (as applied to the plane strain case) with $\Delta u = \Delta p$.

A direct comparison of the ratio of measured to predicted pore pressures (which is directly proportional to the gradients of the first phases shown in Fig. 4.39) is given in Fig. 4.40a for many of the piezometers. Those piezome-

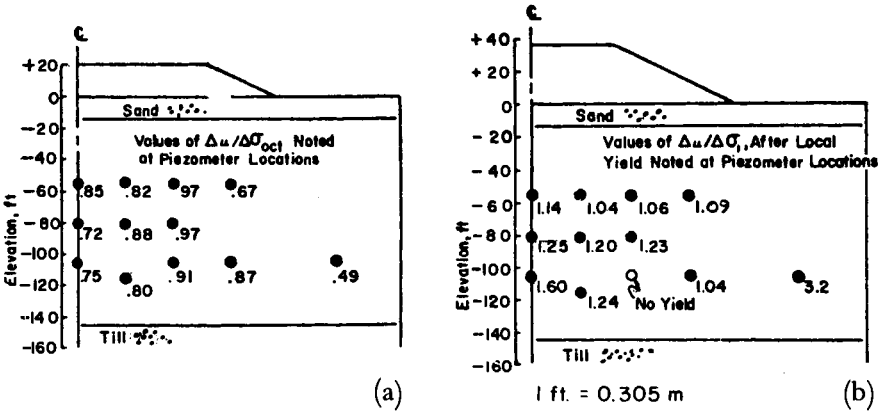

Fig. 4.40. Pore pressure changes under Boston test embankment: (a) before local yield, presented as a ratio of measured values to values calculated by three-dimensional elastic theory, (b) after local yield, presented as a ratio of measured values to values calculated by D'Appolonia et al (1971).

ters near the upper sand layer or near the till showed a substantial degree of dissipation due to drainage, and they were discounted by D'Appolonia et al.

During contained perfectly plastic failure, it has been shown that the change in pore pressure, Δu, should be equal to the (local) change in major principal total stress. Values of the ratio $\Delta u/\Delta\sigma_v$ for the same set of piezometers were calculated by D'Appolonia et al, and these are reproduced in Fig. 4.40b. The values are all greater than unity but generally close to it. The under-predictions indicate either that, as suggested above, the soil after local yield progresses through a plastic stage before the onset of local failure (a response of $\Delta u/\Delta\sigma_v > 1$ is possible in the plastic phase), or that a small degree of post-peak softening occurs in the soil, as discussed in Section 4.5.6.

4.8 DEFORMATION ANALYSES

4.8.1 Basic philosophy

It was pointed out in Section 4.1 that recent advances in numerical analysis now make it possible for complete solutions to be obtained to the complex boundary value problems that arise in geotechnical engineering. It was argued that full account should be taken of our present understanding of soil behaviour, and of realistic soil models that provide a rational framework for integrating shear strength with consolidation, and which can be used for acceptable deformation analyses.

In any design situation, such as that concerned, for example, with the construction of a road embankment on soft ground, estimates need to be made of both the magnitude and rate of settlement together with checks for overall

stability. Ideally, both classes of calculation should be carried out with one method of analysis and one soil model, with one consistent set of parameters. It is the Authors' belief that failure conditions cannot yet be properly reproduced in a finite element computation, and that estimates of deformations and of failure height are at present best made by separate analyses. It is urged, however, that the two classes of analysis are carried out with *one* soil model, and in terms of *effective* stresses, although at this stage of development of these ideas it is clearly prudent to carry out alternative calculations using more traditional methods.

Classical calculations of short-term stability of soft clays have required the choice of a unique value of undrained shear strength for each element of soil along the postulated failure surface, and these have been conducted in terms of total stresses. It is now clearly recognized, however, that the shear strength of soft clay depends on many factors. One of these is the *effective* stress changes it experiences, and these, in turn, depend on the *direction* of the principal effective stress increments (because of the anisotropic response of the clay). This aspect of soil behaviour has been well summarised by Ladd et al (1977). In addition, progressive failure and creep influence the actual shear stress experienced by the soil elements along a failure surface at the moment when failure occurs. In order, therefore, to carry out rational stability analyses, account must be taken of the deformations and effective stress changes that occur prior to failure.

This philosophy has led to the development at Cambridge of: (1) a family of elasto-plastic work-hardening models of soil behaviour, specified in terms of effective stress parameters, and (2) a purpose-made finite element program which can be used readily with any of the models at the choice of the operator.

4.8.2 Predicted performance of an embankment on soft ground

The M.I.T. Foundation Deformation Prediction Symposium held in November 1974 provided an ideal opportunity for the relevance and use of these novel models to be assessed in a real engineering situation and, furthermore, to be compared directly with other, more conventional, models and analyses.

A full-scale field trial of an instrumented embankment, loaded rapidly to failure, was carried out. Prior to the trial, ten different groups (drawn from consulting firms and universities) made genuine Class A predictions (Lambe, 1973) of excess pore pressures and deformations at specified points in the ground (Boston blue clay), which could be subsequently compared with the observed values. Full details of the symposium are given in two reports by Constructed Facilities Division, MIT (1975), and the Cambridge predictions have been described separately by Wroth (1977).

The basic model used by Wroth was the plane strain version of 'modified Cam-clay' developed by Simpson (1973). The computed predictions proved

to be very satisfactory and, as a self-consistent set, were as close, if not closer, to the observed field behaviour than any of the other predictions. The majority of the other predictions were based on elastic models of the Boston blue clay, some linear, some non-linear, but all were in terms of total stresses only. In contrast, the finite element computations based on the Cam-clay model are carried out in terms of total stresses (to satisfy equilibrium) and effective stresses (for soil behaviour), so that the excess pore pressures are calculated automatically.

The Cam-clay model also has the major advantage that it requires five basic soil parameters for a complete description of the behaviour of all elements in a particular stratum; the model does not require the engineer to exercise judgement about how the stiffness of the soil may vary with depth or with the level of effective stress. Moreover, the five basic parameters can be obtained from one representative and good quality consolidation test and one drained triaxial compression test. The use of any non-linear soil model requires estimates to be made of the in-situ stress state of all elements of soil represented in the computation and, in particular, of the in-situ horizontal effective stress, σ_h'.

It is well documented that the values of certain deformational characteristics of soils measured in the laboratory and the field, particularly undrained moduli, may differ considerably. These differences arise in part from poor procedures in sampling, transportation, storage and testing of laboratory samples, from problems with stress relief, and from the adoption of incorrect stress paths in testing. It is clear that research is needed in these areas, as laboratory testing must always be a basic part of any soils investigation. It is clear, too, that in-situ field testing should be an integral part of any soils investigation; for this reason, a self-boring pressuremeter has been developed at Cambridge.

4.8.3 The Cambridge self-boring pressuremeter

The overriding design requirement for the self-boring pressuremeter is that it should be capable of being introduced into the ground with the very minimum of disturbance. The method of achieving this is illustrated in the schematic diagram of Fig. 4.41. Very similar devices have been developed independently at the Laboratoires des Ponts et Chaussées in France by Baguelin & Jézéquel (1974) and by Baguelin et al (1975).

The basic Cambridge instrument consists of a cylinder that is jacked steadily into the ground while the soil that enters the open bottom of the cylinder is transported to the surface. The soil is removed by a rotating cutter and is flushed to the surface by water which is pumped down the inside of the cutter driving rods and up the annular space between the rods and the inside of the cylinder. The pressuremeter consists of an inflatable cell formed by a rubber membrane which can be expanded radially in a carefully controlled manner. Accurate measurements are taken of the internal pressure and the radial

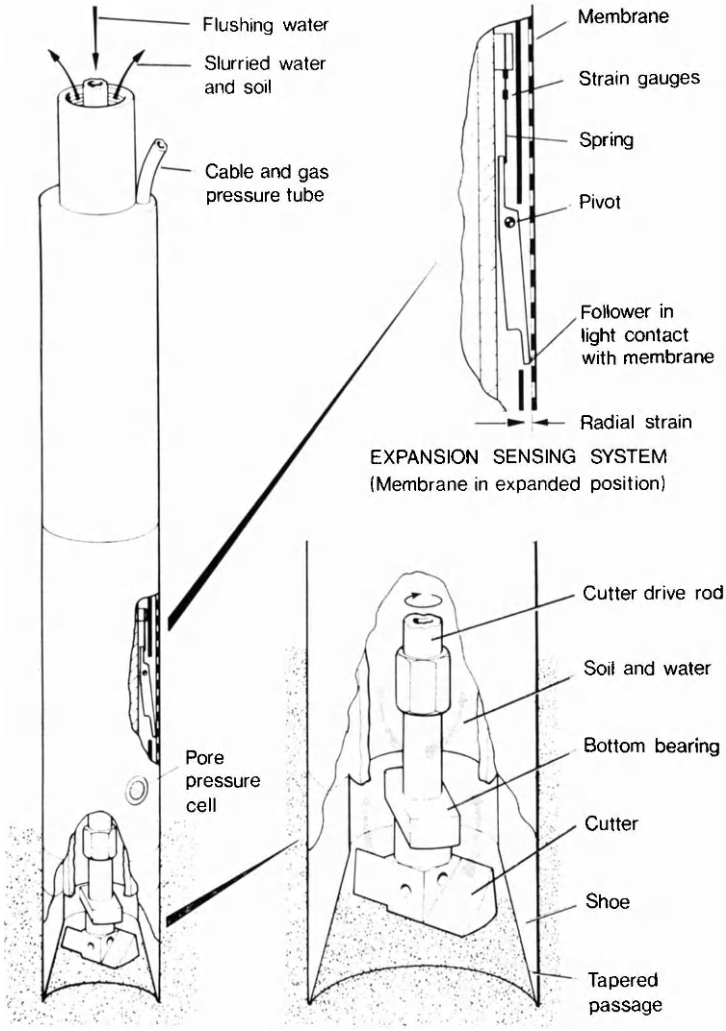

Fig. 4.41. Schematic diagram of self-boring pressuremeter.

expansion of the membrane. Fuller details of the instrument have been given by Wroth & Hughes (1973, 1974) and Windle (1976).

Typical results of an undrained expansion test in soft clay are shown in Fig. 4.42, reproduced from the paper by Hughes et al (1975), at a site at Canvey Island very close to that mentioned earlier in this chapter.

It is possible to derive the complete undrained stress-strain curve $A'B'C'D'E'$ for the clay in Fig. 4.42 directly from the observed data, $OABCDE$. From this, a value for the peak undrained strength (at point C') can be obtained. In addition, some idea of the sensitivity and of the initial stiffness of the clay is given by the shape of the curve. More accurate values of the shear modulus

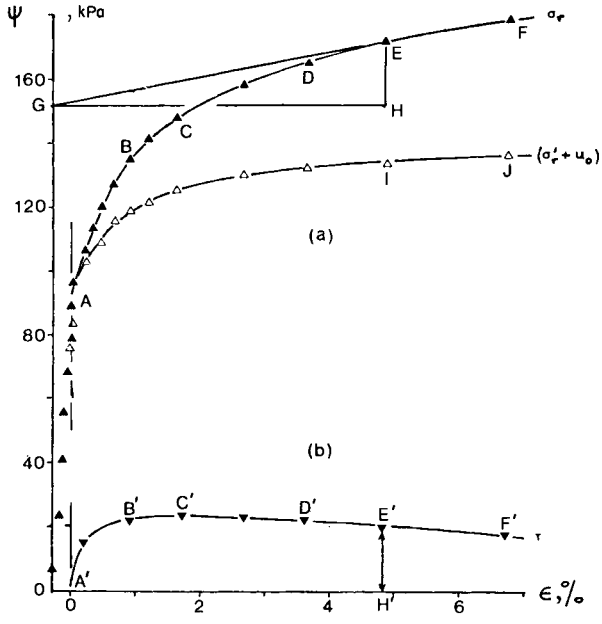

Fig. 4.42. Results of undrained expansion test at a depth of 6.1 m at Canvey Island (Hughes et al, 1975).

can be obtained by including some small unloading-loading cycles during the expansion test. The curve *AIJ* represents the value of the radial effective stress (plus the ambient pore pressure) as observed by the pore pressure cell which is fixed to the membrane and is free to move with it. The point *A* indicates the total in-situ radial stress in the ground before expansion of the soil commenced.

A series of tests was conducted in one vertical profile, and the undrained strengths are compared in Fig. 4.43 with results from in-situ vane tests, Dutch cone tests and unconsolidated undrained triaxial tests. A corresponding profile of in-situ stresses is reproduced in Fig. 4.44.

It is suggested that the self-boring pressuremeter can now be used for accurate in-situ measurement of the following properties of soft clay: (1) undrained shear strength, s_u, (2) shear stiffness, G, and (3) in-situ horizontal effective stress, σ'_h. Values of all these parameters are required for a full deformation analysis of a problem in foundation engineering.

4.9 CONCLUSIONS

Natural soft clay deposits will usually behave as lightly overconsolidated soils, with an overconsolidation ratio commonly between 1.0 and 2.5. In most soils, this small degree of overconsolidation will have been caused by ground

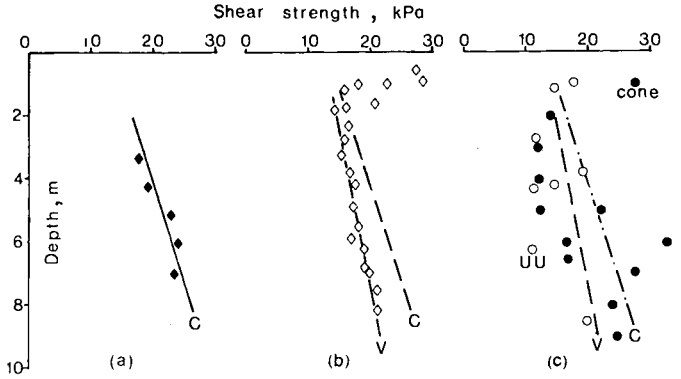

Fig. 4.43. Profiles of undrained strength at Canvey Island from (a) self-boring pressuremeter (Camkometer), (b) vane, (c) Dutch cone and triaxial tests (Hughes et al, 1975).

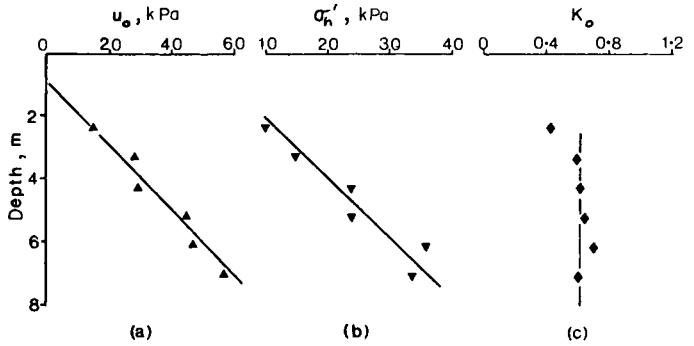

Fig. 4.44. Profiles of in-situ stresses at Canvey Island (Hughes et al, 1975).

water movements or delayed consolidation, although other factors such as weathering and near surface desiccation may have contributed. Values of K_0 commonly range from that for normally consolidated soil, e.g. 0.5 to 0.7, up to 1.2.

When a soil is unloaded to become overconsolidated, it moves into a stress region bounded by a failure line and a yield locus. When reloaded, the soil behaves elastically until it reaches a failure or a yield condition. After yielding, it continues to deform plastically, possibly with work-hardening, until it reaches failure. On reaching failure, it can continue to carry additional isotropic total stress increments if the soil does not soften, or total stress increments with $\Delta\sigma_v$ less than $\Delta\sigma_h$ if it does soften.

Theoretical pore pressure responses may show two distinct phases if the soil passes directly from elastic behaviour into a failure state, or three phases if a plastic condition develops between the elastic and failure states. In some cases, the pore pressure responses in the plastic and failure phases may be indistinguishable.

In this chapter, three published examples of pore pressure responses in the field were examined. One of these, a circular embankment loading on sensitive clay was studied in some detail, and it was found that at the end of the initial elastic phase, contained failure occurred with a distinct change in pore pressure response with further loading. The plastic phase was absent. In the second case, again a circular embankment but on soft clay of comparatively low sensitivity, the pore pressure response under loading was distinctly three-phased. In the final case record studied, a road embankment loading on Boston blue clay, a distinct change in pore pressure response occurred at the end of the elastic phase, followed by a phase in which plastic yielding, if it occurred, was not clearly distinguishable from the contained failure response.

In the elastic phase, soil can exhibit marked anisotropy, and it is found that vertical specimens under one-dimensional-consolidation are initially stiffer than horizontal specimens. Under increasing load, however, the settlement-consolidation pressure curves merge into each other and show identical behaviour for consolidation pressures greater than about three times the preconsolidation pressure for the soil.

In an attempt to assess the influence of anisotropy in a soft clay, the two factors which must be taken into account are:

(1) Under increasing effective stress in both directions, the lateral stiffness will usually be less than the vertical stiffness.

(2) In either the vertical or lateral direction, the stiffness under decreasing effective stress may be greater than under increasing effective stress.

It has been shown in this chapter that the influence of these two factors on each other may be such that, in the region underneath a surface loading, the soil may behave essentially as an isotropic elastic material, and the calculation of stress increments using elastic equations and isotropic conditions is justified. Towards the edge of the loaded area, and outside it, however, the soil behaves in a strongly anisotropic manner, and stress increments cannot be adequately calculated by the usual elastic equations.

The anisotropy which has a marked effect on the behaviour during the elastic phase also strongly influences the undrained shear strength, because it dictates the stress path and, consequently, the point where the stress path meets the failure envelope. This tends to give a decreasing undrained triaxial compression strength as the orientation of the specimen rotates from vertical to horizontal, a behaviour which has been observed by a number of workers. Other workers, however, have found inclined specimens to give lower undrained shear strengths than either vertical or horizontal specimens, and this behaviour seems to be associated with preferred particle orientation.

Although the influence of strain rate on undrained shear strength has not been considered in detail in this chapter, the point has been made that, in some natural clays, a drop of up to 15% in shear strength is observed for a

tenfold decrease in testing rate. Consequently testing rates in the laboratory should be considered very carefully and, for 38 mm × 76 mm diameter specimens, these should be less than 0.5% axial strain per minute.

The stage in the development of geotechnical engineering has been reached when it is no longer necessary nor 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 connected.

It is possible to carry out finite element computations using realistic elasto-plastic models of the behaviour of soft clay which provide acceptable predictions of field performance. It has been suggested that these isotropic models are successful for the case of loading of soft ground, which is actually anisotropic, because the major effects occur in such a way that the stress distribution is well represented by isotropic elastic theory.

Although the Cam-clay model is believed to be well suited for describing the loading of soft clay, it must be emphasised that it would not be appropriate for modelling other soils in other boundary value problems, such as a large excavation in overconsolidated clay.

There will be many instances when the problem posed to the engineer does not warrant the sophisticated analysis or computations outlined in this chapter. Even so, it is confidently suggested that the concepts that have been put forward will provide a rational framework for assessing the problem and for reaching qualitative conclusions.

ACKNOWLEDGEMENTS

Some of the ideas presented in this paper initially appeared in two internal reports for the United States Army Waterways Experiment Station, and the Authors wish to acknowledge, in particular, the stimulus and encouragement given to them by Mr S.J. Johnson to develop these concepts.

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Chapter 5

MODERN TECHNIQUES FOR THE ANALYSIS OF ENGINEERING PROBLEMS IN SOFT CLAY

by

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Chapter 5

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5.1 INTRODUCTION

As a result of the development and ready availability of high-speed digital computers, recent years have seen rapid development of new methods of analysis for geotechnical engineering problems. The power of the computer has made it possible to analyze engineering problems in great detail and with a degree of sophistication undreamed of only two decades ago. This review of modern techniques of analysis, however, is not solely concerned with these recent developments, since the Authors consider that *modern techniques* include all analytical methods which are being used effectively at the present time, regardless of the date of their development. This chapter, therefore, focuses on a variety of analytical techniques, ranging from simple semi-empirical methods and charts to sophisticated finite element methods.

Each available analytical technique, when used appropriately, is a valuable engineering tool, and an important facet of the art of modern engineering is the selection of the analytical tool most appropriate to the job at hand. The purpose of this chapter is to aid in that selection for engineering problems in soft clays, by discussion of the advantages and shortcomings of the available analytical techniques for a variety of these problems.

No matter how sophisticated an analytical technique may be, nor what degree of computational accuracy it entails, the value of the results depends completely on a correct assessment of soil properties. This fact is likely to be fully appreciated when the analytical technique is simple and the calculations quickly performed, but it is equally true when the analytical technique is complex and time-consuming.

The analytical techniques discussed in this chapter are presented on the basis of the following geotechnical engineering problems:

(1) bearing capacity of shallow foundations,
(2) settlement of shallow foundations,

(3) rate of settlement of structures,
(4) axial load capacity of pile foundations,
(5) settlement of pile foundations,
(6) lateral load capacity of pile foundations,
(7) lateral deflection of pile foundations,
(8) stability of embankments and excavations,
(9) embankment deformations,
(10) excavation bracing,
(11) permanent retaining structures, and
(12) regional settlements.

Each of these topics is considered in a separate section of this chapter.

5.2 BEARING CAPACITY OF SHALLOW FOUNDATIONS

For soft normally consolidated clays, the undrained case is almost invariably critical, and it is customary to use a total stress ('$\varphi=0$') approach to calculate the undrained bearing capacity, q_u, by means of an equation of the form:

$$q_u = s_u N_c + \sigma_{vb} \tag{5.1}$$

where N_c is the bearing capacity factor, s_u is the undrained shear strength of the clay, and σ_{vb} is the total stress at the level of the base of the foundation (outside the footing). N_c is a function of the footing shape and other factors, and it has been evaluated from plasticity theory for a variety of cases. The influence of an underlying rigid base on N_c has been considered by Livneh (1965), and correction factors have been incorporated into Eq. 5.1 to allow for this and other effects (e.g. inclination of the applied load, soil layering and finite compressibility of the soil); Vesic (1975) has given a very useful summary of these factors. For footings subjected to eccentric loading or applied moments, it is customary (although somewhat conservative) to reduce the area of the footing so that its centre of gravity coincides with the point of action of the resultant vertical load. Davis & Booker (1971) gave solutions for a strip footing on a soil whose strength increases linearly with depth; these solutions are shown in Fig. 5.1. Davis & Christian (1971) analyzed the case of an anisotropic soil and showed that, for a strip footing on a uniform anisotropic soil, the value of s_u in Eq. 5.1 may be taken with sufficient accuracy as:

$$s_u = 0.9 \left(\frac{s_{uv} + s_{uh}}{2}\right) \tag{5.2}$$

where s_{uv} and s_{uh} are the undrained strengths of the soil in the vertical and horizontal directions respectively.

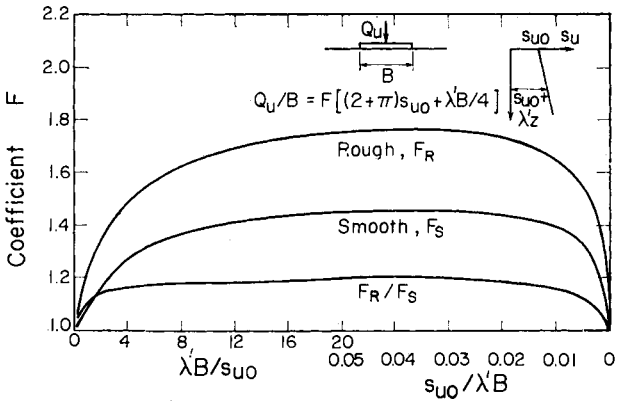

Fig. 5.1. Bearing capacity of strip footings on soil with linearly increasing undrained strength (Davis & Booker, 1973a).

Kinner & Ladd (1973) found that the use of Eq. 5.2 gave good predictions of the bearing capacity of model footings on Boston blue clay. They demonstrated the importance of overconsolidation on the clay bearing capacity, and they showed that the *SHANSEP* concept of normalized soil parameters (Ladd & Foott, 1974), which are dependent on overconsolidation ratio *(OCR)*, can be applied to bearing capacity predictions. The SHANSEP concept is also very useful in the determination of the effects of partial consolidation on the undrained bearing capacity of a foundation.

For non-homogeneous clays, solutions based on plasticity theory are more difficult to obtain, and bearing capacity has generally been calculated using slip circle or similar equilibrium analyses. While such analyses have been widely used, some caution should be exercised with slip circle analyses since they tend to give upper bound solutions and may lead to significant overestimates of bearing capacity (in excess of 100% in some extreme cases involving soils whose strength increases with depth: Davis & Booker, 1973).

The development of nonlinear finite element analyses has enabled the ultimate bearing capacity of a foundation to be determined without the necessity for artificial separation of the failure process from the deformation process. Good agreements between finite element solutions and solutions from plasticity theory have generally been obtained for simple cases using, for example, a bilinear stress-strain model for the soil (D'Appolonia & Lambe, 1970) and an elasto-plastic model (Davis et al, 1974). Problems still remain, however, in the accurate definition of the failure load from a finite element analysis because of the difficulty of obtaining convergence of the solution at loads approaching failure (Davis et al, 1974). Notwithstanding this difficulty, Desai & Reese (1970) reported reasonable correlation between calculated and measured load-settlement behaviour to failure of model footings on single and double layers of clay, using a piecewise linear representation of the stress-strain

behaviour of the soil. For practical problems, the difficulty of obtaining a solution that converges precisely on the value which would be given by classical plasticity theory may not be of significance because of the uncertainty of defining the failure load of actual foundations; this latter uncertainty, however, may arise for unrelated reasons (e.g. the development of large strains in the soil, and the influence of creep).

Small et al (1975) incorporated an elasto-plastic soil model into a finite element consolidation analysis and obtained very interesting theoretical solutions indicating the effect of the rate of loading on the bearing capacity of a strip footing. Some results of their calculations are shown in Fig. 5.2, and these demonstrate the gradual transition from undrained to drained conditions as the rate of load application decreases. Such an analysis is potentially very powerful since it removes the need to assume that conditions will be undrained, whereas in many practical cases which involve gradual construction on soft clay, significant consolidation may occur over the construction period, with a corresponding increase in bearing capacity.

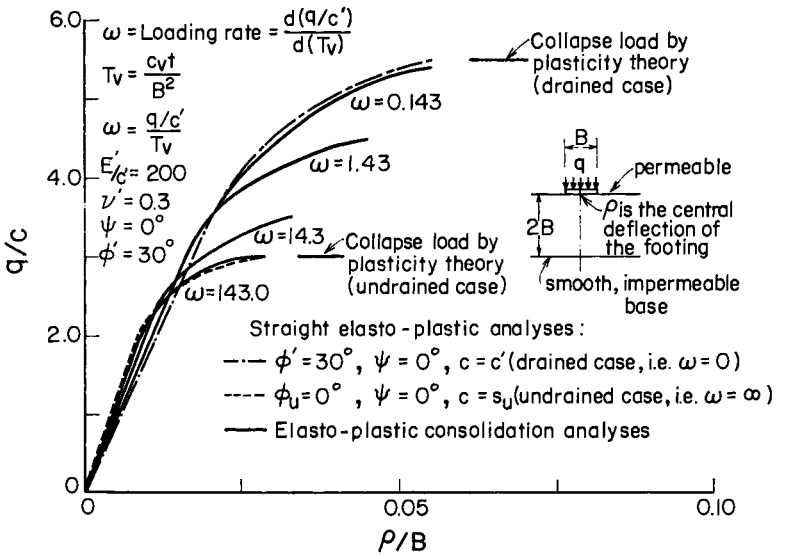

Fig. 5.2. Effect of loading rate on the load-settlement behaviour of a strip footing (Small et al, 1975).

A very comprehensive series of design charts which give bearing capacity factors for a wide variety of cases was presented by Giroud (1973). A useful summary of some of the more commonly used solutions was given by Vesic (1975), who also discussed their applicability to real problems.

5.3 SETTLEMENT OF SHALLOW FOUNDATIONS

Probably the most widely used method of settlement analysis is the conventional one-dimensional method first developed by Terzaghi. Considerable success has been reported with this method in predicting final settlements, especially for foundations on overconsolidated clays, and there has been an understandable reluctance on the part of the practicing engineer to look beyond this method. The three-dimensional nature of the settlement process and the existence of immediate (or undrained) settlements, however, have been increasingly recognized in recent years. Much of this recognition started with the appearance of the paper by Skempton & Bjerrum (1957), and it gained considerable further impetus with the development of the stress-path method by Lambe (1964, 1967) and the rational application of elastic theory (Davis & Poulos, 1963, 1968; Kerisel & Quatre, 1966). Such methods, though still essentially simplified, are more soundly based in that both shear and consolidation deformations are considered. If creep settlements are ignored for the present, the total settlement, ρ_T, is calculated as the sum of the immediate settlement, ρ_i, and the final consolidation settlement, ρ_c. Thus:

$$\rho_T = \rho_i + \rho_c \tag{5.3}$$

and at any time t, the settlement, $\rho_T(t)$, is:

$$\rho_T(t) = \rho_i + U\rho_c \tag{5.4}$$

where U is the average degree of consolidation. The settlement analysis, therefore, involves the determination of the three quantities ρ_i, ρ_c and U. The determination of ρ_i and ρ_c will now be discussed; the determination of U is dealt with in Section 5.4.

The immediate settlement, ρ_i, is usually calculated from elastic theory using the undrained soil deformation parameters, undrained Young's modulus, E_u, and undrained Poisson's ratio, υ_u (=0.5 for saturated clay). A problem arises, however, if the foundation stresses are sufficiently high to cause local yielding of the soil, since the strains in a soil element cannot then be calculated from the stresses on that element but depend on the stress state in the entire mass. A simple method of correcting elastic theory to allow for local yield effects has been described by D'Appolonia et al (1971), and this requires a knowledge of the undrained bearing capacity and the initial shear stress ratio in the soil.

Consolidation settlement, ρ_c, can be calculated in various ways. The stress-path method computes ρ_c directly from the laboratory measured strains, ε_c, as:

$$\rho_c = \int \varepsilon_c \, dz \tag{5.5}$$

If elastic theory is used, ρ_T is given as:

$$\rho_c = \rho_c - \rho_i \tag{5.6}$$

where ρ_T is the total final settlement calculated from an elastic analysis using the drained soil parameters (e.g. Young's modulus, E', and Poisson's ratio, υ'), and ρ_i is the immediate settlement calculated from an elastic analysis using the undrained parameters E_u and υ_u.

Another approach to the determination of ρ_c is to determine the shear strain, ε_s, and volumetric strain, ε_v, and then to calculate the axial consolidation strain, ε_c (assuming an isotropic material), as:

$$\varepsilon_c = \varepsilon_s + \frac{\varepsilon_v}{3} \tag{5.7}$$

The consolidation settlement can then be calculated from Eq. 5.5. A variety of linear and nonlinear relationships between shear strain and shear stress increments, and volumetric strain and volumetric stress increments, have been proposed, the most noteworthy being represented by the Cambridge 'Cam-clay' model and its derivatives. Burland (1971) has described the practical use of this approach for settlement predictions.

From a practical point of view, it is attractive to attempt to relate ρ_c to the settlement calculated from oedometer test results, ρ_{oed}, by the simple relationship:

$$\rho_c = \mu\rho_{oed} \tag{5.8}$$

Skempton & Bjerrum (1957) related μ to the pore pressure parameter A, while Simons & Som (1969) related μ to the incremental stress paths in the soil due to the foundation loading. It is also possible to obtain values of μ from elastic theory, and Fig. 5.3 shows typical solutions for μ for the centre of a circular footing on a homogeneous soil layer subjected to instantaneous uniform loading.

Another significant development in 'traditional' settlement analysis has been the application of elastic theory for a soil in which the modulus increases linearly with depth (rather than a uniform modulus, as the commonly used solutions assume). This theory was first presented by Gibson (1967), and subsequently by Gibson et al (1971), Brown & Gibson (1972), Carrier & Christian (1973) and Gibson (1974). One of the important effects of non-homogeneity is to localize the surface deflection profile around a loaded area, as compared with a soil with uniform modulus, and such a profile corresponds more closely with that measured in field problems by Burland et al (1973) and others. Another important effect is that the ratio of immediate settlement to final settlement is less than for a uniform elastic soil, as shown in Fig. 5.4. Such lower ratios are in accord with many observations (Burland & Wroth, 1974).

Possibly the most dramatic development in settlement analysis has been the advent of modern computer-oriented numerical methods, such as the finite difference method and, in particular, the finite element method. Early applications employed linear elastic theory. Two examples are the use of

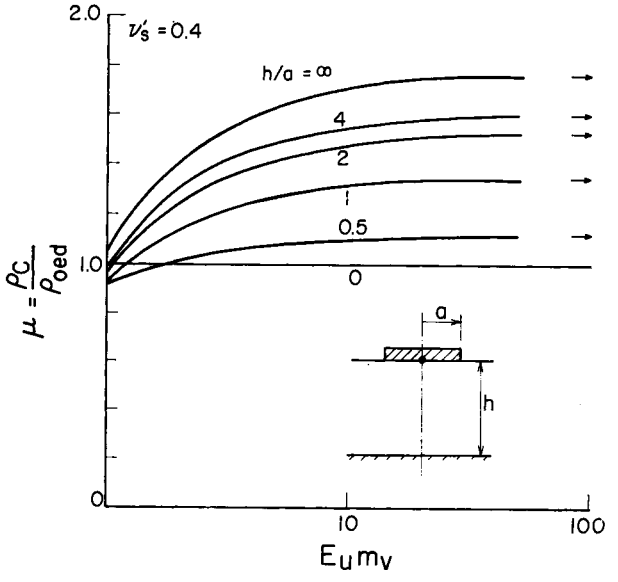

Fig. 5.3. Ratio of final consolidation settlement to settlement predicted from oedometer tests, ρ_c/ρ_{oed}, under the centre of a circular footing, as obtained from elastic theory.

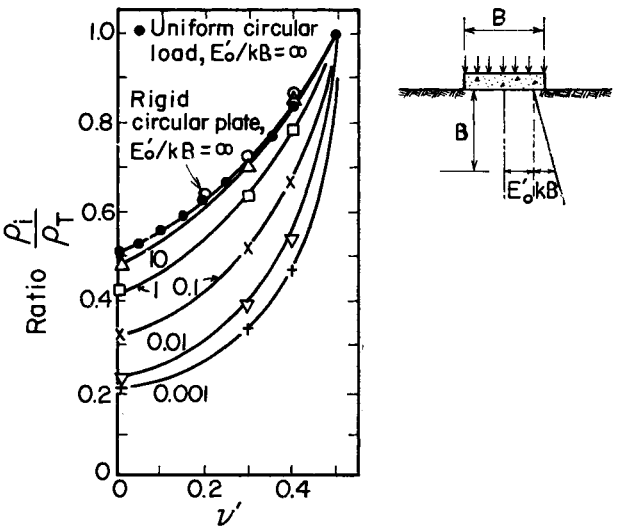

Fig. 5.4. Effect of soil non-homogeneity on the ratio of immediate settlement to final total settlement, ρ_i/ρ_T, for a smooth rigid circular plate on an elastic half-space (Burland & Wroth, 1974).

finite difference techniques for elastic anisotropic layered media (Gerrard & Mulholland, 1966), and the use of the finite element method for stresses and displacements in an elastic layer subjected to surface loading (Milovic & Tournier, 1971). The outstanding advantages of the finite element method in geotechnical engineering are: (1) the distribution of stress and displacement in the whole mass are evaluated simultaneously, (2) the method is able to take non-homogeneity into account, and (3) more realistic constitutive laws for soil behaviour can be introduced into the analysis.

Much research effort has been directed to the development of nonlinear finite element analyses, and a variety of models of soil behaviour have been employed. Among these are bilinear and multilinear laws, elastic-plastic behaviour, strain-softening laws, viscoelastic behaviour, and the Cambridge model of clay behaviour. The incorporation of such a model into a finite element analysis removes the necessity for the artificial separation of settlement and stability analyses, and it provides a means of obtaining the entire load-settlement behaviour to failure of a foundation. Many of the models, however, involve difficulties in the determination of the appropriate parameters, or difficulties associated with convergence (e.g. strain softening models) and, consequently, only simpler models have been extensively used in practice. Among these are the bilinear model (D'Appolonia & Lambe, 1970) and the hyperbolic elastic model (Duncan & Chang, 1970). The formulation of these models is generally carried out in terms of a Young's modulus and a Poisson's ratio of the soil, both of which are stress-dependent. Alternative formulations are possible, however, in terms of other parameters, such as bulk modulus and shear modulus (Drnevich, 1975).

Simon et al (1974) compared the undrained load-settlement behaviour of model footings predicted from finite element analyses based on both bilinear and hyperbolic stress-strain models for the soil. A typical comparison of their predictions is shown in Fig. 5.5, together with the measured settlement curve. In this case, the hyperbolic relationship gave a better prediction than the bilinear analysis, although the difference was not great until the stress level approached failure.

An important aspect of settlement analysis which requires further research is the prediction of creep settlements, for both the long-term case (commonly termed *secondary consolidation*) and for undrained conditions. Some simplified methods for predicting long-term creep settlements have been proposed, including one-dimensional methods (e.g. Buisman, 1936; Zeevaert, 1957) and three-dimensional approaches (Koizumi & Ito, 1964; Poulos et al, 1976). These methods involve an extension of the simplified methods of immediate and consolidation settlement analyses and the addition of a creep 'tail' to the predicted time versus settlement relationship. In cases where undrained creep is likely to be a problem, the empirical stress-vertical strain-time relationship proposed by Singh & Mitchell (1968) can be used to estimate the rate of creep settlement, and this may again be combined approximately with the conven-

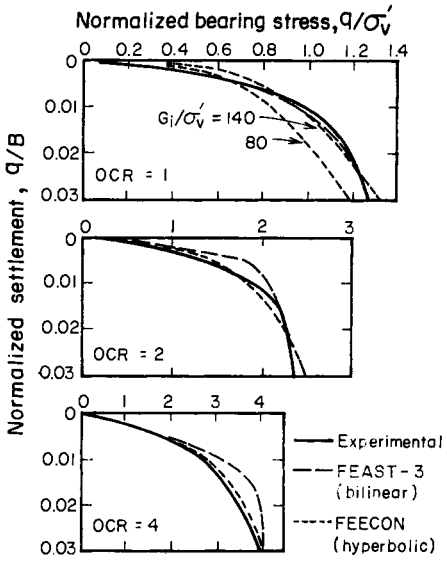

Fig. 5.5. Comparisons of load-settlement curves predicted by finite element analyses with those obtained experimentally for model footings on Boston blue clay. (Simon et al, 1974).

tional consolidation settlement versus time curve. Another approach is described by Edgers et al (1973), who carried out a succession of finite element analyses using a Young's modulus which decreased with time (in accordance with the increase in axial strain with time of a clay sample). Christian & Watt (1972) described a more refined analysis based on linear visco-elasticity. None of these methods, however, can consider the combined effects of undrained creep and local yield; these factors may give rise to very significant creep settlements for cases involving soft clays and low safety factors.

An aspect of settlement analysis which has been given increasing attention, especially in respect of the estimation of differential settlements, is structure-foundation interaction. It is now possible to link computer analyses of the structure and of the foundation, and to analyze the whole system instead of treating the two system components independently; independent treatment involves the calculation of the structural loads on the assumption that the foundation is rigid, and then the application of these loads to the foundation to estimate the settlements. Chamecki (1956) outlined a hand method of combining the structural and foundation analyses into a single analysis, but only with the advent of computer analyses of structures has the method become feasible for practical problems. For an isolated foundation, the most common procedure is to incorporate a spring of appropriate stiffness to represent the foundation in the structural analysis. For mat and raft foundations, more refined procedures have been developed (e.g. Lee & Harrison,

1970; Lee & Brown, 1972), and a great deal of research has been done into structure-foundation interaction for problems which involve dynamic loading, but a detailed discussion of these topics is outside the scope of this chapter.

The major effect of structure-foundation interaction is to reduce differential settlements between adjacent parts of the structure, with a corresponding redistribution of structural loads and moments. The extent of this differential settlement reduction is primarily dependent on the relative rigidity of the structure and the foundation. A dimensionless parametric study of the effect of structural rigidity on the settlements of a simple plane frame on various types of foundation was made by Poulos (1975b), and some typical results are shown in Fig. 5.6. The interaction effects are most marked for a relatively stiff structure on a highly compressible foundation. In practice, the major difficulty in carrying out a complete structure-foundation interaction analysis is the assignment of a stiffness to the structure, especially where substantial cladding and glazing are present. In addition, the structural stiffness varies as the construction of the structure proceeds (Heil, 1969).

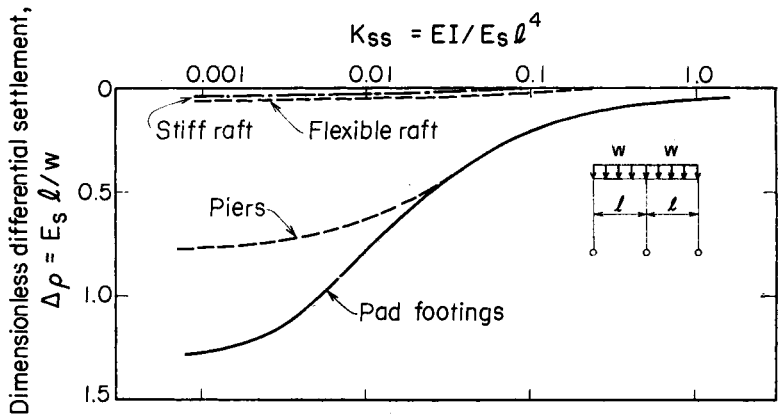

Fig. 5.6. Effect of type of foundation on the differential settlement of a simple plane frame (Poulos, 1975b).

Despite the development of sophisticated methods of settlement analysis, the use of design charts based on simpler theory can be very useful for the majority of routine calculations. These charts usually take one of the three forms:

(1) stress distributions within an elastic mass,

(2) distributions of settlement with depth (generally on the axis) of an elastic mass, and

(3) solutions for the surface settlements of loaded areas on an elastic mass.

Collections of such charts based on the theory of elasticity have been provided by Giroud (1973) and Poulos & Davis (1974), while other useful charts for inhomogeneous elastic soils have been given by Brown & Gibson (1972) and Carrier & Christian (1973).

Many instances of the successful use of charts have been reported, and Butler (1974) demonstrated how a relatively simple consistent approach by means of elastic theory can give surprisingly good predictions of settlements of a considerable number of foundations on overconsolidated clay. Davis & Poulos (1975) and Poulos (1972a) found that a relatively simple analysis, discretely applied, gives settlements similar to those obtained from finite element analyses. Indeed, Hawkes et al (1976) have shown that the simpler analysis predicts the undrained settlement of an embankment more closely than a finite element analysis employing a bilinear stress-strain relationship.

The preceding examples suggest that, for many practical cases, the critical factor upon which the success of a settlement prediction depends is the input soil deformation data rather than the sophistication of the analysis method.

5.4 RATE OF SETTLEMENT OF STRUCTURES

The major basis for the estimation of rate of settlement has been the original Terzaghi one-dimensional theory. Much of the subsequent work on consolidation and the rate of consolidation settlement has been concerned with one-dimensional analysis, and many refinements have been made to the original Terzaghi theory. The development of numerical methods of analysis, particularly the finite difference method (Gibson & Lumb, 1953), stimulated the incorporation of realistic models of soil profiles and soil behaviour into consolidation analyses. There are published examples of the modelling of layered soils (Davis & Lee, 1969), nonlinear soil behaviour (Mikasa, 1965; Viggiani, 1973; Davis, 1971; Olsen et al, 1974), secondary consolidation or creep (Wu et al, 1966; Garlanger, 1972) and problems which involve large strains (Gibson et al, 1967; Mesri & Rokhsar, 1974; De Simone & Viggiani, 1976). Some of these modifications in soil behaviour produce departures from the Terzaghi theory but, with the exception of soil layering (which can have an enormous effect on settlement rate), these departures are not perhaps as dramatic as might have been expected. For example, the consideration of nonlinear soil behaviour in which permeability and compressibility are taken to be functions of effective stress (Davis & Raymond, 1965) leads to a rate of settlement which is identical to that from the Terzaghi theory, although the excess pore pressure distribution differs considerably.

In many circumstances, the effects of two- or three-dimensional consolidation can be far more significant than the effects of nonelastic soil behaviour. Problems involving two- and three-dimensional strain and drainage conditions have been solved either by the use of the simple diffusion theory of consolida-

tion (Terzaghi, 1943) or by means of the complete theory of consolidation formulated by Biot (1941). The former theory considers only the flow of pore water through the soil mass, whereas the Biot theory considers consolidation as a process in which the pore water movement is coupled to the soil deformation. With both theories, numerical methods are generally necessary to obtain solutions to problems of practical interest. Finite differences have been used to solve the diffusion theory for problems such as consolidation due to sand drains (Richart, 1959; Krizek & Krugmann, 1974) and the rate of settlement of strip and circular footings on a finite layer (Davis & Poulos, 1972). Such solutions, although based on the simplified consolidation theory, have been shown to agree sufficiently closely for practical purposes with solutions from the complete Biot theory; Fig. 5.7 shows a typical example of this agreement.

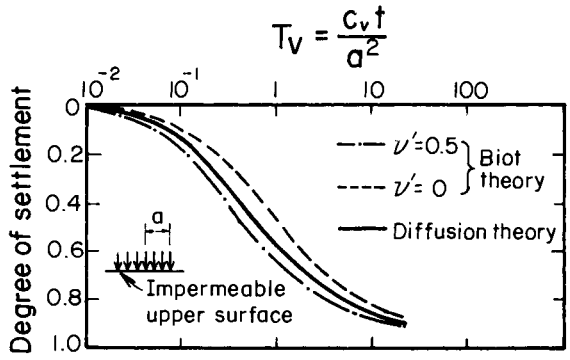

Fig. 5.7. Comparison of rates of settlement predicted by Biot theory and diffusion theory for a circular load on a semi-infinite mass (Davis & Poulos, 1972).

Several finite element formulations to solve the Biot consolidation theory have been developed (e.g. Sandhu & Wilson, 1969; Christian & Boehmer, 1970; Christian et al, 1972; Hwang et al, 1971). Problems which involve soil layering and anisotropy are readily solved, and it is also possible to incorporate nonlinear stress-strain properties. For example, Ohta & Hata (1973) used the Cambridge concept of soil behaviour, while Lewis et al (1976) outlined how a hyperbolic stress-strain relationship can be considered. Small et al (1975) employed an elastic-plastic soil model to obtain the complete time-settlement behaviour of a foundation, including the effects of undrained local yielding, and they investigated the effect of rate of loading on the load-settlement behaviour to failure.

One of the significant results of two- and three-dimensional consolidation analyses is that the existence of anisotropic permeability in a soil can have a profound effect on the rate of settlement. The significance of silt layers in clay on the rate of consolidation around sand drains was pointed out by Rowe (1964), and foundations on a clay containing silt layers or varves can be

similarly affected, the rate of settlement being accelerated by an order of magnitude in some cases (Davis & Poulos, 1972; Lacasse & Ladd, 1975). Figure 5.8 illustrates the dramatic effect of anisotropic permeability on the rate of settlement of a circular footing; the neglect of this factor may explain several instances in which the predicted rate of settlement has been far slower than that observed.

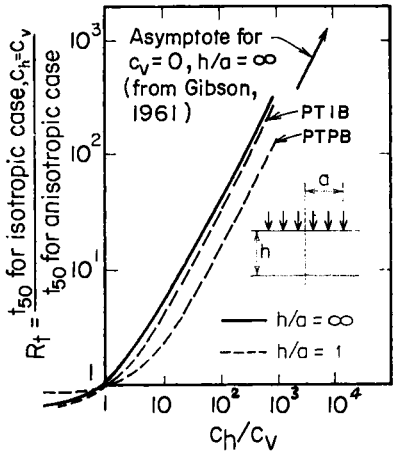

Fig. 5.8. Effect of anisotropic permeability on the rate of settlement of a circular footing on an elastic layer. (PT = permeable top; PB = permeable base; IB = impermeable base) (Davis & Poulos, 1972).

Parametric solutions for rates of settlement of shallow foundations of various shapes have been presented by Gibson et al (1970), Yamaguchi & Murakami (1973) and Davis & Poulos (1972). These solutions are restricted to uniform soil layers, and a certain amount of idealization may therefore be necessary in their application to practical cases; for example, a layered soil profile may be transformed approximately into an equivalent single uniform soil layer by the method described for one-dimensional cases by Davis & Lee (1969). As well as being directly useful, some of the charts may give an indication of the acceleration of the rate of settlement because conditions are two- or three-dimensional rather than one-dimensional. Allowance for two- and three-dimensional effects may then be possible by adjustment of the value of the coefficient of consolidation, c_v, and by employment of a simpler one-dimensional analysis; such a procedure may be very useful where facilities for a one-dimensional analysis only are available, and for cases in which the surface rate of settlement on the axis of a foundation is of main concern. An example of the successful use of this approach has been described by Poulos (1972a).

5.5 AXIAL LOAD CAPACITY OF PILE FOUNDATIONS

Attention is confined here to static methods of analysis, and consideration is not given to dynamic methods such as pile driving formulae or wave equation analyses. Almost all static approaches calculate the ultimate load capacity, Q_u, of a single pile as:

$$Q_u = Q_s + Q_p \tag{5.9}$$

where Q_s is the ultimate skin (shaft) resistance, and Q_p is the ultimate point (base) resistance. A total stress approach is most frequently adopted to calculate Q_s and Q_p for piles in clay, the assumption being made that the critical case occurs for undrained conditions. With this approach:

$$Q_s = \int_0^L c_a C \, dz \tag{5.10}$$

and

$$Q_p \approx A_p s_u N_c \tag{5.11}$$

where c_a is the ultimate pile-soil shear strength (adhesion), C is the pile circumference, L is the embedded pile length, A_p is the area of the pile base, s_u is the undrained shear strength of the clay in the vicinity of the tip, and N_c is a bearing capacity factor (generally taken to be 9 on the basis of analytical and experimental evidence).

The ultimate adhesion, c_a, between the pile and the clay is generally related to the undrained shear strength of the clay by the simple relationship:

$$c_a = \alpha \, s_u \tag{5.12}$$

where α is the *adhesion factor*. Much of the uncertainty in the calculation of ultimate pile load capacity lies in the selection of a value of α. This can be back-figured from the results of pile load tests. Accumulated data suggest, that in soft clays ($s_u \leqslant 50$ kPa), α is about unity; for stiffer clays, however, α appears to be very variable and may be influenced by the pile geometry and the nature of any strata overlying the stiff clay (Tomlinson, 1970).

Chandler (1968), Vesic (1969) and Burland (1973) have advocated the use of an effective stress approach rather than a total stress analysis. Vijayvergiya & Focht (1972) described an approach which involves the use of effective stresses and undrained shear strength and which has been applied to ultimate load predictions for offshore piles. While an effective stress approach is more fundamental, the uncertain effects of the installation of the pile make it difficult to determine the in-situ lateral stresses between the pile and the soil (which is a key quantity in the analysis). Correlations presented by Burland (1973) have proved of value, however, and recent analyses of the effects of pile driving by Carter et al (1979) promise to provide a more rational basis of effective stress analysis of driven piles.

The finite element method has been used to estimate the load-settlement behaviour of piles to failure (e.g. Ellison et al, 1971; Balaam et al, 1975), but its use almost invariably involves the adoption of special interface elements at the pile-soil boundary and, hence, it is still necessary to specify values of pile-soil adhesion along the shaft. The ultimate shaft load, therefore, is still as dependent on these input adhesions as is the conventional static analysis. Esu & Ottaviani (1975) also argued that, when interface elements are used in analyses of bored piles in clay, it is difficult to determine the correct parameters for these elements. They also questioned whether failure takes place at the interface rather than within the soil next to the pile, and they carried out analyses in which the undrained shear strength of the clay immediately around the pile was reduced by an adhesion factor. By this approach, together with a hyperbolic stress-strain relationship, they obtained good agreement with one of the experimental curves reported by Whitaker & Cooke (1966) provided that appropriate soil parameters were used; the measured and calculated curves are shown in Fig. 5.9, together with the axial load distributions and the progress of failure along the pile. In this case, the failure starts near the head and base of the pile and extends gradually along the pile as the load

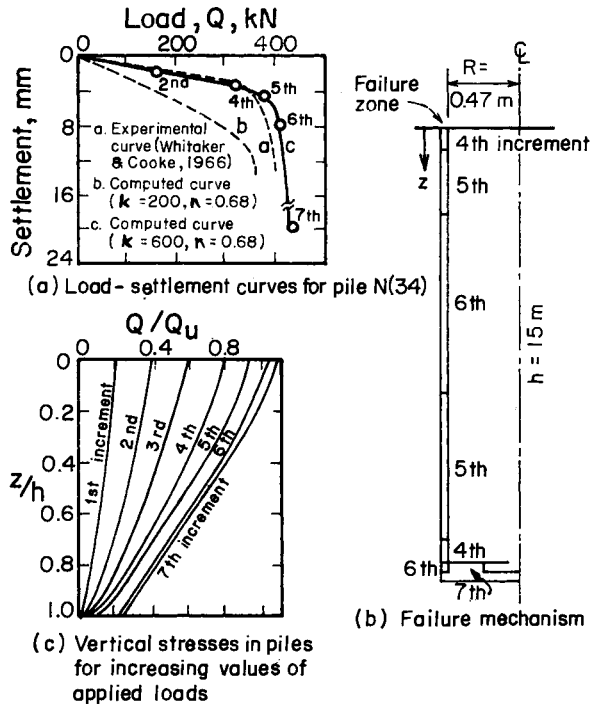

Fig. 5.9. Results of a finite element analysis of a single pile (Esu & Ottaviani, 1975).

increases. As found by previous investigators, only a small fraction of the applied load is carried by the pile base, even at loads approaching the ultimate.

For a large diameter pier or pile which derives a significant proportion of its ultimate load capacity from the base, finite element analyses (and other theoretical methods of load-settlement analysis) have confirmed earlier experimental observations that the ultimate capacity of the shaft is developed much more rapidly than the base capacity. Different safety factors may be appropriate, therefore, for the shaft and base resistances. A more rational basis for the design of larger diameter piles is the use of a settlement criterion rather than the more usual criterion of a specified factor of safety against overall failure of the pile; a load-settlement analysis by the finite element method or an elastic-based method is then very useful.

The estimation of ultimate load capacity of a pile group in clay is generally done by the application of some efficiency factor (less than unity) to the sum of the computed ultimate loads of the piles in the group. The assessment of the group effect and the efficiency factor is still largely empirical, with extensive use being made of empirical formulae such as the Converse-Labarre equation, or empirical rules for load capacity reduction (see, for example, Morgan & Poulos (1968) for a discussion of some of these), or the results of small-scale model tests by Whitaker (1957). Wide use is also made of the 'equivalent block' approach described by Terzaghi & Peck (1967). An empirical approach which appears to give good agreement with the results of small-scale model tests is the estimation of the ultimate group load capacity, Q_{uG}, as:

$$\frac{1}{Q_{uG}^2} = \frac{1}{Q_B^2} + \frac{1}{(\sum Q_i)^2} \tag{5.13}$$

where Q_B is the 'equivalent block' bearing capacity, as determined from the Terzaghi & Peck method, and $\sum Q_i$ is the sum of the individual single pile ultimate loads.

It is conceivable that three-dimensional nonlinear finite element analyses could shed some light on the interaction between pile bases at failure, although such analyses are likely to be prohibitively expensive, since a fine mesh of elements is essential in the vicinity of each pile if a reasonable indication is to be obtained of how interaction influences the pile-soil adhesion as the pile spacing varies.

In summary, the existence of sophisticated methods of analysis appears to have had relatively little impact on the calculation of ultimate load capacities of single piles or pile groups in clay. Improvements in the predictive capability of ultimate pile loads, therefore, will probably come largely from more precise methods of determining pile-soil adhesion (including the effects of group action) to put into the simple static theory rather than from the development of new theoretical methods of analysis.

5.6 SETTLEMENT OF PILE FOUNDATIONS

The calculation of settlements of pile foundations has received great stimulus from the recent development of numerical methods, and the approaches developed for pile settlement analyses can be broadly grouped into the three classes:

(1) 'step integration' approach,

(2) elastic approach using boundary elements, and

(3) finite element method.

The step integration approach was first described by Seed & Reese (1957) and developed by Coyle & Reese (1966). In this method, measured or estimated relationships between pile-soil shear stress and pile movement at various points along the pile are used to establish the load-deflection relationship for the pile. Nonlinear pile-soil behaviour is readily taken into account. However, the method inherently assumes that the deflection of any element of the pile is uniquely related to the shear stress on that element (this is akin to the subgrade reaction theory used in the analysis of laterally loaded piles) and, therefore, it cannot be used to analyse pile group behaviour.

The approach based on elastic theory generally makes use of the Mindlin equations for subsurface loading within an elastic half-space to establish relationships between pile-soil shear stress and pile deflection. Unlike the step integration method, the deflection and shear stress on any element of the pile are not uniquely related, since stress transmission through the mass can occur. Examples of analyses based on this approach are described by D'Appolonia & Romualdi (1963), Thurman & D'Appolonia (1965), Poulos & Davis (1968) and Mattes & Poulos (1969). Although the method initially relies on the usual assumption of elastic pile-soil behaviour, it is a relatively simple matter to modify the analysis to allow for the pile-soil slip and base failure in order to obtain a nonlinear load-settlement relationship for the pile. Modifications may also be made to allow (in some cases, approximately) for a variety of complicating factors, such as the presence of softer or stiffer layers below or along the pile, or the presence of a pile cap or an enlarged base. One of the significant advantages of the elastic approach is that it allows analysis of the effects of pile interaction within the soil and, hence, the evaluation of the settlement of a pile group. Pichumani & D'Appolonia (1967) and Butterfield & Banerjee (1971a, b) have described this type of analysis. Poulos (1968) and Poulos & Mattes (1971) evolved a simpler approach in which the settlement of a pile group may be related to the settlement of a single pile by superposition of 'interaction factors' which express the increase in settlement of a pile because of the presence of an adjacent pile. These interaction factors can be obtained by a simple extension to the single pile analysis, and a considerable number of solutions for these factors are available for idealized cases.

In the application of finite element analyses to pile settlements, the usual method of utilization may result in inaccuracies in the calculated displacements and stresses in the vicinity of the pile because of the large difference between the stiffnesses of the pile and soil. Methods to overcome such inaccuracies (apart from the use of an extremely fine mesh near the pile or the use of sophisticated elements), were described by Ellison et al (1971), who used special elements at the pile-soil interface to allow for consideration of slip, and by Balaam et al (1975), who analysed the pile and soil separately in a method which, in fact, is a generalization of the elastic method of analysis. Both of these methods can be used to estimate the load-settlement relationship to failure. An interesting feature of the load-settlement curve obtained by Esu & Ottaviani (1975) (Fig. 5.9) is its linearity up to a very high load, despite the use of a markedly nonlinear stress-strain law for the soil. This result reinforces previous indications that simple elastic analyses can successfully be used to predict pile settlements at normal working loads.

For pile groups, the axial symmetry present in the analysis of a single cylindrical pile is lost and, hence, the use of the finite element method involves a three-dimensional analysis. The only published account of such an analysis appears to be that by Ottaviani (1975), and even in that case, a purely elastic analysis involved a large amount of computer time; a nonlinear analysis would almost certainly be prohibitively expensive. Simplified approaches to the finite element analyses of groups were described by Hooper (1973) and Desai et al (1974), both of whom reduced the pile group to either an axially symmetrical or plane strain problem which could be analyzed relatively simply and inexpensively. The major problem in these analyses is the determination of an equivalent stiffness of the plane strain or axially symmetrical bodies that represent the rows of piles, especially if both lateral and vertical loading is to be applied to the piles.

Comparisons between solutions for a single pile from finite element analysis and from the elastic approach using the Mindlin equations were described by Poulos (1977b); for homogeneous isotropic soils, the solutions are in close agreement. In view of the uncertainties associated with the selection of the pile-soil parameters, the use of a finite element analysis only appears to be warranted in cases where a markedly heterogeneous or layered soil profile exists, or where stresses and displacements away from the pile are required.

A series of design charts for single pile settlements, based on the elastic approach, were presented by Poulos (1972b, 1974), while charts for the estimation of pile group settlements are also available (Poulos, 1977a). These charts give a ready appreciation of the major factors which influence the settlements of piles and pile groups.

Several comparisons have been made between measured and calculated settlement behaviours of single piles and pile groups (Poulos, 1974; Koerner & Partos, 1974). The agreement is generally good enough to suggest that the elastic method can be usefully applied to many practical problems.

5.7 LATERAL LOAD CAPACITY OF PILE FOUNDATIONS

The design of laterally loaded piles is generally governed by considerations of allowable lateral deflection rather than ultimate load capacity and, consequently, there has been less concentration of research effort on ultimate capacity than on lateral deflection prediction. Empirical approaches to the estimation of lateral load capacity appear to be confined to the determination from a lateral pile load test of the load to produce a predetermined lateral deflection at the pile head (e.g. 10–20 mm). Of the available analytical approaches, the most frequently used methods appear to be those based on a static analysis which considers lateral force and moment equilibrium of the pile at the point of failure (e.g. Broms, 1964a, 1964b; Brinch Hansen, 1961). The possibility of failure of the soil along the whole length of the pile (rigid pile or 'short-pile' failure) and failure of the pile material itself ('long-pile' failure) can be taken into account by Broms' analyses. Satisfactory predictions of the ultimate lateral load capacity of conventional single piles by such methods have been reported by Broms (1964a, 1964b) and Poulos (1973).

One area in which simple methods are inadequate is in the determination of the ultimate lateral resistance of shallow embedded foundations subjected to combined vertical and lateral loads and moment. Such foundations are finding increasing use in offshore technology. The difficulty in applying the simpler analyses results from the interdependence of the deformation mode and the failure mode. Smith (1976) demonstrated how a nonlinear finite element analysis may be useful in defining the coupling of the various modes of deformation and failure which cannot generally be categorized as, for example, simple bearing or sliding modes.

The determination of the ultimate lateral load capacity of a group of piles poses a related but even more difficult problem than the shallow pier, and only empirical or simplified methods are available. Oteo (1972) suggested an analysis based on earth pressure theory, while Poulos (1975a) proposed the use of a lateral group efficiency to be applied to the sum of the individual pile load capacities. There is very little test data available on which to base estimates of the lateral efficiency factor. Potentially useful results, however, were obtained by Yegian & Wright (1973) from a two-dimensional finite element analysis of two piles in a nonlinear soil mass. They analyzed a plan section of two rectangular piles subjected to lateral loads, and they investigated the interaction between the two pile sections and the consequent reduction of the ultimate load capacity as the spacing between the two piles decreased. The extension of such a nonlinear analysis to more than two piles could provide useful estimates of the lateral efficiency factor, at least for relatively rigid piles. For extremely flexible piles, the lateral efficiency factor is likely to be close to unity, since each of the piles will probably fail by failure of the pile material ('long-pile' failure), and interaction between the piles in the group will not significantly affect the ultimate load capacity.

The most comprehensive design charts available for single piles are those given by Broms (1964a, 1964b, 1965), which consider both the cases of uniform clay with constant cohesion and uniform sand with constant unit weight. For each case, 'short-pile' and 'long-pile' failure are considered for both a free-head (unrestrained) pile and a fixed head (restrained) pile. The beneficial effects of head fixity on lateral load capacity are clearly indicated by these solutions.

5.8 LATERAL DEFLECTION OF PILE FOUNDATIONS

Empirical methods for the estimation of lateral pile deflections still retain considerable popularity, especially among structural engineers, and these generally involve the replacement of the embedded portion of the pile by an equivalent free-standing length of cantilever which can be readily analyzed. Some of the more soundly based methods, however, have provided a more rational basis for selection of the equivalent cantilever. The most widely used of these methods are those based on the theory of subgrade reaction, in which the soil is modelled as a series of independent springs. One of the earliest applications of this approach was described by Palmer & Thompson (1948). Reese & Matlock (1956) obtained a comprehensive solution for pile deflections, rotations, moments and shears for a linear soil response and a linearly increasing soil stiffness with depth, which can be used to obtain approximate nonlinear solutions, and which has proven very useful in practice. Much research has been directed towards means of accounting for nonlinear soil behaviour, since it has been repeatedly observed that the load-deflection response of a laterally loaded pile is very markedly nonlinear.

Nonlinear analyses based on subgrade reaction theory are now widely used and developed, and a considerable amount of data has been accumulated from field tests to enable the required soil response characteristics to be used in the analysis (Matlock, 1970). These characteristics usually take the form of '$p-y$' curves, i.e. pressure versus deflection relationships at various points along the pile; such characteristics are analogous to the axial load-settlement relationships used in the step-integration analysis of pile settlement (see Section 5.6). Several instances of the successful use of the '$p-y$' approach have been reported (e.g. Reese & Welch, 1975) and, by choice of the appropriate distribution of soil resistance, excellent correlation between measured and theoretical deflections, rotations and bending moments can be obtained.

While subgrade reaction based analyses have been successfully used for single piles, they cannot be used to analyse the deflections of pile groups (except by making an adjustment to the '$p-y$' curves to allow for group effects), because they cannot consider the transmission of stress through the soil. Analyses based on elastic theory overcome this limitation and allow analysis of the interaction between piles as well as of single pile behaviour.

It then becomes possible to relate theoretically pile group deflections to single pile deflections. For single piles, the elastic analysis is readily modified to allow for soil yield adjacent to the pile. The simplest procedure is to consider a limiting pile-soil pressure at each point on the pile. The resulting nonlinear load-deflection curve has been found to be in reasonable agreement with measured behaviour except when the pile is very rigid. Some comparisons between measured and calculated load-deflection behaviour are shown in Fig. 5.10.

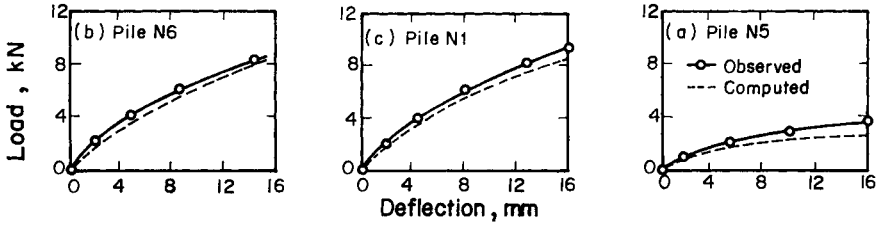

Fig. 5.10. Comparison of observed lateral pile deflections with those calculated from elastic-based analysis (Poulos, 1973).

An extension of the single pile nonlinear analysis to pile groups has been described by Poulos (1975a). An alternative means of utilization of the group analysis has been developed by Focht & Koch (1973), who combined the 'p—y' analysis for a single pile with the elastic analysis of group interaction to obtain the load-deflection relationship for pile groups.

Because of the success and widespread use of analyses such as those described above, there has not been great impetus for the application of the finite element method to this problem. Nevertheless, some analyses have been performed (Wittke, 1973; Desai & Appel, 1976), although these have proved to be expensive because of the necessity to use full three-dimensional analysis if nonlinearity is considered. Desai & Appel found that it is desirable to include interface elements in such an analysis, since pile-soil slip can have a considerable effect on the load-deflection behaviour, particularly as the lateral deflections become large and the lateral load approaches its ultimate value. A method of applying an axially-symmetrical finite element analysis to nonsymmetrical problems was described by Wilson (1965), but this has only limited application to the laterally loaded pile problem since it can only be used for a linear elastic soil model.

For groups subjected to combined vertical and lateral loading, the majority of methods in use are primarily structural analyses which consider the group as an equivalent structural system (e.g. Hrennikoff, 1950; Priddle, 1963; Francis, 1964; Saul, 1968). Consideration of the presence of the soil is generally confined to the employment of subgrade reaction theory for the determination of an equivalent bending length for the piles. Analyses based

on elastic theory, and therefore capable of allowing for pile interaction through the soil, have been described by Poulos & Madhav (1971) and Banerjee & Driscoll (1976). It is also possible to use a finite element analysis, although this will only generally be feasible if the pile group is replaced by an equivalent plane strain representation.

A considerable number of design charts are available for lateral deflections. For example, charts based on subgrade reaction analyses have been presented by Reese & Matlock (1956), Barber (1953) and Broms (1964b), while charts based on elastic theory were given by Poulos (1971a, 1973) for single piles and Poulos (1971b) for pile groups. Most charts are for simplified distributions of soil modulus with depth, generally either constant or linearly increasing, and solutions using the latter type of distribution have been found to give surprisingly accurate predictions for piles in soft clays or sand deposits. Davisson & Gill (1963) have shown that the soil near the ground surface has the major influence on pile behaviour, and Poulos (1972c) demonstrated that, unless a pile is very rigid, the nature of the soil along the lower part of the pile has little or no influence on the pile head deflection, even if the pile is socketed into rock.

5.9 STABILITY OF EMBANKMENTS AND EXCAVATIONS

Methods of evaluating the stability of embankments on soft clay and excavations in soft clay will be considered in the five categories:

(1) designs based on experience in the same or similar clay,
(2) designs based on simple rules-of-thumb,
(3) designs based on simple stability charts,
(4) designs based on equilibrium stability analyses, and
(5) designs based on finite element analyses.

The use of experience without formal analysis has the advantage of being able to account for factors which are difficult to evaluate analytically, such as the effects of construction procedure on the development of mud waves and the penetration of fill materials into the foundation clay, and progressive failure effects in sensitive clays. Test fills or unintended failures may be used directly as a basis for design decisions, or they may be used as a means for calibrating analysis procedures for use in design. As an example, Duncan & Buchignani (1973) described a failure of a 33 m high underwater slope in San Francisco Bay mud. This slope, which was steeper than 45° (0.875 horizontal on 1.0 vertical), had a calculated factor of safety equal to 1.17 at the time it failed. The experience gained from this failure can be used directly for the design of other slopes—it indicated that underwater slopes in San Francisco Bay mud should not be steeper than about 1:1. The experience can also be used as an indicator of the degree of inaccuracy involved in the methods of sampling, testing, and analysis used for design. Experience from test fills, such

as the one on Bangkok clay described by Eide & Holmberg (1972), can be employed in a similar way.

Simple rules-of-thumb may be very valuable guides to design and, although they may not be as elegant as some other analysis procedures, they are often extremely useful. For example, a rough estimate of critical embankment height can be made using bearing capacity theory; the appropriate rule-of-thumb is that the critical height of an embankment on soft clay is about 5 s_u/γ. Although this rule neglects the effects of side slopes, embankment strength, and strength variation with depth, the simplicity of such a rule makes it useful for many practical purposes. The accuracy of rules-of-thumb may be enhanced considerably by adjustments based on experience.

Slope stability charts, such as those developed by Taylor (1948), Janbu (1968), and Hunter & Schuster (1968), provide very rapid analyses of slope stability problems in soft clays. These charts are based on limit equilibrium analysis procedures which, from the point of view of mechanics, are perfectly accurate for '$\varphi=0$' conditions (Wright, 1969). Some degree of inaccuracy may be introduced by averaging the soil strengths around the slip surface, and by the adoption of average unit weights to obtain an equivalent homogeneous slope condition for analysis. If the averaging of soil strengths and unit weights is done systematically, however, the resulting inaccuracy can be minimized, and is usually no more than a few percent. In nearly every case, therefore, the degree of uncertainty involved in the use of the slope stability chart is less than the degree of uncertainty involved in the measurement of the shear strength by means of laboratory or field tests.

One of the most useful charts available for embankments on soft clay and slopes in soft clay is the one developed by Janbu (1968) (Fig. 5.11). The diagrams in Fig. 5.11b. can be used to locate the critical circle; after this has been done, a weighted average shear strength can be calculated for the slope using the number of degrees of angle subtended by each soil layer as the weighting factor. Although not reproduced here, charts are also available which can be used to incorporate the effects of water outside the slope, surcharge loads, and tension cracks (Janbu, 1968; Duncan & Buchignani, 1975).

Another chart which is useful for excavated slopes in soft clay is the one developed by Hunter & Schuster (1968) (Fig. 5.12), which includes the effect of increase in strength with depth, which is a characteristic of nearly all soft clays. By the approximation of this variation as linear, the chart can be used for rapid analyses of stability.

Lo (1965) developed slope stability charts and correction coefficients for factors of safety for slopes in clays which are anisotropic with respect to undrained strength. Charts were developed for the case where the anisotropic strength is constant throughout the slope, and for the case where the strength increases linearly with depth. The influence of anisotropy increases as the degree of anisotropy of the soil increases, and as the slope becomes flatter.

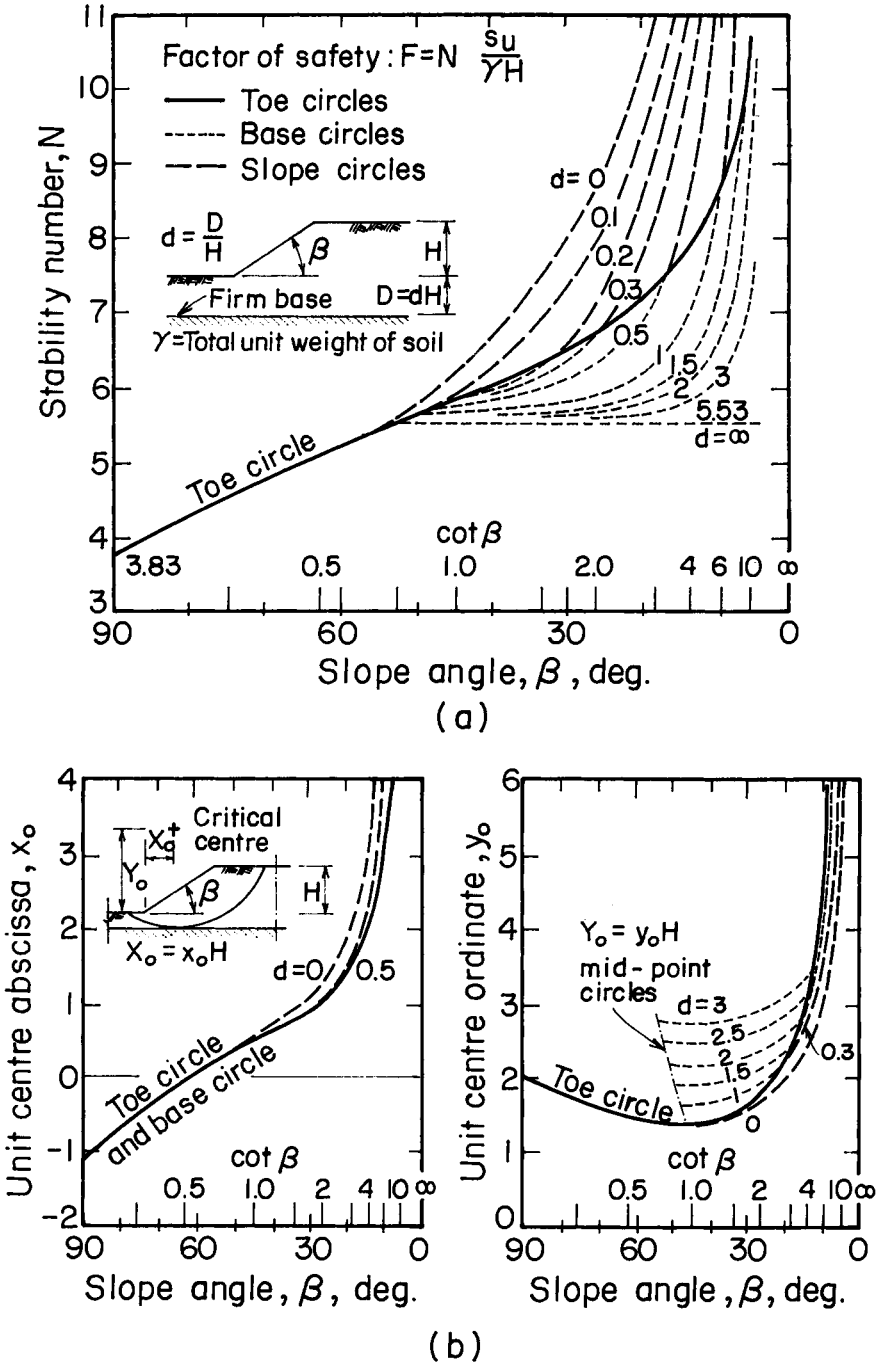

Fig. 5.11. Slope stability charts devised by Janbu (1968) for '$\varphi=0$' soils: (a) stability number, (b) coordinates of centre of critical circle.

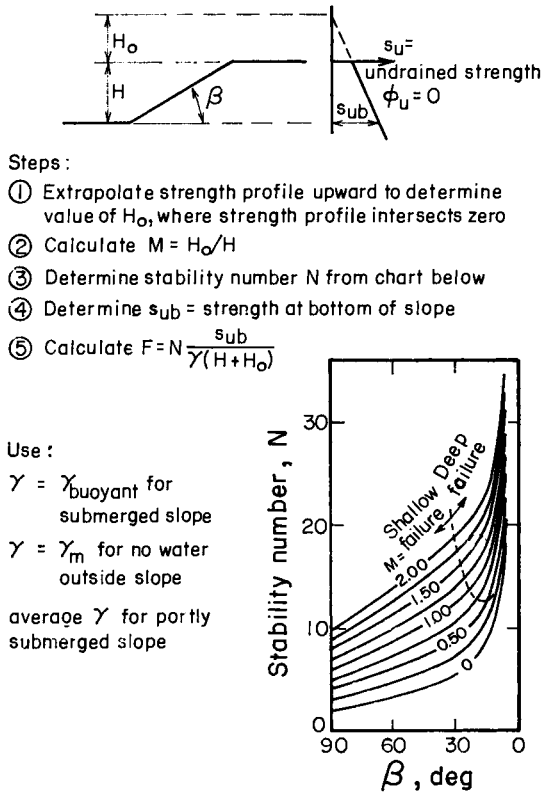

Fig. 5.12. Slope stability charts devised by Hunter & Schuster (1968) for '$\varphi=0$' soils whose strength increases linearly with depth.

Detailed equilibrium stability methods can be used for the analysis of conditions which are too complex for analysis by stability charts. The most commonly used methods for analysis of circular surfaces are the Ordinary (Swedish) Method of Slices and Bishop's Modified Method (Bishop, 1955). For $\varphi=0$ conditions, both are equally accurate, and both lead to exactly the same value of safety factor. If a particular problem involves some soil with $\varphi>0$ as well as soft clay with $\varphi=0$, Bishop's Modified Method may be somewhat more accurate, especially for conditions where the layer with $\varphi>0$ contains high pore pressures and the strength of that layer is treated in terms of effective stresses.

For some conditions, numerical problems are encountered in Bishop's Modified Method (Whitman & Bailey, 1967; Chirapuntu & Duncan, 1975). The conditions which lead to these problems are illustrated in Fig. 5.13. These often occur at the top and bottom of the slip circle where the normal stress on the shear surface assumes a large negative value. When this happens, the calculated factor of safety is unreasonably low (lower than that calculated

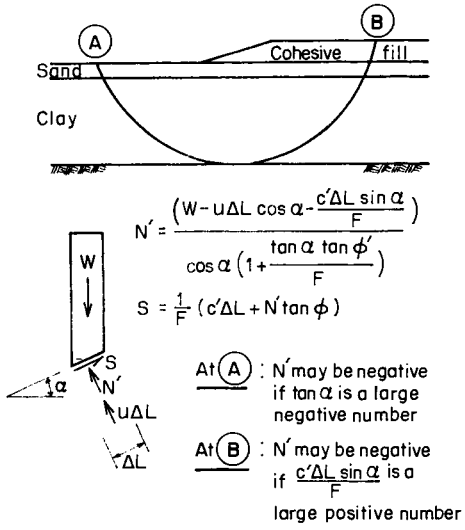

Fig. 5.13. Numerical difficulties with Bishop's Modified Method of slope stability analysis for embankments on soft clay.

using the Ordinary Method of Slices). For these conditions, it has been found that the Ordinary Method of Slices usually provides an accurate answer, as judged by comparison with the more rigorous methods of Spencer (1967) and Morgenstern & Price (1965), which satisfy all conditions of equilibrium. Thus, when these numerical problems arise in Bishop's Modified Method, the simpler Ordinary Method of Slices may be used with confidence.

For the analysis of non-circular surfaces, or wedge analyses, there are the two principal alternatives: (1) force equilibrium analyses (e.g. Lowe, 1967), and (2) methods which satisfy all conditions of equilibrium (e.g. Morgenstern & Price, 1965; Spencer, 1967). All force equilibrium analyses require an assumption about the inclination of the side forces between slices. Studies conducted by Wright (1969) showed that the side force assumption employed by Lowe & Karafiath (that the side force inclination is the average of the slope of the ground surface and the slip surface at that interslice boundary) results in factors of safety which are too high by about 10%. The assumption of a steeper inclination results in an even greater unconservative error. For $\varphi=0$ wedge analyses performed by hand calculation, it is probably best to assume that the side forces between slices are horizontal (Duncan & Buchignani, 1975). If the analyses are performed by computer, a method that satisfies all conditions of equilibrium can be employed, which eliminates the need for an assumption about the side force inclination.

The finite element method can be used to estimate movements in excavated and embankment slopes, but it does not provide any better means for evaluating factors of safety against instability than do the simpler equilibrium

methods. In fact, because the equilibrium methods are simpler and at least as accurate, their use is preferable. The use of the finite element method for estimating movements in slopes is discussed in Section 5.10.

In assessing the stability of an embankment to be built on soft clay, the engineer is faced with two important questions which must be answered before an analysis of the stability can be performed:

(1) Will the embankment crack, and should an allowance for a crack be made in the stability analysis?
(2) Will it be possible to mobilize the strength of the embankment and the strength of the fill simultaneously, or should a reduced fill strength be used as an allowance for progressive failure?

Chirapuntu & Duncan (1977) have described simple procedures which may be used to answer these questions. Using finite element analyses to study the stresses in a wide range of embankments on soft clay foundations, they developed simple charts which can be used to estimate the likelihood of cracking and the degree of strength mobilization in an embankment.

5.10 EMBANKMENT DEFORMATIONS

Problems involving the prediction of embankment deformations can generally be classified into one of two categories:

(1) Embankments which rest on soft foundations, in which case the majority of deformations come from the soft foundation (e.g. highway embankments constructed on normally consolidated clays).
(2) Embankments which rest on a relatively stiff foundation, in which case the deformations occur primarily within the embankment itself (e.g. earth dams).

Attention will be concentrated here on the first category, since the second lies outside the immediate scope of this chapter and involves a number of complications which are not normally present in the soft foundation problem (e.g. deformations due to saturation of the initially unsaturated soil and to water loading, a distinctly three-dimensional geometry, and deformations which can be strongly dependent on the sequence of construction). Nevertheless, many of the techniques described below are frequently applicable to both types of problem.

Most embankments on soft clay are relatively long and can be adequately analyzed as plane strain problems. Many of the techniques developed for settlement and rate of settlement analyses of shallow strip foundations have been applied to the prediction of embankment deformations, and extensive use has been made of one-dimensional settlement theory (e.g. Su & Prysock

1972; Holtz & Lindskog, 1972; Margason & Arango, 1972), while two-dimensional methods based on elastic theory have also been used with some success (e.g. Poulos, 1972a). In most such cases, use is made of the 'normal, loading approximation' (Perloff et al, 1967), which assumes that the stress below the embankment at the surface of the underlying clay is purely vertical and equal at any point to the height of the embankment above that point multiplied by the unit weight. It has long been recognized, however, that the ability of the foundation to deform influences this contact stress distribution and results in the development of shear stresses at the base of the embankment accompanied by a decrease in the maximum vertical stress as compared with that given by the normal loading approximation. An approximate means of determining the stresses beneath a granular embankment was described by Davis & Taylor (1962), who assumed fully plastic conditions to prevail within the embankment itself and who used plasticity theory to derive the vertical and shear stress distributions at the base of the embankment. These stress distributions were then used with an elastic analysis to determine the deformations of the underlying clay layer. Perloff et al (1967) obtained elastic solutions for the stresses within an embankment and the underlying foundation, assuming both materials to be identical.

The finite element method has found one of its major uses in geotechnical engineering in the analysis of embankment deformations. The earliest reported application was by Clough & Woodward (1967), who also simulated the process of construction of the embankment by adding elements, representing layers of fill, at successive stages of construction. The stresses and deformations by this *sequential construction* analysis were compared with those obtained from a *gravity turn-on* analysis, which represents embankment loading applied in a single stage. For linear elastic stress-strain behaviour of the embankment and foundation, the stresses were independent of the construction sequence (apart from minor numerical inaccuracies), as they should be; however, the deformations within the embankment itself were markedly influenced, although the underlying foundation deformations were almost unaffected. The reason for the different embankment deformations is not difficult to understand since, in sequential construction, deformation of a point in the embankment cannot occur until that portion of the embankment is constructed, whereas in a single-stage (gravity turn-on) analysis, all points in the embankment deform simultaneously. The relationship between the deformations from these two types of analysis has been discussed by Poulos et al (1972). If nonlinear behaviour of the embankment or foundation is considered, then both stresses and displacements are dependent on the construction sequence. However, precise modelling of the construction sequence is generally only significant for analyses of earth dams, when most of the deformations arise from those in the embankment itself.

Most practical applications of the finite element method to embankments on soft clay have involved the use of nonlinear soil behaviour. Piecewise linear

representations of the measured triaxial stress-strain behaviour were used by Clough & Woodward (1967) and Shibata et al (1976), while Hollingshead & Raymond (1971) also adopted a multilinear model for an embankment on muskeg. Davis & Poulos (1975) used an elastic-plastic model to predict deformations of an existing embankment to which fill was added after a period of about four years. The *SHANSEP* philosophy, devised by Ladd & Foott (1974) for obtaining soil parameters, was employed and very good agreements were obtained between predicted and measured settlements and pore pressure increases; it is also interesting to note that simpler deformation calculations based on elastic theory, with allowances for undrained local yield, gave close agreement with the finite element analysis values.

A comparison between the various methods of calculating embankment deformations was made by Poulos (1972a), from which it was found that relatively simple methods of settlement calculation based on elastic theory gave reasonable agreement with a finite element analysis and with recorded settlements, both immediate and long-term. Tavenas et al (1974) similarly found a reasonable measure of agreement between elastic-based methods (corrected again for local yielding) and the results of finite element analyses of three test embankments on a cemented clay, using a hyperbolic stress-strain relationship. An interesting feature of their results was that, despite the use of this markedly nonlinear stress-strain relationship, the resulting relationship between immediate settlement and fill height remained linear until the maximum shear stress level within the foundation reached about 60%; the theoretical linearity of the fill height versus settlement curve was confirmed by the measured settlements.

Stille et al (1976) used a hyperbolic stress-strain relationship and made allowance for anisotropy of the undrained modulus. Very good agreement was obtained between measured and calculated settlements and horizontal movements beneath the embankment, although it was necessary to use an undrained Poisson's ratio of 0.4 to obtain the most satisfactory agreement, a value which is not consistent with truly undrained conditions in a saturated clay. Wroth & Simpson (1972) described the application of the Cambridge 'Cam-clay' model to the finite element analysis of an embankment on a soil profile which consisted of clay and peat. Using a single set of parameters (consisting only of the compression index, recompression index, drained friction angle and void ratio at unit stress on the critical state line), they made successful predictions of the short-term and long-term settlements and horizontal movements.

Unfortunately, in the cases described by Wroth & Simpson (1972) and Stille et al (1976), and referred to above, no alternative stress-strain models were employed to allow appraisals of the improvements in the predictions over those obtainable with simpler models. Some evidence of the implications of the adopted stress-strain representation of the soil was given by Ladd (1972), who analyzed a test embankment on sensitive clay using both bilinear and hyperbolic stress-strain models, and Simon et al (1974), who carried out similar

investigations for two embankments on soft clay deposits. In the latter paper, the variation in computed deformations with varying embankment and foundation stiffnesses was investigated in one case; increases in the strength and modulus of the fill by 50% reduced the horizontal movement in the upper 6 m of clay by between 10 & 20%; the use of hyperbolic rather than bilinear soil behaviour generally resulted in larger deformations (Fig. 5.14).

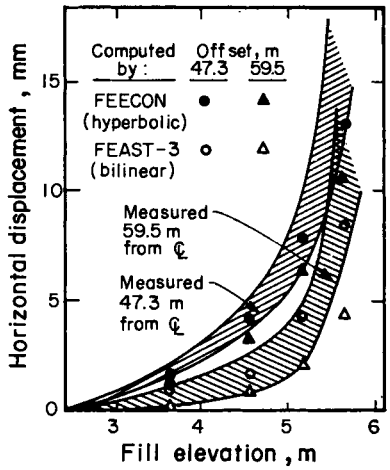

Fig. 5.14. Comparison of measured horizontal displacements beneath an embankment with those predicted by hyperbolic and bilinear finite element analyses (Simon et al, 1974).

As with its applications to shallow foundations, nonlinear finite element analysis provides, in principle, a means of determining deformations and stability of an embankment from a single analysis. Smith & Hobbs (1974) performed an interesting study of the undrained behaviour of embankments using a simple elasto-plastic soil model and an initial stress finite element procedure. It was found that these analyses gave failure loads very close to those obtained from limit equilibrium $\varphi=0$ analyses, and that the pattern of yielding was not significantly affected by the method of load application, whether 'sequential construction' or 'gravity turn-on'. The analyses also gave close agreement with the results of centrifuge tests on two model earth slopes, and Smith & Hobbs considered that the use of the simple elasto-plastic model of soil behaviour was adequate to predict the progress of yielding within and beneath an embankment. The numerical procedure adopted with the simple model was strongly convergent, and it was recommended as being suitable for immediate practical application.

Predictions of rate of settlement of embankments on soft clay are generally carried out by means of solutions or techniques for shallow foundations. The construction process and the stiffness of the embankment, however, may

influence the progress of consolidation. Smith & Hobbs (1976) described a finite element analysis for the consolidation of soft clay beneath an embankment, in which simultaneous changes in geometry, loading and material properties during and after construction can be taken into account. Biot's theory for a saturated elastic soil was employed. Two field structures were analyzed, both of which consisted of stiff pervious highway embankment fills of homogeneous cross-section founded on soft alluvial clay. For one embankment, two analyses were made, the first using two lifts to simulate the embankment, and the second using eight lifts. As shown in Fig. 5.15, the eight-lift solution produced much better agreement with measured values of both settlement and excess pore pressure. A parametric study was also made of the effect of embankment stiffness and compressibility on the distribution of excess pore pressure over the foundation cross-section, and the results of this study reveal that increased bank stiffness leads to higher pore pressures at the centre-line and lower pore pressures at the toe (Fig. 5.16). Alteration of the Poisson's ratio of the embankment from 0.4 to 0.45 had only a marginal effect, but reduction of this value to zero gave an excess pore pressure distribution similar to that for a relatively stiff embankment. Thus, for the accurate prediction of foundations pore pressures beneath an embankment, consideration of the embankment stiffness appears to be vital.

Despite the strong dependence of pore pressures on embankment stiffness, the magnitude and rate of settlement do not appear to be unduly sensitive to this factor. Some indication of this insensitivity can be inferred from the

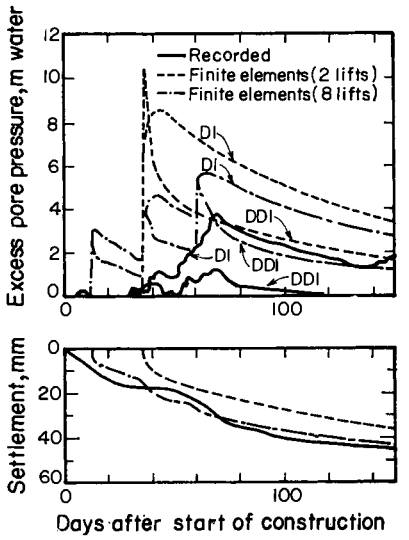

Fig. 5.15. Comparisons of observed settlements and pore pressures under the centre-line of an embankment with predictions based on finite element analysis. (DI & DDI are piezometer designations) (Smith & Hobbs, 1976).

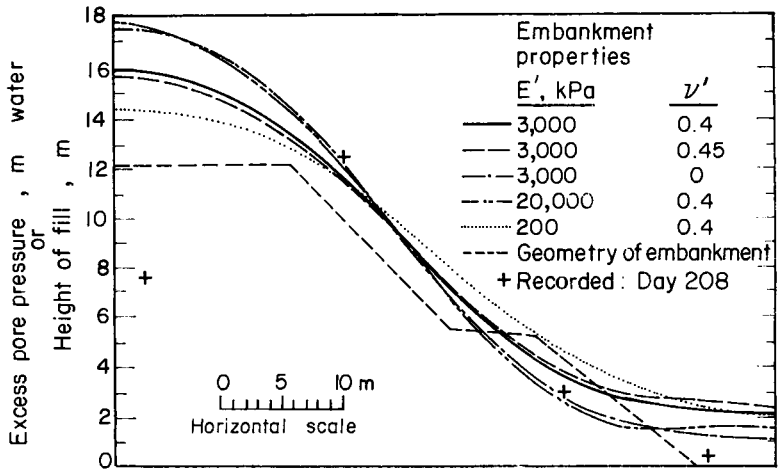

Fig. 5.16. Effect of embankment properties on excess pore pressures predicted from finite element analyses (Smith & Hobbs, 1976).

results of Davis & Poulos (1972), who found that the rate of settlement was not strongly influenced by the shape of the initial excess pore pressure distribution. Furthermore, conventional one-dimensional consolidation theory gave good settlement predictions in one of the cases analyzed by Smith & Hobbs (1976).

Time-dependent stress-strain behaviour of the soil skeleton to allow for the consideration of creep deformations has been incorporated into finite element embankment analyses by Christian & Watt (1972), Thoms et al (1976) and Akagi et al (1976). However, none of these analyses considers consolidation, and they can only be used to indicate the movements due to undrained creep or long-term creep (secondary consolidation), rather than the entire time-deformation history of an embankment.

Relatively few design charts are available specifically for embankments on soft clay. Mention has already been made of the solutions of Perloff et al (1967) for a homogeneous embankment-foundation system, while some of the solutions of Clough & Woodward (1967) have been represented in dimensionless form by Poulos & Davis (1974).

Chirapuntu & Duncan (1975) used hyperbolic stress-strain relationships to study the factors which affect the development of tensile stresses in an embankment on a clay foundation. Such tensile stresses can lead to cracking on the fill, which can cause a very significant reduction in the factor of safety of an embankment on soft clay, because the shearing resistance of the fill is reduced to zero along the crack. Their analyses indicated that the tensile stresses are greatest at the bottom of the embankment, and that cracks propagate in an upward direction from the bottom. It was also found that the height to which the fill can be built before tension develops increases with

increasing foundation strength, increasing foundation stiffness and increasing embankment width. However, increases in the unit weight of the embankment, the stiffness of the fill or the foundation depth lead to a decrease in the height of fill at which tension develops. From the analyses carried out, the following equation was developed to predict the embankment height, H_T, when tensile stresses develop in the embankment:

$$H_T = 5.1 \frac{s_{uo}}{\gamma_E} \left(\frac{K_F}{K_E}\right)^{0.75} \left(\frac{B}{D}\right)^{0.25} \tag{5.15}$$

where s_{uo} is the undrained strength of the foundation soil at the surface, γ_E is the unit weight of the embankment, K_F & K_E are the modulus numbers of the foundation and embankment respectively, B is the total embankment width at its base, and D is the thickness of the compressible foundation.

5.11 EXCAVATION BRACING

Methods for the analysis and design of bracing systems for excavations in soft clay may be divided into the three categories:

(1) Empirical rules, such as those developed by Peck (1943, 1969), Tschebotarioff (1951) and Kjaernsli (1963).
(2) Classical Rankine, Coulomb and slip circle analyses for layered soils, with empirical adjustments to the magnitude and distribution of earth pressure (Foss, 1973; Clough et al, 1977).
(3) Finite element analyses (Clough et al, 1972; Clough, 1973).

Peck (1943) first developed empirical rules for earth loads on bracing in clay based on his studies of strut loads in the Chicago subway excavations. These rules have been modified (Peck, 1969, 1973) as additional data has become available. The current recommendations (Peck et al, 1974) are summarized in Fig. 5.17. These apparent earth pressure diagrams, which are intended for fairly homogeneous conditions, indicate total loads (earth and water) for short-term undrained conditions. The diagrams provide conservative envelopes of pressure from which maximum strut loads can be calculated. For conditions where $\gamma H/s_{ub}$ exceeds 4, the strut loads exceed those given by Fig. 5.17 for reasons explained by Peck (1969, 1973) and Flaate & Peck (1973). The simplicity of these rules, and the fact that they are based on a considerable amount of field experience, makes them probably the most widely used method for the estimation of strut loads for design.

In order to take account of factors such as layered soil conditions, surcharge loads near the excavation, and unlevel ground outside the bracing, classical earth pressure theories may be used. The Rankine and Coulomb earth pressure theories give total earth loads which are about 55% less than those given by

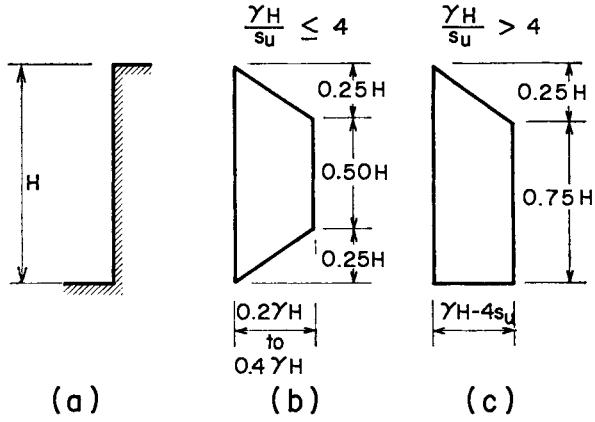

Fig. 5.17. Apparent pressure diagrams for the calculation of loads in struts of braced cuts: (a) cut, (b) diagram for clays if $\gamma H/s_u < 4$, (c) diagram for clays if $\gamma H/s_u > 4$, provided $\gamma H/s_{ub} < 4$ (Peck et al, 1974).

Peck's envelope derived from maximum measured strut loads. Slip circle calculations give higher pressures when the resultant earth load is assumed to be above the lower third point (Flaate & Peck, 1973), but they still predict total earth loads which are approximately 40% to 50% smaller than those represented by the area under the apparent earth pressure diagram based on maximum strut loads.

Foss (1973) developed empirical strut load distribution factors which he recommended for use with total earth loads calculated from classical earth pressure theory (Table 5.1). It is important to note that he used the net earth pressure load down to a depth below the bottom of the excavation, where the active and passive earth pressures become equal, rather than the earth pressure calculated considering only the soil above the base of the excavation. As

Table 5.1. Distribution of strut loads for the design of excavation bracing systems (Foss, 1973)

Number of layers of struts	Load in each layer (% of total load)					
	A (top)	B	C	D	E	Sum
2	90	90	—	—	—	180
3	30	65	65	—	—	160
4	0	55	55	55	—	165
5	0	40	40	40	40	160

Note: Total load is calculated considering active pressure on outside of bracing and passive pressure on inside. Total load = active earth load minus passive earth load, down to depth where active and passive pressures are equal. Sum exceeds 100% to allow for scatter in actual values.

indicated by Bjerrum et al (1972), consideration of the pressures in the soil below the excavation may lead to a better understanding of the behaviour of braced excavations in clays.

Clough (1975), Golder (Golder et al, 1970), Tschebotarioff (Golder et al, 1970), and a number of other engineers whose work is unpublished, employ classical earth pressure theories for complex soil and site conditions, and they adjust the calculated earth load magnitudes and distributions using engineering judgment so that they conform more closely with measured earth loads.

Clough (1973) and his colleagues (Clough et al, 1972; Clough & Tsui, 1974; Murphy et al, 1975; Tsui & Clough, 1974) have demonstrated the usefulness of finite element analyses for tied-back (anchored) and braced excavations. Results of a typical analysis for a tied-back wall in Seattle, Washington, are shown in Fig. 5.18. The method may be used to simulate inhomogeneity, nonlinear soil behaviour, construction sequence, wall and strut stiffness, and so on. It provides a means for the estimation of settlements and horizontal movements as well as bracing and strut loads. Because of the scatter in measured strut load values, design loads should be larger than those determined from finite element analyses to allow for differences resulting from variations in construction procedures and other factors.

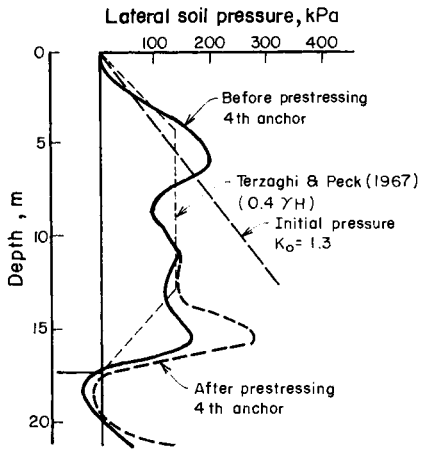

Fig. 5.18. Comparison of measured earth pressure distribution on an anchored wall with predictions by finite element analysis (Clough et al, 1972).

Clough (1973) indicated that the costs of finite element analyses are not excessively high and can probably be justified for most major projects if conventional procedures do not provide sufficient indications of possible behaviour. The major limitation on more widespread use of the method is the fact that few engineers have the experience and capability to perform the required analyses at the present time. This can be expected to change in the

future and, as a greater number of engineers develop the experience and judgment needed to apply the method practically, its potential can be more fully realized.

The most useful aspects of the finite element method for the analysis of excavation bracing are (Clough, 1973; Clough & Schmidt, 1980):

(1) It provides a means for the comparison of alternative designs and for the estimation of the behaviour of novel bracing systems.
(2) It provides a means for the estimation of movements as well as earth loads.
(3) It provides a more realistic assessment of wall bending moments than does the usual simple assumption that the wall is simply-supported between adjacent struts.

5.12 PERMANENT RETAINING STRUCTURES

For the purpose of discussion, it is convenient to consider separately the behaviour of stiff or rigid retaining structures, such as building walls and permanent retaining walls, and flexible structures, such as anchored bulkheads.

Stiff permanent retaining structures are usually designed for a combination of water pressure and 'at rest' effective earth pressure for the long-term condition (Moore & Spencer, 1972). Long-term settlements and movements may be estimated using conventional consolidation theory or finite element analyses with stress-strain parameters representative of the drained condition.

The behaviour of anchored bulkheads driven into clay was studied extensively by Rowe (1957). He found that the bending moments in anchored sheet-pile walls are overestimated by the classical 'free earth support' method of analysis, as a result of partial fixity and flexure below the dredge level. The 'free earth support' method merely consists of the assumption of Rankine or Coulomb active earth loads on the back of the wall, and partially developed passive pressures on the portion of the front of the wall which extends below the dredge level. This simple procedure provides a suitable method of determining the depth of embedment required to prevent kicking out at the toe due to inadequate passive resistance below the dredge level. Rowe's model tests and theoretical studies, however, showed that the 'free earth support' method overestimates the maximum bending moment and underestimates the load in the anchor for most conditions. Rowe recommended a simple design procedure which accounts for these factors, and this is the most accurate method currently available. This procedure involves the following steps:

(1) The 'free earth support' method is used to determine the required depth of embedment, the anchor (tie-rod) load, and the maximum bending moment in the sheet-pile wall. Stability analyses are performed to be sure that the wall is safe with respect to sliding along deep-seated

surfaces which might pass at some depth beneath the bottom of the wall.

(2) The anchor load for design is taken as 1.3 times the value calculated using the 'free earth support' method.

(3) The maximum bending moment for design is computed by multiplying the 'free earth support' maximum moment by the moment reduction factor shown in Fig. 5.19, which Rowe derived from his model tests and theoretical studies.

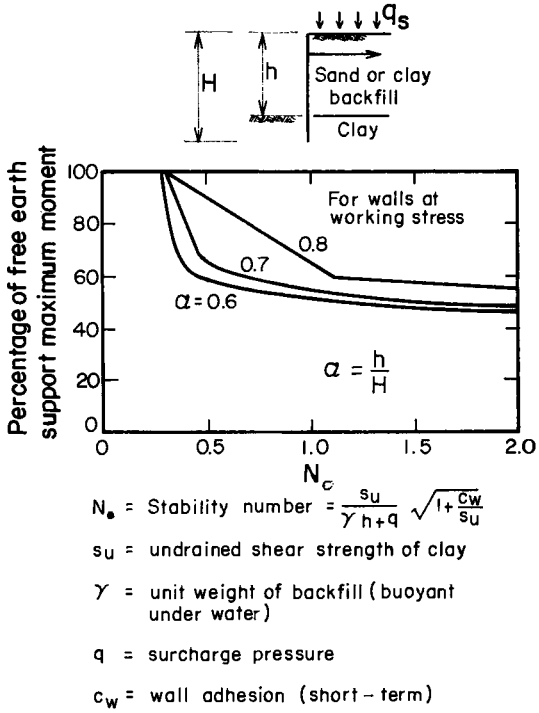

Fig. 5.19. Moment reduction factors recommended by Rowe (1957) for anchored sheet-pile walls in clay.

A number of workers (Richart, 1960; Haliburton, 1968; Rauhut, 1969; Bjerrum et al, 1972) have applied numerical methods, including the finite element method, to the analysis of the interaction between anchored sheet-piles and the surrounding soil. While these analysis procedures may provide a needed and useful means of approaching complex and unusual problems, for simple problems they appear to offer little or no advantage over Rowe's procedures.

5.13 REGIONAL SETTLEMENTS

General subsidence of large areas and throughout a number of cities has been observed, and this has been related to the effects of pumping water (or oil) from underlying permeable deposits. Examples are Mexico City (Carrillo, 1948; Hiriart & Marsal, 1969), London (Wilson & Grace, 1942), Long Beach (Mexican Soc. SMFE, 1969), Kawasaki, Japan, (Lambe, 1969). If a compressible clay layer overlies the permeable layer, the reduction in pore water pressure at the base of the clay causes a downward flow of water, a reduction in the pore pressure in the clay and, hence, an increase in effective stress; this process results in the consolidation and settlement of the clay stratum. The resulting consolidation of the clay can be analyzed approximately by consideration of the dissipation of a triangular distribution of excess pore pressure. For a uniform clay layer, a solution to this problem for the case when the pore pressure drop occurs instantaneously is available from the simple Terzaghi theory (Lambe & Whitman, 1969) if one-dimensional conditions are assumed. Carrillo (1948) also obtained a solution to this problem, and he showed that, if there is a uniform rate of drop in pore pressure in the aquifer, there is an acceleration of the rate of settlement. It is interesting to note that this phenomenon was actually observed in Mexico City (Fig. 5.20) in the period 1937–1952 and that the rate of acceleration of the measured settlement is reasonably consistent with the theoretical solution.

In many practical instances, the clay profile is layered and, in such cases, use may be made of numerical methods of solution for one-dimensional consolidation problems, both linear and nonlinear. Allowance may also be

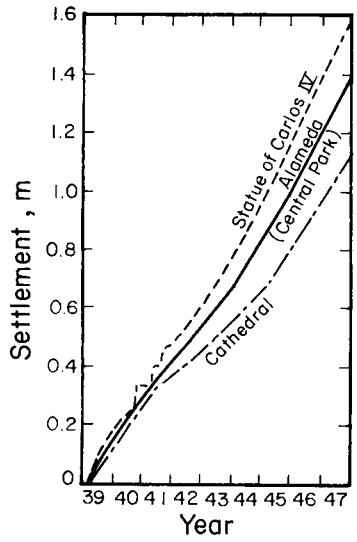

Fig. 5.20. Settlements of central area of Mexico City. (Carillo, 1948).

made for such factors as creep (secondary consolidation) and large strains; large settlements of the order of 6 m were observed in Mexico City between 1891 and 1967 (Hiriart & Marsal, 1969).

While a one-dimensional analysis may be adequate near the centre of the area affected by pumping, three-dimensional conditions will prevail in areas near the lines of wells. By consideration of the simplified two-dimensional situation illustrated in Fig. 5.21, it can be seen that the pore pressure drop, Δu, in the aquifer is dependent on the distance from the lines as well as the vertical position within the aquifer. If the aquifer is taken to be very permeable compared to the clay, the equilibrium distribution of Δu can be determined from the standard formula for de-watering of a confined aquifer (De Wiest, 1965). The consolidation of the clay, therefore, is a two-dimensional process (or, in the more general case, a three-dimensional process), and this can be analyzed by the numerical methods developed for the solution of consolidation problems. A number of solutions based on the diffusion theory of consolidation are available, primarily oriented towards irrigation and drainage problems (e.g. that of Ibrahim & Brutsaert, 1965). From a geotechnical point of view, however, the problem can be solved most effectively by a finite element analysis using the coupled Biot consolidation theory, from which deformations as well as pore pressures can be derived as a function of time. Parametric studies based on this approach do not appear to have been published as yet, although analyses for specific cases have been carried out (e.g. Lewis & Schrefler, 1979).

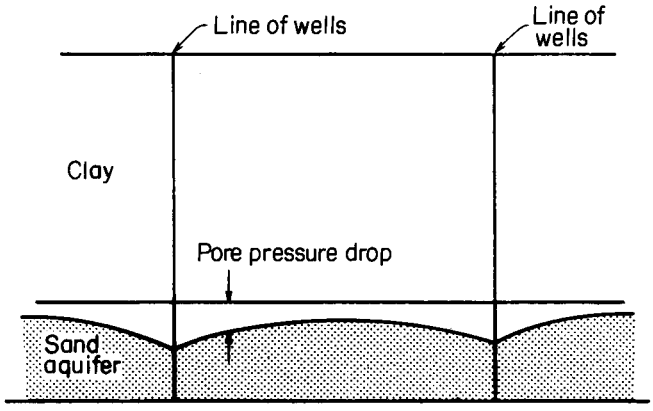

Fig. 5.21. Pore pressure decreases caused by pumping from two lines of wells.

5.14 CONCLUSIONS

As a result of the rapid development and widespread use of computers in recent years, there has been a great increase in the attention given to the analytical aspects of geotechnical engineering in soft clays, and considerable

research effort has been devoted to developing and improving analytical techniques. As a result, the number of analytical approaches which may be applied to engineering problems is much greater now than it was two decades ago.

The state-of-the-art of analysis has advanced to the point where very sophisticated analyses can be performed for a variety of problems. The costs, the time, the facilities and the expertise required, however, are often not available within the constraints of practical engineering work, particularly on small jobs. In these frequent cases, the greatest benefits of the sophisticated new analytical methods are to be derived from the results of parametric studies which have been reduced to simple chart form. This suggests that, for modern analytical techniques to provide the greatest possible benefit to the engineering profession, their continued development will require: (1) their application to specific problems to examine their suitability and to investigate the requirements for their successful application, and (2) the development of solutions in chart or tabular form which can be readily used by practicing engineers.

This review of modern analytical methods for engineering problems in soft clay shows that a considerable number of solutions are available in chart form, and the effects of many variables can be evaluated with engineering accuracy for a number of different types of practical problem. It has been the purpose of this chapter to make the selection of suitable analytical techniques easier for the engineer concerned with the solution of engineering problems in soft clays. Although they have focused on analytical methods, the Authors wish to emphasise that the accuracy of any analytical procedure rests ultimately on the selection of suitable soil parameter values. No analytical technique, no matter how sophisticated or how elegant, can provide useful results unless the soil properties employed are appropriate for the in-situ properties of the soil mass involved in the problem. When considerable effort and expense is devoted to the evaluation of soil properties, it is appropriate to employ the most comprehensive analytical techniques available. When accurate soil properties data are not available, the use of complex analytical methods is not warranted.

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Chapter 6

FOUNDATIONS ON SOFT CLAY

by

BENGT BROMS and SVEN HANSBO

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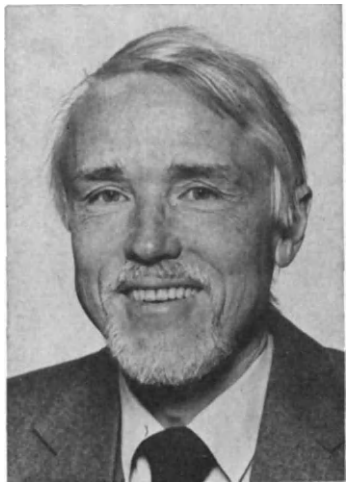
Professor Broms is the author of about 250 papers and books on reinforced concrete, rock mechanics, soil mechanics and foundation engineering. He has worked as a geotechnical consultant in more than 25 countries. During the period 1977 to 1981, he was Vice-President for Europe of the International Society for Soil Mechanics and Foundation Engineering.



Sven Hansbo has been Professor of Geotechnical Engineering at Chalmers University of Technology in Gothenburg, Sweden, since 1963. As a civil engineering graduate of Chalmers, he began his career at Gothenburg Harbour Engineering Office in 1949, returning to Chalmers in 1953 as Research Assistant, to graduate as tekn. lic. in structural mechanics in 1956. From 1956 to 1961, he worked with the Swedish Geotechnical Institute where, in 1960, he also completed a Doctorate thesis. Since 1961, he has been chief consultant in geotechnical engineering to the consulting engineering firm of AB Jacobson & Widmark.

Professor Hansbo was awarded the 1973 Prize of Honour of the Stockholm Building Society, and the 1974 Prize of Honour of the Swedish Geotechnical Society, of which he has been President since 1974. He was Chairman of the Organising Committee of the Tenth International Conference on Soil Mechanics and Foundation Engineering, held in Stockholm in 1981. He has been Vice-President of the Swedish Society of Civil Engineers since 1981.

Chapter 6

FOUNDATIONS ON SOFT CLAY

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6.1 INTRODUCTION

This chapter presents a review of pile and raft foundations in soft clays. In the first part of the chapter, the behaviour of point bearing and floating piles is treated. The second part deals with raft foundations, including pile supported rafts.

Shallow foundations are very seldom used in cases where the foundation base consists of very soft clay. Only when the foundation is placed on a thick dry crust and the thickness of the soft compressible clay below is nearly uniform can unpiled footings in some cases be recommended. Under these conditions, the stress increase in the underlying soft clay is so small and so evenly distributed that the average and differential settlements of the building caused by consolidation of the compressible clay layer can be accepted. In connection with construction activities, however, the ground level is often raised by placement of fill, or the ground water level is lowered by drainage, or trees are planted close to the building, and such activities sometimes lead to large differential settlements which seriously damage the building. It is the Authors' opinion, therefore, that shallow foundations should only be used in exceptional cases where the geological features of the clay are favourable and the future activities at the building site are completely foreseeable. Shallow foundations are therefore not treated in this chapter.

6.2 PILES

6.2.1 Pile types

Precast concrete piles are commonly used in areas where there is soft clay. In Sweden, for example, the total length of all piles driven in 1974 was about 3 million metres. Of these, 75% were precast concrete piles. In Holland, precast concrete piles with a length of about 6 million metres accounted for about 85% of all piles installed in 1970.

Precast concrete piles are usually spliced when the length exceeds 14 to 16 m. Different splicing systems have been developed in Sweden, as described by Broms (1971b), and in Mexico (Girault, 1964b). An example is shown in Fig. 6.1. The ultimate strength in bending of splices is usually equal to or exceeds that of the pile section. The reinforcement (minimum 1.5 to 2.5%) is normally governed by the stresses during the handling of the pile. The cross-section of a pile is square, circular or hexagonal depending on the requirements placed on the pile. Fellenius (1972b) summarized the use, primarily in Sweden, of precast concrete piles. Precast concrete piles with enlarged tips are commonly used in Holland to increase the point resistance and to decrease the effect of negative skin friction on the bearing capacity (van Weele, 1964). Prestressed spun concrete piles are common in Japan.

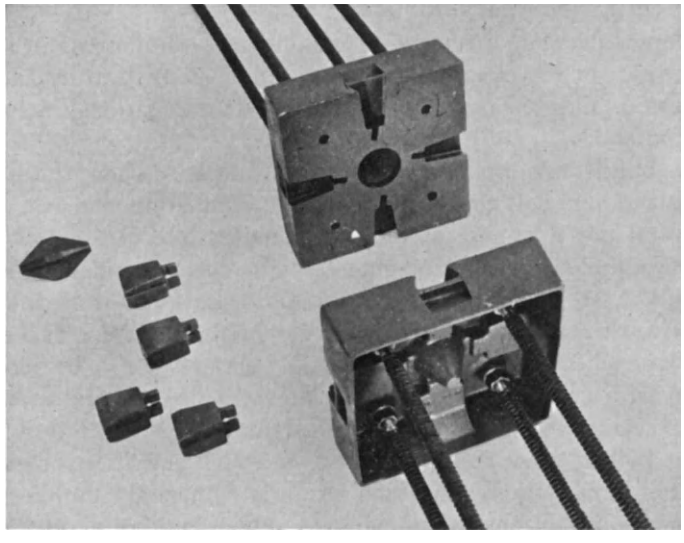

Fig. 6.1. Typical splice for concrete piles (Swedish type JF).

Cast-in-place piles are rarely used in soft clay except in countries where labour costs are low. A permanent casing is normally required because of the risk of necking, but this appreciably increases the cost and makes this type of pile expensive. Large diameter cast-in-place piles are normally only used in soft clay to support heavy concentrated loads.

Timber piles are mainly used as embankment piles or as floating piles when the depth of the compressible clay layer is large (>40 to 50 m). The tip diameter should not be less than 0.125 m. When timber piles are spliced, the top pile is often made of concrete to prevent decay above the ground water-table.

Rails and reinforcement bars are sometimes used to support light structures. The required minimum diameter of the reinforcing bars is normally 32 mm. Although bars of only 19 mm diameter have been used as piles (Fellenius, 1964), experience has shown that these are too slender for practical use.

Point-bearing steel and concrete piles are frequently provided with points of hardened steel (rock shoes) so that the piles can be driven into the underlying rock even when the surface is sloping (Bjerrum, 1957; Rehnman & Broms, 1971). The penetration depth is normally one to two times the diameter of the pile point, i.e. 60 to 120 mm. Rock shoes are also used when the soil contains stones and boulders to decrease the risk of damage to the pile during driving. Floating concrete or timber piles are generally only protected by steel rings.

The corrosion of steel piles in soft clay below and above the water-table is small because of the low permeability of the soil (Romanoff, 1962; Sandegren, 1974). Deterioration of timber piles does not occur below the permanent ground water-table, but timber piles and grillages can rot above the water level. The bearing capacity can decrease considerably within a few months if the clay is cracked and fissured.

6.2.2 Pile driving in soft clay

Precast concrete piles are usually driven by three to five ton drop hammers. The weight of the hammer is in general one to two times the total weight of the pile, or 30 to 40 times the weight of the pile per metre when the pile is long. Up to 100 m long piles have been driven successfully in soft clay (Pejrud, 1965; Hellström, 1968). Timber piles are generally driven with two to three ton drop hammers.

6.2.2.1 Stresses during driving

When a hammer strikes a pile, a stress wave is initiated which travels down the pile with the speed of sound. Its maximum intensity depends mainly on the striking velocity of the hammer (Broms & Hellman, 1971). For a concrete pile, the maximum stress is about $30\sqrt{h_e}$ MPa, where h_e is the height of free fall in m. For example, for $h_e = 0.6$ m the maximum stress at the pile head will be about 23 MPa.

Additional studies are desirable of the factors which affect the intensity and shape of the stress wave and of how the shape is affected, for instance, by cracks in the pile and by splices. The amplitude is reduced as the stress wave travels down the pile. The damping in soft clay is normally about 0.5 to 1%/m, but values of up to about 4 %/m have been observed, as illustrated in Fig. 6.2 (Fjelkner & Broms, 1972; Sovinc & Vogrincic, 1976).

In soft clay, the initial compression wave is reflected at the pile point as a tension wave. High tensile stresses can therefore develop in the top part of the pile when a light hammer is used and when the height of fall is large. The height should therefore be small in soft clay to prevent yielding of the reinforce-

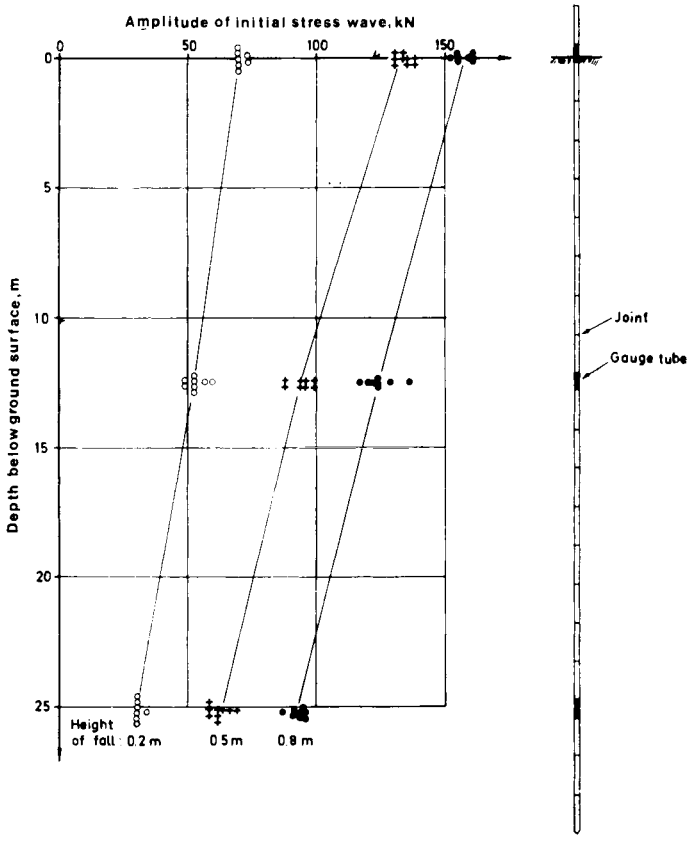

Fig. 6.2. Stress distribution in a pile during driving (Fjelkner & Broms, 1972).

ment or separation of the joints by the reflected tensile wave. Spliced timber piles are particularly affected since such piles are often spliced with short steel tubes of a low tensile strength; even a relatively low tension can then be harmful. Separation of the joints can be prevented by increasing the weight of the hammer and by reducing the height of fall.

It is likely that the bearing capacity of a point bearing pile can be estimated by studying the reflected wave from the pile tip. In order to interpret the results, it is necessary to know how such factors as height of fall, weight of hammer, splices and cracks affect the shape and intensity of the stress wave as it travels down the pile.

6.2.2.2 Effect of driving on shear strength

During driving, the soil around a pile is partially remoulded within a zone which extends to at least one to two pile diameters from the surface of the pile. The resulting high pore water pressures can reach and even locally exceed the

total overburden pressure. The reconsolidation, and the resulting increase in the shear strength of the soil after the driving, normally takes place very rapidly because of the high hydraulic gradient in the soil and the radial cracks that possibly form around the pile during the driving. The final shear strength at the surface of the pile is usually higher than the initial shear strength of the clay. Measurements indicate also that the reduction in the water content can be large close to a pile. It is common, therefore, that when piles are pulled out, they are covered by a thin layer of stiff clay, and clay layers of up to 0.1 m thick have been observed on piles pulled 30 years after driving (Fig. 6.3). This layer of reconsolidated clay probably has a large influence on the load transfer from the pile to the surrounding soil and on the settlement, particularly if the sensitivity of the undisturbed soil is high.

Fig. 6.3. Clay adhering to a pile pulled 30 years after installation.

The high pore water pressures and the resulting decrease in the shear strength of the soil caused by pile driving has been the direct cause of, or has contributed to, several landslides in Scandinavia (Jakobson, 1952; Broms & Bennermark, 1967; Aas, 1975; Slunga, 1975). Particularly affected are soft clays containing thin sand or silt seams, and reductions in shear strength of up to 50% have been observed. There are no reliable methods available today to predict this reduction in the undrained shear strength, and the present

practice for stability calculations in Sweden is to reduce the shear strength by 20 to 30% to account for the effects of pile driving.

Changes in water content and shear strength caused by pile driving have been investigated by Cummings et al (1950), Reese & Seed (1955), Seed & Reese (1957), Flaate (1972) and others. The shear strength of the remoulded clay increases very rapidly after driving (see, for example, Seed & Reese, 1957; Orrje & Broms, 1967; Flaate, 1972). An indication is the large change in driving resistance which can be noticed even after a relatively short interruption in the driving; for example, after changing the cushion in the helmet. Several blows are generally required to 'wake' the pile. In calculations, it is generally assumed that the shear strength one to three months after driving corresponds to the initial shear strength of the soil.

Orrje & Broms (1967) measured by means of field vane tests the reduction in the shear strength during driving and its subsequent increase with time. The shear strength was reduced within a zone which extended one to two pile diameters from the pile. The clay did not regain its initial shear strength within this zone. Other observations indicate that a reduction in strength can occur at a relatively large distance from a pile group (Hansbo, 1968). Flaate (1972) measured the reduction in water content and the increase in shear strength that took place after driving a 13.7 m long timber pile with an average diameter of 0.25 m. The pile was driven through a silty clay which had an initial average shear strength of 58 kPa. At some points, the shear strength increased more than 100% within a 0.15 m wide zone, while the corresponding reduction in water content was about 4%. The increase took place rapidly.

The remoulding of soil is affected by such factors as the pile length, the method of driving and the properties of the clay, mainly its sensitivity. For some quick clays, the remoulded shear strength can be so low during driving that the clay flows up along the pile and out onto the ground surface.

6.2.2.3 Pore water pressures caused by pile driving

The driving of piles causes high excess pore water pressures which generally increase with increasing shear strength of the clay and with decreasing pile spacing (Bjerrum & Johannessen 1961; Nishida, 1963; Lo & Stermac, 1965; Lo, 1968; Lambe & Horn, 1965; Hanna, 1967; Orrje & Broms 1967; Koizumi & Ito, 1967; Clark & Meyerhof, 1972; Torstensson, 1973b). The increase is normally larger for pile groups than for single piles. Koizumi & Ito (1967) measured pore pressures just after driving and found that they were three to four times the total overburden pressure. Meyerhof (1976) concluded from field observations that the maximum increase in the pore water pressure for a soft clay corresponds to 5 s_u to 7 s_u, where s_u is the undrained shear strength of the clay. Torstensson (1973b) measured an increase that was 4 s_u to 5 s_u, which is in good agreement with the theoretical values derived from an analysis of the expansion of an infinitely long cylindrical cavity in an ideal elastic-plastic medium. Lo (1968) related the pore pressure ratio, $\Delta u/\sigma'_c$, to the

sensitivity, S_t, of the clay, where Δu is the pore pressure increase and σ_c' is the preconsolidation pressure. The ratio $\Delta u/\sigma_c'$ was found to vary from about 0.5 at $S_t=1$ to about 0.9 at $S_t=50$.

Analyses and observatons indicate that the pore water pressure just at the surface of a pile can be so high that hydraulic fracturing occurs during the driving and that a system of radial cracks develops around the driven pile (Massarsch, 1976). Radial cracks have been observed, for example, by Tomlinson (1971) and by Holtz & Holm (1972). An indication of such radial cracks is the rapid dissipation of the excess pore water pressures after the driving (see Casagrande, 1947; Bjerrum & Johannessen, 1961; Orrje & Broms, 1967; Flaate, 1972; Hagerty & Garlanger, 1972; Massarsch, 1976).

The radial cracks will close when the pore water pressure around the pile corresponds to the initial lateral effective stress in the soil. Thereafter, dissipation will be slow. Drainage will then take place mainly in the radial direction away from the pile, as indicated by the reduction in shear strength of the clay between piles which has been observed by Orrje & Broms (1967) and others.

Torstensson (1975) developed a pore pressure probe with which the pore pressure increase during driving can be studied, as can the dissipation of excess pore pressures after driving. The initial excess pore pressure along a cylindrical pile was found to vary as illustrated in Fig. 6.4. For a tapered pile, the pore pressure distribution is probably independent of the distance from the pile tip.

Koizumi & Ito (1967) investigated the stress distribution in a slightly overconsolidated silty clay during driving. As expected, the largest increases in pore pressures and lateral pressures were observed when the pile tip passed the observation point. The maximum pore water pressure corresponded to the

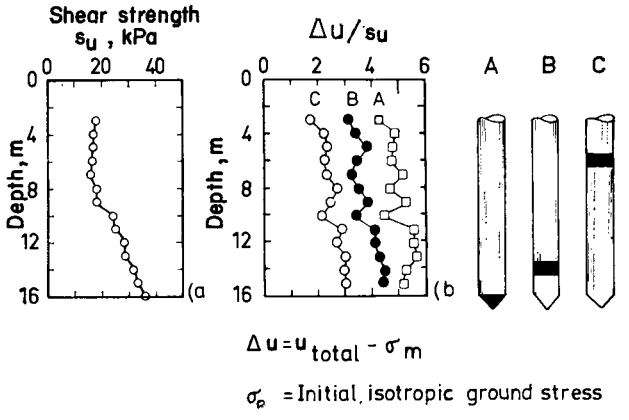

Fig. 6.4. Variation in pore pressure along a pile after driving (Torstensson, 1975) (s_u = undrained shear strength from field vane tests).

maximum total pressure. After about three weeks, the lateral effective stress was about twice the initial vertical effective stress.

Attempts have been made to reduce the pore water pressures during driving by means of *Geodrains* attached to the surface of the pile (Holtz & Boman, 1974), and test data suggest that the drains reduced the initial excess pore pressures by about 50% (Fellenius, 1975). Pore pressures during driving can also be reduced by pre-coring.

The Authors have experienced that the high pore water pressures caused by pile driving can affect the stability of a large area around a pile group, particularly if the clay is laminated or varved. For example, 0.1 to 0.2 m wide cracks caused by pile driving have been observed in slopes with an average inclination of less than three degrees. The present knowledge of the effects of the high pore water pressures caused by pile driving is limited.

6.2.2.4 Ground heave caused by pile driving

Heaves caused by pile driving can be large, especially when the spacing of the piles is small and their length is large. Surface heave can lift adjacent piles when the upward force along the upper part of a pile exceeds the pullout resistance of the lower part. Measurements indicate that heave within a pile group usually corresponds to about 40 to 60% of the total volume of the piles when the ground surface is horizontal (Casagrande, 1947; Tschebotarioff & Schuyler, 1948; Zeevaert, 1959; Klohn, 1963; Lambe & Horn, 1965; Orrje & Broms, 1967; Bergfelt, 1968; Hagerty, 1969; Torstensson, 1973a; Massarsch, 1976). The total volume of the heaved soil is often considerably less than the total volume of the piles, probably because of the rapid dissipation of excess pore pressures and the consolidation of the soil during driving. Compression of air in a partially saturated soil can be another explanation.

Usually, about one half of the total heave appears as surface heave outside the pile group (Hagerty, 1969). Heave within the foundation generally seems to vary between 60% of the total volume of the piles (Avery & Wilson, 1949) to 30% for a soft sensitive clay (Orrje & Broms, 1967). For steel H-piles, a heave within the foundation area of about 100% of the total pile volume has been observed (Adams & Hanna, 1971). Ground heave of about 0.5 m is common in deep excavations when the spacing of the piles is relatively small (i.e. less than 4 to 5 pile diameters). The heave is relatively large in deep excavations, especially if the factor of safety with respect to bottom heave is low. Massarsch (1976) observed that the bottom heave in a 2.8 m deep excavation in soft clay was twice that around the excavation at equal pile spacing and equal pile length. Massarsch also used the finite element method to study how surface heave is affected by such factors as geometry of the pile group, the sequence of driving and the excavation depth. An example is shown in Fig. 6.5, where the displacement of the soil during driving of a row of piles at the bottom of an excavation is illustrated. With the present know-

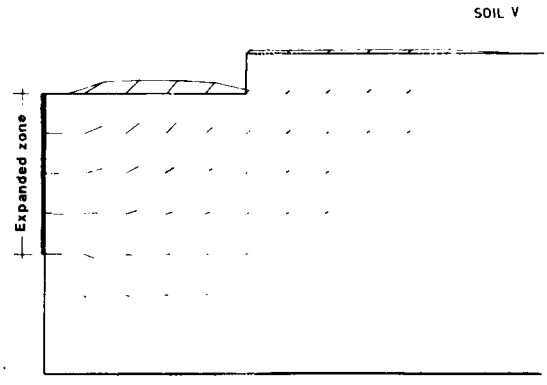

Fig. 6.5. Finite element analysis of surface heave caused by pile driving (Massarsch, 1976).

ledge, it should be possible to predict (at least qualitatively) the heave within a pile group and how it is affected by the depth of the excavation.

The driving of piles in soft clay increases the lateral pressure in the soil. The increase is large when the spacing of the piles is small. The stress distribution around a driven pile has been analyzed by several authors (e.g. Ladanyi, 1963; Nishida, 1960, 1963; Vesic, 1972). The stress increase is found to depend on such factors as the compressibility of the clay, as expressed by the modulus of elasticity, E, and Poisson's ratio, υ, as well as the shear strength of the clay. The variation in the expansion pressure is relatively small. For quick clays with a very high sensitivity, the consistency can be changed to that of a heavy fluid by remoulding during driving. In this case, the lateral pressure at the pile surface corresponds to that of a heavy fluid.

Embankment piles close to a bridge abutment are generally driven before the structural piles for the bridge are installed in order to reduce lateral displacements. Often, a V-shaped pattern is selected for driving the piles to force the soil laterally away from the axis of the road.

Heave and lateral displacements caused by pile driving in soft clay can be reduced by precoring. This method was first proposed by Arfwidson (1940). An open tube of 0.25 to 0.30 m diameter is generally used, the tube being driven 6 to 8 m into the soft clay, while air trapped in the pipe during driving is released through a vent at the top. Compressed air is injected at the bottom during withdrawal in order to eliminate any vacuum underneath the pipe. After withdrawal, the clay is forced out of the pipe by means of compressed air. Present practice in Sweden for floating piles in soft clay requires the diameter of the tube to be at least 40 mm smaller than the diameter of the pile so that the bearing capacity will not be reduced.

Reconsolidation of the soil and dissipation of excess pore pressures after driving takes place rapidly, as mentioned above. The resulting settlements are often of the same magnitude as the heave caused by the pile driving.

6.2.3 Ultimate bearing capacity of single piles and pile groups in soft clay

6.2.3.1 Single piles

The ultimate bearing capacity of floating piles in soft clay has been investigated by Wendel (1900), Fellenius (1938, 1955), Skaven-Haug (1940), Tomlinson (1957, 1971), Peck (1958), Bergfelt (1957), Seed & Reese (1957), Kerisel (1967), Broms (1966), Flaate (1968, 1972), Holmberg (1970), Torstensson (1973b) and others.

Bearing capacity is generally calculated from the undrained shear strength as evaluated from field vane, unconfined compression or fall-cone tests. Results from static or dynamic penetration tests or pile driving formulae cannot be used. The ultimate bearing capacity is generally calculated as the sum of the skin resistance, Q_s, and the point resistance, Q_p, as expressed by the relationship:

$$Q_u = Q_p + Q_s \tag{6.1}$$

A factor of safety of 2 to 3 is generally considered adequate.

Only a small axial deformation (a few millimetres) is required in soft sensitive clays to mobilize the maximum skin resistance, in contrast to the point resistance. For piles in sensitive clays, it is therefore advisible to rely only on the skin resistance. The point resistance usually accounts for less than 5 to 10% of the pile capacity, as pointed out very early by Wendel (1900) and Forsell (1918). Any additional reduction because of progressive failure is not recommended.

The point resistance is generally calculated from the relationship:

$$Q_p = (N_c\, s_u + \gamma l) A_p \tag{6.2}$$

where N_c is a bearing capacity factor, s_u is the undrained shear strength of the clay down to a depth below the pile point equal to two pile diameters, γ is the total unit weight of the soil, l is the pile length and A_p is the area of the pile point. The value $N_c=9$, proposed by Skempton (1951), is generally used. Values less than 9 have been reported, for example, by Brand (1971). The bearing capacity factor is influenced by the sensitivity of the clay and the modulus of elasticity of the soil at the pile point (Ladanyi, 1967, 1973; Roy et al, 1974), and N_c can be as low as 4 for clays with a very high sensitivity (quick clays).

The skin resistance, which generally accounts for 90 to 95% of the total bearing capacity of a pile, is normally calculated for soft clay from the relationship:

$$Q_s = c_a\, A_s \tag{6.3}$$

where $c_a = \alpha\, s_u$ is the unit adhesion along the pile and A_s is the surface area. The effective surface area of a pile can be smaller than the nominal area, as is often the case for steel H-piles where failure takes place through the clay

between the flanges of the piles (see e.g. Blessey, 1970). For driven circular or square concrete piles, the effective area is somewhat larger than the nominal area because of the high shear strength of the reconsolidated clay at the pile surface.

When a pile is loaded, the skin resistance is mostly first mobilized at the ground surface. When the load on the pile is increased, the skin resistance further down the pile is mobilized (van der Veen & Boersma, 1957). The load distribution along a single pile in soft clay has been analyzed by several people, including D'Appolonia & Romualdi (1963), D'Appolonia & Hribar (1963), Salas & Belzunce(1965), Poulos & Davis (1968) and Poulos & Mattes (1969b). The elastic theory of Mindlin (1936) was used in several of these analyses, the main difficulty in its application being the evaluation of the elastic constants. The load distribution has also been calculated by assuming that the shear stress at the pile surface is proportional to the relative displacement, δ, of the pile with respect to the surrounding soil. This relationship can be expressed by $\tau = K_s\delta$, where K_s is a displacement modulus. The displacement modulus is a function of the modulus of elasticity of the pile material, the shear modulus of the clay and the dimensions of the pile (Torstensson, 1973b; see Fig. 6.6). Non-linear relationships were proposed by Seed & Reese (1964) and Coyle & Reese (1966). Tests by Nilsson (1965) indicated that K_s decreases with increasing pile diameter.

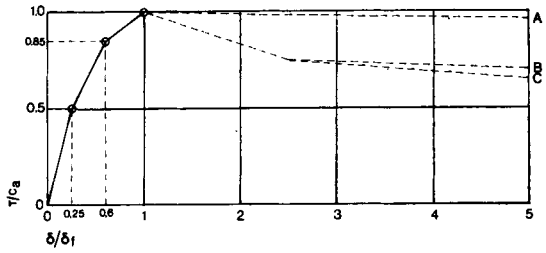

Fig. 6.6. Idealized relationship between shear stress ratio, τ/c_a, at the pile surface and relative displacement ratio, δ/δ_f, of a pile with respect to the surrounding soil (Torstensson, 1973b).

Torstensson (1973b) showed that K_s for a soft high plasticity clay can be expressed by the relationships (see Fig. 6.6):

$$\begin{aligned} K_s &= 2\ c_a/\delta_f \text{ at } \delta \leqslant 0.25\ \delta_f \\ K_s &= c_a\ (0.25/\delta + 1/\delta_f) \text{ at } 0.25\ \delta_f \leqslant \delta \leqslant 0.6\ \delta_f \\ K_s &= c_a\ (0.625/\delta + 0.375/\delta_f) \text{ at } 0.6\ \delta_f \leqslant \delta \leqslant \delta_f \end{aligned} \tag{6.4}$$

where c_a is the shear resistance at the pile-soil interface, δ is the relative displacement between the pile and soil and δ_f is the relative displacement between pile and soil at failure.

After failure ($\delta > \delta_f$), the relationship depends on the displacement rate. When the displacement rate is very low, τ/c_a is nearly constant. For $\tau < 0.5c_a$,

K_s can be determined from the theory of elasticity with an assumed shear modulus, G, of 150 s_u for the clay.

The bearing capacity of very long piles may be affected by progressive failure, so that the bearing capacity will be less than that calculated from Eq. 6.1. Thus, because of the compression of the pile, the skin resistance may be reduced close to the ground surface before the maximum resistance further down the pile has been mobilized. For clays with a high sensitivity, the remoulding and subsequent reconsolidation of the clay around a driven pile reduces the sensitivity and the risk of progressive failure. Available test data indicate that (1) the reduction of the bearing capacity because of progressive failure is generally small, (2) the undrained shear strength as determined by vane tests can be used in the analysis, and (3) no reduction in adhesion is required because of the length of the pile.

The adhesion, c_a, has been related to the undrained shear strength measured by field vane, unconfined compression and fall-cone tests through the coefficient α ($c_a = \alpha\, s_u$). Various values of α have been suggested.

The time between the driving and the loading of a pile also has an important effect on the bearing capacity. The maximum bearing capacity of concrete and steel piles is generally reached four to six months after driving when the plasticity index of the clay is relatively low ($I_p < 50$). For clays with a high plasticity index, the maximum bearing capacity is generally reached two to three months after driving (Fellenius, 1955). For timber piles, four to six weeks are usually required.

Tomlinson (1971) observed that the adhesion is high for piles that have been driven through a sand or gravel layer into an underlying layer of cohesive soil. Sand and gravel is brought down with the piles, so that a skin of compacted material is formed around the piles. For a similar reason, Tomlinson measured reductions in adhesion for piles which were driven through soft clay into an underlying layer of stiff clay.

Adhesion is also influenced by the loading rate. Test data reported by Eide et al (1961), Marsal & Mazari (1969) and Torstensson (1973b) indicate that the adhesion at loading rates between 0.001 mm/min and 1 mm/min increases 15 to 25% with a tenfold increase in the loading rate. The largest relative increase has been observed for clays with a low initial shear strength.

The shape of the pile also affects the ultimate bearing capacity. Test data suggest that the adhesion, c_a, is higher for tapered piles than for cylindrical piles because of the good contact between the pile and the surrounding soil, and also because of the high lateral pressure which develops around a tapered pile during driving. Flaate (1972) reported, however, that the adhesion factor, α, was as low as 0.5 for two timber piles driven into a soft clay with a shear strength of 58 kPa half a year after driving, and 0.6 about $5\frac{1}{2}$ years after driving.

Test data also suggest that the adhesion for timber piles which are driven with the root downwards will be low: it is common when splicing timber piles

that the top section is driven with the root downwards. An adhesion factor equal to 0.5 is normally used for such piles.

Data reported by Eide et al (1961) for piles in silt suggest that the short-term bearing capacity of a pile which has been subjected to sustained loading will be about 20% higher than the corresponding short-term bearing capacity of a pile which has not been loaded previously. Wendel (1900) reported a reduction in the bearing capacity of 20% at sustained loading for clays with a high plasticity index.

The gradual increase in adhesion and shear strength with time can be more or less compensated in long-term loading by a reduction in adhesion caused by creep when the liquid limit of the soil is high ($w_L > 50$). The long-term load tests carried out by Eide et al (1961) in normally consolidated clays suggest that the strength increase caused by consolidaton can be of the same order of magnitude as the reduction caused by creep.

Stress history also affects bearing capacity. The ultimate strength for soft clay will in general increase with increasing number of load cycles if sufficient time is available for reconsolidation of the soil between each load cycle. If there is no time for reconsolidation, then the shear strength can be expected to decrease gradually with the number of load cycles. Test data reported by Broms (1972b) indicate that the bearing capacity of cyclically loaded piles in soft clay corresponds to the undrained shear strength as evaluated by fall-cone tests.

The differences in adhesion that have sometimes been observed between timber, concrete and steel piles have been attributed by Tomlinson (1957) to vibrations during driving.

The ultimate skin resistance of a pile in clay has also been related to the effective lateral pressure in the ground after driving by:

$$Q_s = \sigma'_h \tan \varphi'_a A_s = K_o \sigma'_v \tan \varphi'_a A_s \tag{6.5}$$

where σ'_h and σ'_v are the average effective lateral and vertical pressures respectively, φ'_a is the friction angle of the pile material with respect to the effective pressure in the surrounding soil, and K_o is the coefficient of lateral earth pressure at rest (Clark & Meyerhof, 1972, 1973).

The factor $K_o \tan \varphi'_a$ varies from 0.25 to 0.40 when $s_u < 100$ kPa (Burland, 1973). For design purposes, a value equal to 0.3 for $K_o \tan \varphi'_a$ is recommended. For long piles, the resistance should be reduced, as pointed out by Meyerhof (1976).

In the opinion of the Authors, it is not advisable to use effective stress analysis for soft clays with high plasticity because the effective stresses around the piles are unknown. It is recommended, therefore, that the undrained shear strength should be used in the design instead. The undrained shear strength also reflects the ultimate bearing capacity with respect to suddenly applied loads from catastrophic events such as earthquakes, explosions, etc.

The effective lateral pressure around a driven pile is probably strongly affected by arching caused by reconsolidation of the remoulded soft clay

around the pile. The total lateral pressure gradually decreases with time, while the effective lateral pressure increases as the excess pore water pressure gradually dissipates. The increase in lateral effective pressure is probably less than the decrease in pore water pressure. Measurements by Koizumi & Ito (1967) indicate that the effective lateral pressure on the pile shaft is slightly higher than the 'at rest' pressure. To the Authors' knowledge, no data are available which indicate how the final effective lateral pressure is affected by remoulding or by the sensitivity of the clay.

Broms (1966) found from an analysis of available test data that a safe lower limit of the bearing capacity of timber piles in soft clay ($s_u < 50$ kPa) corresponds to the undrained shear strength as evaluated by field vane or fall-cone tests when the shear strength is reduced with respect to the liquid limit or the plasticity index of the soil. For precast concrete piles, a safe lower limit for the adhesion is 80% of the undrained shear strength. For steel piles, the adhesion can be as low as 50% of the undrained shear strength. In Norway, reduction coefficients of 1.0, 0.9 and 0.7 are used for timber, concrete and steel piles respectively when the shear strength is 30 kPa (Norwegian Pile Committee, 1973). In Canada (National Research Council, Canada, 1975), an adhesion equal to the undrained shear strength is used for timber and concrete piles when $s_u < 25$ kPa; for steel piles, an adhesion factor of 0.8 is used.

According to Torstensson (1973b), the influence of the loading rate can best be determined from field vane tests by using different rotation rates. The measured relationship between shear strength and time to failure can be applied directly to c_a, and c_a can be taken as the vane strength multiplied by a correction factor equal to 0.9. The same adhesion can be used irrespective of the pile material. Torstensson's assumptions have been verified by recent full-scale tests in Gothenburg, Sweden.

Brand (1971) reported that the adhesion for cast-in-place concrete piles corresponded closely to the undrained shear strength ($\alpha = 1.0$). The piles were installed in soft Bangkok clay with an average shear strength of about 15 kPa.

The adhesion can be expected to be affected by the construction procedure and by fissures present in the soil. It is also generally found that the reduction coefficient α will decrease with increasing shear strength.

6.2.3.2 Pile groups

Pile groups with point bearing piles in soft clay are generally designed on the assumptions that the piles act as individual struts which can only resist axial loads, and that each pile acts independently and does not affect the bearing capacity of adjacent piles. The bearing capacity and lateral resistance of the soil between the piles are generally neglected. The geometry of a pile group of point bearing piles, therefore, can become very complicated when the direction and magnitude of the applied forces vary, as is often the case for bridge abutments.

There is a growing interest in the use of point bearing piles which take account of the bearing capacity of the soil between the piles for structures

subjected to high lateral forces, such as off-shore structures. This development has also enhanced the interest in such questions as the effect of cyclic or sustained loads on the lateral resistance of piles. If the bearing capacity of the soil between the piles is to be utilized, it is necessary to know accurately, from borings and in-situ measurements, the strength and deformation properties of the soil and the changes caused by the installation of the piles. The remoulding that takes place during the driving of piles in soft clay and the subsequent consolidation of the soil around the driven piles affect the ultimate lateral resistance and the load-deformation relationships of the piles, as discussed in Section 6.2.7.1.

The ultimate bearing capacity of a floating pile group is governed either by the ultimate bearing capacity of the individual piles in the group when the spacing of the piles is large *(pile failure)*, or by the shear strength of the soil along the perimeter of the pile group *(block failure)* when the spacing is small. Terzaghi & Peck (1948) and Peck et al (1953) proposed the method shown in Fig. 6.7 to calculate the ultimate bearing capacity at block failure. The ultimate bearing capacity is assumed to be equal to that of a massive foundation with the same outside dimensions as the pile group. Model tests reported by Whitaker (1957, 1960), Sowers et al (1961), Saffery & Tate (1961), Tate (1963) and Barden & Monkton (1970) indicate, however, that the ultimate bearing capacity is overestimated by this method. At optimum pile spacing, the measured ultimate bearing capacity is approximately 70% of the calculated bearing capacity. In the Canadian National Building Code, a group efficiency factor of 0.7 has been suggested when the pile spacing is $2.5d$ to $4d$. The Authors agree with this suggestion. It should be pointed out that results from full-scale tests are lacking.

The axial deformation required to mobilize the skin resistance around the perimeter of a pile group is small compared with that required to mobilize the bearing capacity at the bottom of the pile group; in the latter case the deformation at failure corresponds normally to 5 to 10% or more of the width of the pile group. The latter deformations are so large that it is usually not possible to utilize for design the bearing capacity of the soil below the pile group. It is suggested, therefore, that only the resistance along the perimeter of a pile group in clay should be included in the calculations. A relatively low factor of safety can then be used.

Model tests also indicate that the pile cap can contribute to the ultimate bearing capacity (Whitaker, 1960) when the spacing of the piles is large ($>3d$). However, it is usually not possible to utilize this increase because of the large necessary deformations mentioned above.

The method which is sometimes used to calculate the ultimate bearing capacity on an eccentrically loaded piles group is shown in Fig. 6.8. Failure takes place when all the piles in the pile group, except for one row, are loaded to failure. The pile cap then rotates about point A located along the axis of this row. The ultimate bearing capacity of the pile group can then be calculated

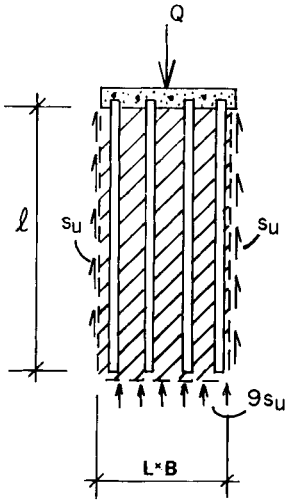

Fig. 6.7. Block failure of a pile group.

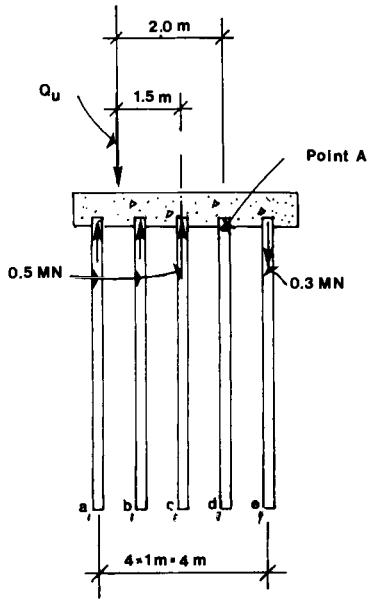

Fig. 6.8. Bearing capacity of an eccentrically loaded pile group.

from the equilibrium of the pile group with respect to the chosen centre of rotation. The piles to the left of point A are loaded in compression (0.5 MN), while the piles located to the right are loaded in tension (0.3 MN). The resulting moment for one row of piles is 3.3 MN, which corresponds to an ultimate bearing capacity of 1.65 MN (3.3/2.0) with respect to the chosen centre of rotation. The load in pile d_1, which can be calculated from an equilibrium equation, is 0.45 MN (1.65–1.20). If the calculated load in pile d_1 exceeds the capacity of the pile in compression, then the assumed centre of rotation is not the real one, and another centre has to be selected. In cases where the pile group is subjected to cyclic loading, the problem of shake down has to be considered which can lower the bearing capacity considerably. Comparisons with results from model tests indicate that this calculation method is conservative (Saffery & Tate, 1961).

6.2.4 Settlements of single piles and pile groups

6.2.4.1 General

The settlement of single piles and pile groups has been reviewed by Broms (1972d, 1976) and Meyerhof (1976). Floating piles in soft clay can be regarded as reinforcement in the soil. Their main function is to reduce the settlements and to increase the bearing capacity. In many cases it may only be necessary to reduce slightly the total or differential settlements. In order for them to act

as vertical reinforcement in soft clay, it is necessary that the piles behave in a ductile manner; they can then take large deformations without reduction in bearing capacity. The ductility of point bearing piles driven to bed rock with a high bearing capacity is limited (especially if the piles are prestressed), and their bearing capacity is governed by the strength of the pile section rather than by the shear strength of the surrounding soil. The bearing capacity of such piles will be exceeded before the bearing capacity of the soil between the piles is mobilized, and the bearing capacity of the soil between the piles cannot be utilized.

The ultimate bearing capacity of floating piles in soft clay is generally governed by the adhesion along the piles *(soil failure)* rather than by the strength of the pile section *(pile failure)*. Except for quick clays, the bearing capacity does not usually decrease with increasing deformation even when the axial deformation of the pile is large. For floating piles, therefore, it should be possible to utilize the bearing capacity of the soil between the piles to carry part of the applied load if large deformations do not matter. One essential requirement is that the bearing capacity of the pile section (pile failure) is always larger than the bearing capacity of the pile with respect to soil failure. The pile cap and the supported structure must also be able to transfer with sufficient safety a load which is larger than the ultimate bearing capacity of the pile group with respect to soil failure.

Relatively few piles might be required to reduce the settlements to an acceptable level when the weight of the excavated soil represents a large part of the total weight of the structure, as is often the case when the structure is provided with a deep basement (Zeevaert, 1957, 1964). The piles should be driven before the excavation, to reduce the heave and the subsequent reconsolidation of the soil (Hansbo et al, 1973). The driving order also affects the heave and lateral displacement of the soil, which will move in the same direction as the sequence of driving, as demonstrated by Massarsch (1976) and others.

The settlements are affected by the load distribution within the pile group. The larger part of the applied load is generally transferred to the surrounding soil through skin friction, and only a small part is transmitted by the pile points. The part carried by skin resistance decreases with decreasing pile spacing and with increasing size of the pile group. The reconsolidation of the soil after the disturbance during driving can cause an increase in the load on the piles because of negative skin friction.

Several methods are used to calculate the settlements of single piles and piles groups in soft clay. The settlements can be calculated by: (1) semi-empirical methods based on oedometer, pressuremeter or plate load tests, (2) elastic methods, and (3) finite element analysis.

Elastic methods are based on the assumption that the soil behaves as an ideally elastic material with a constant modulus of elasticity and a constant Poisson's ratio. This approach has been used by Poulos (1968), Mattes &

Poulos (1969), Poulos & Mattes (1969b, 1971a, 1971b), Poulos (1972, 1975), Butterfield & Banerjee (1971a, 1971b) and Banerjee (1975). The elastic methods do not take into account the stress history of the soil, the effect of time, the loading rate, or the remoulding of the soil caused by driving and the subsequent reconsolidation.

Analyses indicate that the compressibility of a pile has only a small effect on the stress distribution along the pile and on the settlement when the pile is relatively short ($l/d<20$) (see Butterfield & Banerjee, 1971a). The modulus of elasticity at unloading will generally be different from that at loading. It is also assumed in the elastic methods that the tensile strength of the soil is high and that the soil can resist the high theoretical tensile stresses that develop just above the pile point. In soft clay, this last restriction is less severe for floating piles than for point bearing piles, since the point resistance is low. The elastic methods therefore overestimate the group effect if the elastic constants, E and υ, are evaluated from pile load tests.

Non-linear and time-dependent stress-strain relationships can be used in a finite element analysis. Normally, it is only possible to consider two-dimensional or axisymmetric problems, because of the high costs of three-dimensional programs and the limited storage capacity of available computers. Ottaviani (1975) used a three-dimensional finite element program to analyze the stress distribution and settlements of pile groups consisting of 3×3 and 5×5 piles. Hooper (1973) used an axisymmetric program to analyse the stress distribution below a pile raft. A numerical method was developed by Banerjee (1976).

6.2.4.2 Initial settlements

The initial settlement of a single pile in clay at the working load can be calculated from the relationship:

$$\rho_i = \frac{c_{a,m}}{K_{s,m}} \tag{6.6}$$

where $c_{a,m}$ and $K_{s,m}$ are the mean values of c_a and K_s for the pile. The K_s value, which is a function of the relative pile length, l/d, can be found from Fig. 6.9 by assuming that $G=150\,s_u$. The nonlinear stress-strain characteristics of the pile-soil system can be taken into account by dividing the pile into elements, and by calculating in steps the displacement of the pile up to failure (see Eq. 6.4 and Hansbo et al, 1974).

The initial settlement of a pile group can also be calculated from the theory of elasticity (Davis & Poulos, 1968, 1972; Poulos, 1968; Mattes & Poulos, 1969; Butterfield & Banerjee, 1971a, 1971b; Banerjee, 1975). Non-linear solutions are also available (Poulos, 1973).

Poulos (1968) investigated how a loaded pile affects an adjacent pile. At, for example, $l/d=25$ and $s=5\,d$, where s is the pile spacing, the settlement of an adjacent pile will increase by 45 %.

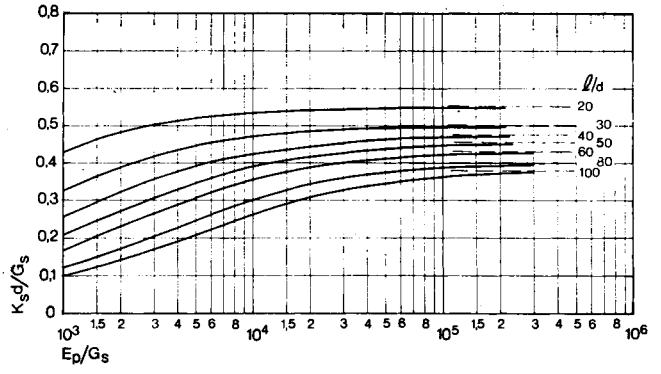

Fig. 6.9. Diagram for the determination of the initial settlement of a floating pile, $\rho_i = c_a/K_s$ (Hansbo et al, 1974).

Doroshkevich & Bartolomey (1965) analysed the settlements of six pile-supported buildings in the USSR using an elastic method, and they concluded that the agreement between calculated and measured settlements was satisfactory, but they did not describe how the elastic constants were evaluated. Cooke & Price (1973) and Cooke (1974) found, on the other hand, that the elastic methods overestimated considerably the settlements of pile groups in clay. The measured settlement ratio for a pile group consisting of two piles was 1.2, compared to a calculated value of 1.6. Test data seem to indicate that the initial settlement of a pile group cannot directly be calculated from the settlement of a single pile. It is recommended that the effect of adjacent piles should instead be calculated from an equivalent modulus of elasticity, as determined from in-situ tests or empirically from e.g. the undrained shear strength of the soil. The initial settlement of a pile group is then the sum of the settlements of single piles, as calculated from Eq. 6.6 or as determined from a pile load test, and the calculated increase in settlement from adjacent piles, using an assumed or measured value for the elastic modulus.

If the thickness of the compressible stratum below the pile group is relatively large, the initial settlement of a floating pile group in clay at a given total load mainly depends on the width of the pile group. At a given pile spacing, when the number of piles is larger than about 16, the initial settlement increases approximately with $\sqrt{n}$, where n is the number of piles in the pile group. If the settlements are excessive, it is often more efficient to increase the spacing and length of the piles than to increase the number of piles without increasing the dimensions of the pile group.

6.2.4.3 Time-dependent settlements

The time-dependent settlement of a floating pile group in a soft clay is usually calculated by assuming that the applied load is transferred to the surrounding soil at the lower third point (Terzaghi & Peck, 1948). The load

distribution in the underlying soil is generally calculated from the Boussinesq equation or by the '2:1 method' without taking into account the effect of the overlaying soil. The soil is divided into layers, and the compression of each layer is calculated separately. The compressibility of each layer is generally evaluated from oedometer tests. Good agreement between measured and calculated settlements by this method has been reported by, amongst others, Ireland (1964) and Torstensson (1971, 1973a).

In several cases, a better agreement was obtained when the applied load was assumed to be transferred to the surrounding soil at the bottom of the pile group (Bjerrum et al, 1957; Yu et al, 1965; Brzezinski, 1969; Girault, 1972). It is suggested that this load distribution should be used when the spacing of the piles is small ($<3d$ to $4d$) and the shear strength of the clay increases with depth.

The settlement of a pile group is affected by the pile cap, as shown by Davis & Poulos (1972) and Banerjee (1975). Butterfield & Banerjee (1971a) found that 20 to 60% of the applied load is usually carried by the soil between the piles. On the basis of a finite element analysis, Ottaviani (1975) indicated that about 10 to 20% of the applied load is carried by the pile cap. The distribution of load depends on the dimensions of the pile group and the spacing and length of the piles. The part carried by the pile cap increases with increasing size of the pile group and with increasing spacing of the piles, but it is generally not possible to utilize the bearing capacity of the pile cap in soft clay.

6.2.4.4 Settlement rate

It is usually very difficult to predict with confidence the settlement rate of pile groups in soft clay. The drainage conditions are in general difficult to establish, since thin sand and silt seams, fissures and cracks present in the soil affect the consolidation rate. Analyses by Torstensson (1971, 1973a) of four pile groups with 25 to 30 m long timber piles in a soft uniform clay indicate that no settlements take place in the soil between the piles above the lower third point of the pile group, and that the settlement rate can be estimated from the assumption that a permeable layer is located at the lower third point of the pile group; one-dimensional consolidation theory was used in the analyses. Similar observations with regard to settlements in the clay between the piles were reported by Hansbo et al (1973) for a piled-raft foundation with a 2.4 m pile spacing.

Measurements in normally consolidated clays in Sweden indicate that the permeability of the clay in the horizontal direction is two to five times larger than the permeability in the vertical direction, depending on the stratification of the soil. The remoulding of the soil caused by pile driving reduces this difference.

6.2.4.5 Differential settlements

Floating piles in clay also reduce differential settlements. The differential settlements of a pile group are generally largest near the perimeter of the

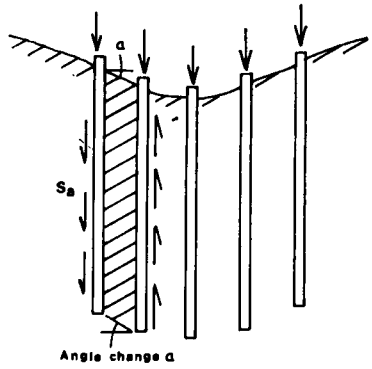

Fig. 6.10. Differential settlements of a pile group.

group, as illustrated in Fig. 6.10. The corresponding angle change, a, can be estimated from the relationship:

$$a = \frac{\tau_{av}}{G} = \frac{2(1+\upsilon)\tau_{av}}{E} \tag{6.7}$$

where τ_{av} is the average shear stress along the perimeter of the pile group, E and G are the modulus of elasticity and the shear modulus of the soil respectively, and υ is Poisson's ratio. The shear distortion will thus depend on the average shear stress along the perimeter of the pile group and on the shear modulus of the soil. Equation 6.7 can be rewritten as:

$$\tau_{av}/s_u = \frac{aE}{2s_u(1+\upsilon)} \tag{6.8}$$

For example, if $E = 300\ s_u$, $\upsilon = 0.5$ and $a = 1/300$, the value of τ_{av}/s_u becomes 0.33. If the average shear stress is limited to $0.33\ s_u$, the maximum angle change will be 1/300.

The shear modulus, G, decreases gradually with the duration of the applied load as the soil around the piles consolidates. Just after a pile group has been loaded, the largest part of the applied load is carried along the perimeter of the group and, moreover, only a small part will be transferred through the bottom. Approximate calculations based upon elastic theory indicate that about 80 to 90% of the applied load is carried along the perimeter of the pile group when the length of the piles is equal to the width of the group. The proportion carried by the soil below the pile group, however, gradually increases with time. The resulting decrease in τ_{av} partly compensates for the reduction in shear modulus with time. Further studies of the factors affecting the differential settlement are recommended.

6.2.4.6 Load distribution

At the working load, the load distribution in a pile group with floating piles is very uneven, as shown by Whitaker (1957, 1960) and Sowers et al (1961). At approximately one-half to one-third of the ultimate bearing capacity, the piles along the perimeter of the pile group carry a larger part of the applied load than the centre piles. The difference decreases with increasing load level and with increasing pile spacing. Tests performed by Torstensson (1969) indicate that the reduction in the difference begins at about 90% of the failure load of the pile group (Fig. 6.11). Hight & Green (1976) reported that the load on the piles around the perimeter of a pile-supported raft was about three times larger than for the piles at the centre.

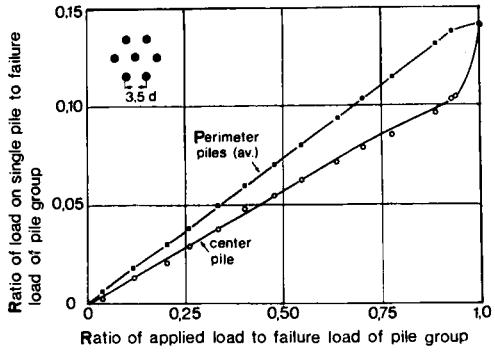

Fig. 6.11. Results of a pile load test performed by Torstensson (1969) (pile length=7.5 m, pile diameter=0.074 m).

The load distribution in rigid pile groups was investigated by Poulos (1968) and Butterfield & Banerjee (1971a) on the basis of elastic theory. Test data indicate, however, that the load distribution is not as uneven as anticipated from elastic theory. The difference in load will decrease with time as the soil around the pile group consolidates.

6.2.5 Buckling and bearing capacity of initially bent piles

Piles are generally not straight after driving, as illustrated in Fig. 6.12, which shows in plan the shape of approximately 70 m long concrete piles after driving. In this instance, the piles were quite badly bent since they were driven through old piling. Load tests indicated, however, that the piles were still able to carry the design load with sufficient safety (Swedish Pile Commission, 1964). The remoulding caused by previously driven piles affects the straightness in that a pile during driving deviates towards the previously driven piles. Batter piles generally end up not as straight as vertical piles.

The roughness of the surface of concrete piles also affects the straightness. The surface facing upwards during casting is usually not as smooth as the

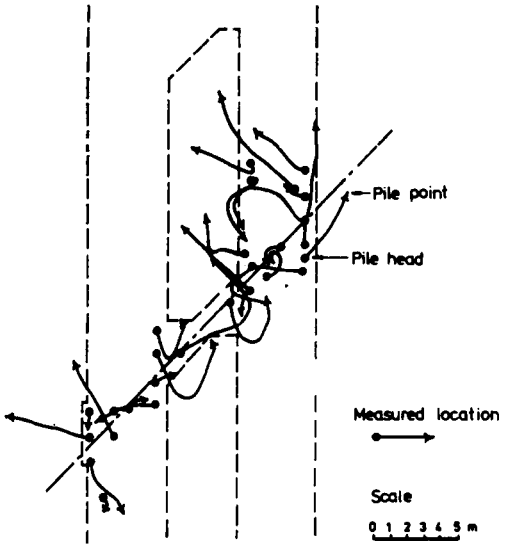

Fig. 6.12. Measured deviations of piles at an abutment along the railway line Göteborg C—Almedal (pile length ≃ 70 m) (Pejrud, 1965).

other sides, and the pile generally deviates in the direction of this rougher face during driving. In order to overcome this effect, the piles can be inclined slightly outwards (1:10 or 1:20). Hanna (1968) pointed out that the soil plug which forms between the flanges of steel H-piles during driving through a stiff surface crust can also cause a deviation of the pile; this deviation can be reduced by driving the piles in pre-bored holes. When reinforcing bars are used as piles, they should be rotated to keep them as straight as possible during driving.

The ultimate bearing capacity of a pile in soft clay can be reduced if the pile is not straight after installation. This has been discussed, for example, by Hanna (1968) and Hellström (1968). Hellström investigated the long-term bearing capacity of an initially bent precast concrete pile in soft clay. The 60 m long pile was loaded to 1.25 MN for 12 months. The minimum radius of curvature of the pile was 177 m along a 10 m long section, but the lateral deflection of the pile did not increase during the observation period.

The theoretical buckling load, P_k^{cr}, of an initially straight pile is governed by the following equation (Forsell, 1918, 1926; Granholm, 1929):

$$P_k^{rc} = 2\sqrt{d\, k_h\, E_p I_p} \tag{6.9}$$

where k_h is the coefficient of horizontal subgrade reaction, d is the width or diameter of the pile and $E_p I_p$ is its flexural rigidity. The theoretical minimum critical load for a 0.3×0.3 m precast concrete pile driven through a soft clay with a shear strength of 10 kPa is 11.4 MN at short-term loading and 2.4 MN

at long-term loading. The theoretical critical load for short-term loading, however, exceeds the proportional limit of the pile material. The actual critical load is thus less than the theoretically calculated load. It was assumed in this analysis that the modulus of elasticity of the pile material (concrete) is 30,000 MPa at short-term and 15,000 MPa at long-term loading, and that the coefficients of subgrade reaction are 80 s_u and 20 s_u respectively. The corresponding critical half wavelength, l_k^{cr}, can be calculated from the expression:

$$l_k^{cr} = \pi \sqrt[4]{\frac{E_p I_p}{d k_h}} \tag{6.10}$$

Splices will decrease the critical load, especially for an axially loaded pile, when the rigidity of the splices is low. When the splice is equivalent to a pinned connection, the critical load is only half of that for an unspliced pile (see e.g. Francis et al, 1964).

The bearing capacity of an initially bent pile is lower than that of a straight pile (Glick, 1948; Golder & Skipp, 1957; Parsons & Wilson, 1956; Johnson, 1962; Broms, 1963, 1967; Bernander & Svensk, 1969). In Norway, the allowable bearing capacity of steel H-piles is reduced when the radius of curvature is less than 400 m; the same limit is used in Sweden for precast concrete piles, and the allowable load is often reduced linearly to zero when the radius of curvature varies from 400 to 100 m.

Broms (1963) calculated the bearing capacity of an initially bent pile by expressing the initial lateral deflection of the pile in a series of sine terms. A similar approach was used by Francis et al (1964). The equations obtained are entirely general and can be used to analyze a pile of an arbitrary shape. An axial load increases the amplitude of the sine function, as expressed by the amplification factors:

$$a_1 = 1/(1-P/P_1^{cr}),\ a_2 = 1/(1-P/P_2^{cr}) \ldots a_m = 1/(1-P/P_m^{cr})$$

where:

$$P_1^{cr} = \frac{\pi^2 E_p I_p}{l^2} + \frac{l^2 d k_h}{\pi^2}$$

$$P_2^{cr} = \frac{(2\pi)^2 E_p I_p}{l^2} + \frac{l^2 d k_h}{(2\pi)^2} \tag{6.11}$$

$$P_m^{cr} = \frac{(m\pi)^2 E_p I_p}{l^2} + \frac{l^2 d k_h}{(m\pi)^2}$$

The lateral earth pressure on an initially deformed pile increases with increasing lateral deflection of the pile, and reaches a maximum at the section where the increase in the lateral deflection is largest. The lateral deflection of the loaded pile is given by the appropriate sine term. It is proposed, therefore,

that calculations should be based on the lateral deflection, δ_k, that corresponds to the lowest critical load, P_k^{cr}, and the corresponding critical length l_k^{cr}, since the critical loads will not vary much for terms with half wavelengths of l_{k-1}, l_k and l_{k+1}.

The maximum lateral deflection, δ_{ko}, of the unloaded pile can be determined from inclinometer measurements by measuring the maximum deviation from a straight line connecting two points on the axis of the pile that corresponds to the critical length, l_k, as indicated in Fig. 6.13. The lateral deflection of the loaded pile can then be calculated from the relationship:

$$\delta_k = \delta_{ko}\, a_k \tag{6.12}$$

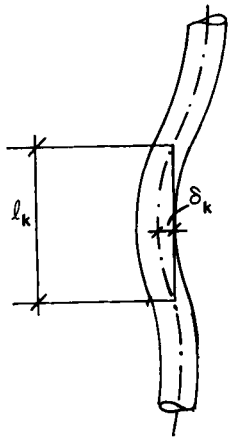

Fig. 6.13. Initially bent pile.

The maximum stress in the pile section can be evaluated by expressing the initial bending moment in the deformed pile in a series of sine terms. The initial bending moment in the pile will be proportional to the radius of curvature of the deformed pile.

The maximum bending moment of the loaded pile is equal to:

$$M_{max} = M_{o,\,max}\, a_k$$

where $M_{o,max}$ is the maximum bending moment of the unloaded pile, and a_k is the corresponding amplification factor, as described above. This initial maximum bending moment can, consequently, be estimated from the expression:

$$M_{o,\,max} = \frac{8\,\delta_k\, E_p\, I_p}{(l_k)^2} = \frac{E_p\, I_p}{R} \tag{6.13}$$

where δ_k is the maximum deviation from a straight line with a length equal to l_k (Fig. 6.13), and R is the radius of curvature.

The maximum stress in the initially straight pile can be calculated from:

$$\sigma_{max} = \frac{Q}{A_p} + \frac{M_{o,max}\,\delta\,a_k}{2\,I_p} = \frac{Q}{A_p} + \frac{E_p\,\delta\,a_k}{2R} \tag{6.14}$$

The straightness of driven piles can be checked with inclinometers or rods which are lowered down into a pipe cast into or welded to the pile.

6.2.6 Negative skin friction

When the soil surrounding piles settles as a result of such factors as driving disturbance, lowering of the ground water-table, placement of fill around the pile group, or vibrations, the load in a pile group of point bearing piles can be increased appreciably (see e.g. Zeevaert, 1959). The associated downdrag force is called *negative skin friction.* The piles will then carry not only the applied load but also part of the weight of the surrounding soil. Failures of pile groups caused by negative skin friction have been reported by Milner (1957), Brand & Luangdilok (1975) and others.

Negative skin friction in piles has been investigated by several authors, including Gant et al (1957), Zeevaert (1959), Buisson et al (1960), van Weele (1964), Johannessen & Bjerrum (1965), Johannessen (1965), Bjerrum et al (1969), Bozozuk & Labrecque (1969), Fellenius & Broms (1969), Bozozuk (1970, 1972), Fellenius (1969, 1971a, 1972a), Brons et al (1969), Endo et al (1969), Sawaguchi (1971) and Walker & Darvall (1973).

6.2.6.1 Calculation of negative skin friction

Salas & Belzunce (1965) investigated negative skin friction by treating the soil as an ideally elastic halfspace; the piles were assumed to be rigid. A similar approach was used by Verruijt (1969). Walker & Darvall (1970) used the finite element method (FEM). Elastic theory was used by, amongst others, Poulos & Mattes (1969a) and Poulos & Davis (1972, 1975) to analyse negative skin friction with consideration of the flexibility of the pile and local slip along the pile.

Negative skin friction has been investigated in the field by Johannessen & Bjerrum (1965), Johannessen (1965), Bozozuk & Labrecque (1969), Fellenius & Broms (1969), Fellenius (1972a), Bozozuk (1970, 1972) and others. The investigation by Johannessen & Bjerrum (1965) indicated that, when the relative movement between the piles and the surrounding soil is larger than 5 to 10 mm, the negative skin friction, c_a, can be estimated from the equation:

$$c_a = K \tan \varphi'_a \, p'_v \tag{6.15}$$

where p'_v is the effective overburden pressure, including the increase caused by the weight of a fill placed around the piles. The coefficient $K \tan \varphi'_a$ depends on the lateral pressure on the pile and on the settlement rate. A value

of 0.25 for the factor $K \tan \varphi'_a$ was proposed in the Canadian Building Code (National Research Council, Canada, 1975).

It can be expected that the effect of the loading rate on the factor $K \tan \varphi'_a$ will be larger for a clay with a high plasticity index and a low shear strength than for a silty soil or a normally consolidated clay of low to medium plasticity index. The values recommended for design when the settlement rate is about 10 mm/year are shown in Table 6.1. It is suggested that the value of $K \tan \varphi'_a$ should be increased by 20% with each tenfold increase in the settlement rate for clays with a high plasticity index ($I_p > 50$). For clays of high plasticity, the value of $K \tan \varphi'_a$ corresponds approximately to one half of the undrained shear strength as evaluated by vane tests. This takes account of the creep that takes place in clays when the plasticity index is high.

Table 6.1. Evaluation of the coefficient $K \tan \varphi'_a$ (Broms, 1976)

Material type	$K \tan \varphi'_a$
Rock-fill	0.40
Sand and gravel	0.35
Silt or normally consolidated clay with low to medium plasticity ($I_p < 50$)	0.30
Normally consolidated clay with high plasticity ($I_p < 50$)	0.20

An analysis by Meyerhof (1976) of available test data indicates that the negative skin friction of clay decreases with increasing pile length because of the compression of the pile shaft. Negative skin friction affects the individual piles only down to the neutral point, where the relative movement of the soil with respect to the piles is zero. Below the neutral point, the skin friction is positive. The neutral point is located close to the pile tip for piles which have been driven through a deep layer of soft clay to bedrock of high bearing capacity. The total length of the piles is then subjected to negative skin friction. If, on the other hand, the piles have been driven into a sand or gravel layer and the applied load is transferred primarily through the pile tip, then the axial displacement of the pile below the neutral point is relatively large. A relatively large deformation (10 to 20% of the pile diameter) is required to mobilize the maximum point resistance. If the penetration of the pile into the sand or gravel layer is relatively large ($>10d$), then the load in the pile is carried mainly by skin friction in the sand or gravel layer. The axial displacement of the lower part of the pile with respect to the surrounding soil which is required to mobilize the skin friction is then relatively small (2 to 5 mm). The Norwegian Pile Commission (1973) suggested that the neutral point can be assumed to be located at the depth where the settlement of the soil is 5 mm.

For piles that have been driven to bedrock of high bearing capacity (e.g. granite or gneiss), it can be assumed that the location of the neutral point

corresponds to the total length of the pile or the total depth of the compressible layers. However, for rock with an ultimate bearing capacity less than that of the pile section, it is suggested that the neutral point be taken at the level where the settlement of the surrounding soil is 5 mm, as proposed by the Norwegian Pile Commission (1973).

It is suggested that the negative skin friction on pile groups can also be calculated by the method indicated in Fig. 6.14. Figure 6.14a shows the load increase when the spacing of the piles is large. In this case, each pile will carry a load resulting from the maximum skin friction, c_a, down to the neutral point. This skin friction can be calculated from Eq. 6.15. When the spacing of the piles in the group is small, the load increase is illustrated in Fig. 6.14b. The pile group will carry a total load which corresponds to the shear resistance of the soil on the perimeter area of the pile group plus the weight of the fill above the pile group, i.e. $2D(L+B)\,s_u+B\,L\,q$. When the weight of the fill is relatively small and the lowering of the ground water level is moderate, the increase in load in the piles corresponds to the load on the shaded areas shown in Fig. 6.14c. Because of negative skin friction, the load increase will be large for the corner piles in comparison with the piles at the centre of the pile group (Broms, 1971a).

When negative skin friction is considered in design, it is only necessary to take into account the negative skin friction in combination with the dead load. Any live load will be carried by positive skin friction, since a very small axial

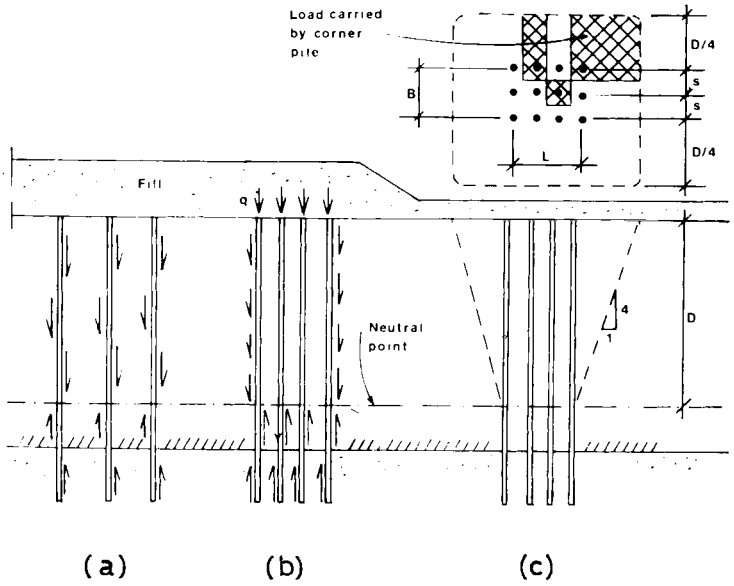

Fig. 6.14. Methods of calculating negative skin friction for: (a) large pile spacing, (b) small pile spacing and large load increase, (c) small pile spacing and small load increase.

displacement is required to mobilize the maximum skin friction resistance (see, for example, Fellenius, 1972a; Torstensson, 1973a). Live loads will only affect the bearing capacity if they are larger than twice the negative skin resistance. It is also suggested that a factor of safety of 2.5 to 3.0 should be applied only to the net ultimate load, $(Q_p - Q_{neg})$, where Q_p is the point resistance and Q_{neg} is the negative skin resistance.

When the length of the piles exceeds 20 to 25 m, the negative skin friction is often of the same magnitude as the applied load. It is then important to estimate accurately the maximum negative skin friction. In load tests, the skin resistance can be separated from the point resistance by such means as load cells or a system of tell-tales. The skin resistance is then equal to the difference between the total resistance and the point resistance.

6.2.6.2 Reduction of negative skin friction

When the pile length exceeds 20 to 25 m, it is often preferable to change from point bearing to floating piles in order to avoid the load increase caused by negative skin friction. The settlement of the pile group will be increased by this change, but if this increase is not acceptable, it is often possible to compensate for this by changing to a structural system which is less affected by differential settlements.

The effect of negative skin friction can be reduced with separate loose sleeves around the piles (van Weele, 1964), by coating the piles with bitumen (Hutchinson & Jensen, 1968; Bjerrum et al, 1969; Walker & Darvall, 1973) or by surrounding the piles with bentonite (Bjerrum et al, 1969). Another possibility is to adjust the elevation of the structure, as proposed by Correa (1969). In Sweden, piles are sometimes dipped in bitumen. The required thickness of the bitumen layer depends primarily on the viscosity of the bitumen, the ground temperature and the roughness of the pile surface. Test data reported by Bjerrum et al (1969) indicate that even a relatively thin layer of bitumen (1 mm) is effective on steel piles. For concrete piles, the thickness of the bitumen layer should be at least 2 mm. The effectiveness of a bitumen coating can decrease with time because of bacteriological action (Simons, 1971), and it should also be noted that a bitumen coating reduces the bearing capacity. Hutchinson & Jensen (1968) reported that the skin resistance for coated piles is only 30 to 80% of that for uncoated piles.

It is important that a bitumen coating is not damaged during driving. Pre-coring might be required through rockfill, sand and gravel, through a stiff surface crust or when the ground surface is frozen. When the piles are protected by a bentonite layer, the piles can be provided with an enlarged tip or point. The cost of protecting a pile is frequently of the same magnitude as the cost of the pile itself and, therefore, piles are only rarely protected against negative skin friction.

6.2.7 Laterally loaded piles and pile groups in soft clay

6.2.7.1 Lateral resistance of piles

Failures of single piles and pile groups have sometimes been caused by the lateral displacement of the surrounding soil; two cases are illustrated in Fig. 6.15. In the case shown in Fig. 6.15a, the mass of soft clay was not stable because of the difference in elevation between the basement floor and the outside ground. The failure shown in Fig. 6.15b, reported by Fellenius & Johansson (1972), was caused by a fill placed outside the building in order to compensate for the settlement resulting from a lowering of the ground water level. The height of the fill was increased gradually as the settlement increased. The lowering of the ground water level caused settlement not only outside the structure but also below the basement floor, and a free space gradually developed below the floor. Finally, the height of the fill was so large that the total overburden pressure at the level of the basement floor corresponded to the bearing capacity of the soft clay. This occurred when the total overburden pressure at the level of the basement floor was approximately 5.5 s_u.

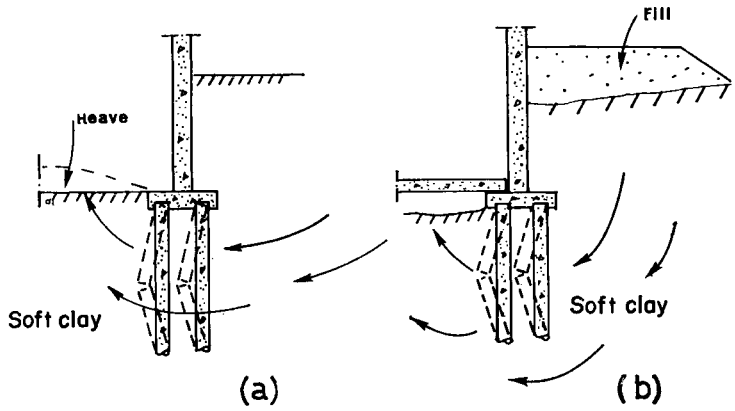

Fig. 6.15. Failure of piles caused by lateral displacement.

Similar effects on piles to those described above have been observed at the edges of high fills (Heijnen & Lubking, 1973) and below bridge abutments where the weight of the approach fill has been so high that the bearing capacity of the underlaying soft clay has been exceeded. The lateral resistance of a pile is low, especially when the thickness of the clay layer is large, and field observations indicate that the lateral displacement of a pile or pile group will be about the same as that of the surrounding soil (Bergdahl & Nilsson, 1974).

The conditions close to a bridge abutment constructed on soft clay and supported on point bearing piles are shown in Fig. 6.16a. The bearing capacity of the soft clay will be exceeded when the net overburden pressure at the bottom of the abutment corresponds to about 5.5 s_u. Damage caused by the

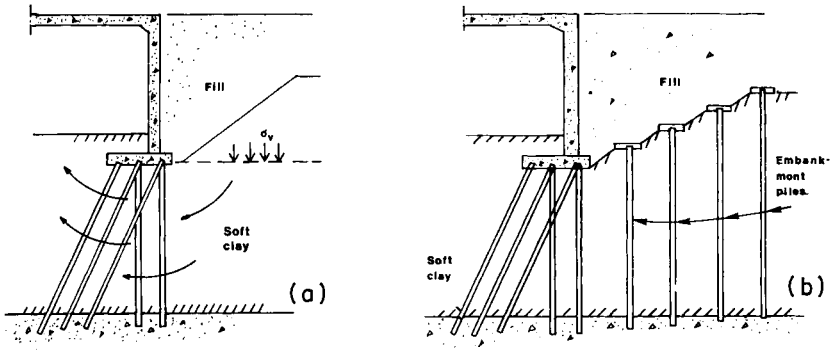

Fig. 6.16. Bridge abutment on point bearing piles: (a) mode of failure, (b) protection by embankment piles.

lateral displacement of soft clay has been reported, for example, by Stermac et al (1968), Leussink & Wenz (1969), Tschebotarioff (1970) and Marche & Lacroix (1972). The soil probably moves laterally due to creep when the net overburden pressure at the bottom of the abutment is as low as 60 to 70% of the ultimate bearing capacity (about 3 to 4 s_u) and the liquid limit and organic content of the soil are high ($w_L > 80$).

Embankment piles or lightweight fills are often used to decrease the lateral pressure on structural piles, as illustrated in Fig. 6.16b. Lightweight fills are used when the height of the embankment is relatively small and it is possible to reduce the net overburden pressure to below the creep strength of the clay. Other methods to reduce the lateral pressure have been described, for example, by Tschebotarioff (1970). The lateral pressure can also be reduced with a spill-through abutment.

Begemann & de Leeuw (1972) investigated the lateral earth pressure acting on a row of piles below the edge of a high fill using elastic theory. They found that, when the piles are infinitely stiff, the lateral earth pressure is twice as high as when the soil is free to move laterally. The lateral displacements of precast concrete piles in soft clay caused by surface loads were measured by Uriel et al (1976), and these were found to be approximately twice those calculated from elastic theory using Boussinesq's equation.

Settlements caused, for example, by a lowering of the ground water level increase the lateral deflection of batter piles, and the deflection can be so large that the piles fail (see e.g. Gmeiner, 1976). The restraint from the pile cap or from a fill placed around the piles has been found to have a large influence on the maximum bending moment (Broms & Fredriksson, 1976). Batter piles should therefore not be used when large settlements are expected. Instead, it is preferable to utilize the lateral resistance of vertical piles or to resist the lateral forces with separate skirts. Fabric reinforcement has also been used to resist lateral earth pressures in embankments (Holtz & Massarsch, 1976).

One of the first to investigate the lateral resistance of piles in soft clay was Wendel in 1900 who measured the lateral resistance of timber piles of different lengths. The lateral displacement and the rotation of the piles were measured at the ground surface. He found that the lateral resistance did not increase when the pile length exceeded 5 m, since the piles broke 1.5 to 2.0 m below the ground surface.

The lateral resistance of piles in soft clay is of special interest for offshore structures in such locations as the Mexican Gulf close to the Mississippi delta, where extensive deposits of very soft clay are common.

6.2.7.2 Lateral deflection of piles

The lateral deflection of a laterally loaded pile increases approximately linearly with the applied load up to about one-half of the ultimate lateral resistance of the pile. The soil separates from the pile as the pile rotates in the soil, and near failure, a wedge of soil forms in front of the pile close to the ground surface. Further down, the soil moves essentially laterally around the pile.

At working loads of about one-third to one-half of the ultimate lateral resistance, the lateral deflection is usually calculated from the assumption that the soil behaves either as an ideally elastic material (Mindlin's equation), or as a series of independently acting elastic springs (Winkler foundation).

Recently, three-dimensional finite element programs have been developed (Wittke, et al, 1974), for which it is usually assumed that the soil behaves as an ideally elastic-plastic material. The main limitations of the finite element method are (1) the difficulty in evaluating the elastic constants to be used in the analysis, and (2) the large storage capacity which is required for three-dimensional programs, which makes the method time-consuming and thus expensive to use.

Elastic theory (Mindlin, 1936) has been used by many, including Spillers & Stoll (1964) and Douglas & Davis (1964). Poulos (1971a, 1971b, 1973) and Oteo (1972) used a similar approach to analyze the behaviour of laterally loaded single piles and pile groups in which the piles were replaced by a series of thin rectangular strips with the same width as the piles. Oteo (1972) used a finite difference method to calculate the lateral pressure distribution. Grasshof (1970) developed a computer program based on Mindlin's solution. Non-linear relationships have also been proposed (Kubo, 1965; Poulos, 1973).

Poulos (1971a) proposed evaluating the modulus of elasticity of the soil from lateral load tests. For clays, the undrained modulus, E_u, was found to vary between 15 s_u and 95 s_u, and a value of 40 s_u was recommended by Poulos for design purposes.

On the basis of the Winkler model, the relationship between lateral deflection and earth pressure distribution along a laterally loaded pile can be expressed as:

$$Q_h = k_h y \tag{6.15}$$

where k_h is the coefficient of horizontal subgrade reaction, which varies with the dimensions of the pile and with the depth below the ground surface. The value of the coefficient of horizontal subgrade reaction close to the ground surface governs the lateral deflection and moment distribution in the pile. It is important, therefore, that a reasonably accurate value is used for k_h close to the ground surface down to a depth that corresponds to four to eight pile diameters.

Poulos (1971b, 1973) also investigated the behaviour of laterally loaded pile groups using elastic theory. The lateral deflection of the group was expressed in terms of a displacement ratio, R_ρ, and a group reduction factor, R_R. Both Poulos (1971b) and Oteo (1972) found that the lateral deflection and efficiency of a pile group decreased rapidly with decreasing pile spacing. Comparison with test data indicates that the elastic theory underestimates the group effect when the spacing of the piles is small (Poulos, 1971b).

Few data have been published on the lateral resistance of pile groups in cohesive soils. Prakash & Saran (1967) concluded from model tests that there are no group effects when the spacing of the piles in the direction of the applied load exceeds six pile diameters.

The lateral deflection of a laterally loaded pile increases with time because of consolidation and creep of the surrounding soil. Test data suggest that the increase in the lateral deflection with time corresponds roughly to a decrease in the coefficient of subgrade reaction to a value between one-third and one-sixth of the initial value. It is generally assumed that the coefficient of subgrade reaction, k_h, decreases linearly with increasing pile diameter, and that $k_h d = K_h$ is constant and independent of the pile diameter (Terzaghi, 1955). Test data indicate, however, that this assumption is conservative when the pile diameter exceeds about 1 m (Weinhold, 1972).

It is often difficult to estimate the coefficient of horizontal subgrade reaction for cohesive soils, because of the remoulding of the clay caused by driven piles and the softening that occurs around cast-in-place piles. The coefficient of subgrade reaction can be estimated, for example, from plate load tests or pressuremeter tests, or empirically from the shear strength of the soil. Broms (1965) suggested that the coefficient of horizontal subgrade reaction can be calculated from the following relationship, which is based on an analysis by Vesic (1961) for a beam supported by a series of elastic springs:

$$k_h = \frac{0.4\, D\, k_{pl}}{d} \tag{6.17}$$

where d is the diameter of the laterally loaded pile and k_{pl} is the coefficient of subgrade reaction as evaluated from a load test with a circular plate of diameter D. Since $D\, k_{pl} = 1.67$ for an elastic material with a Poisson's ratio, υ, equal to 0.5, then $k_h = 0.67\, E/d$. Poulos calculated an equivalent coefficient of subgrade reaction based on elastic theory for a stiff fixed-head pile with an

l/d ratio of 25; at $\upsilon=0.5$, $k_h=0.82\ E/d$, which is 18% more than for the relationship given above.

The coefficient of horizontal subgrade reaction can also be evaluated from lateral load tests on short stiff piles by measuring the lateral deflection and rotation of the pile at the ground surface (Reese & Cox, 1969; Muzas, 1972). Pressuremeter tests can also be used (Menard, 1962; Smoltczyk, 1963). Matlock (1970) suggested that the relationship between soil reaction, p, and lateral deflection, y, can be approximated by the relationship:

$$\frac{p}{q_u} = 0.5 \left(\frac{y}{y_{50}}\right)^{1/3} \tag{6.18}$$

where y_{50} is the lateral deflection at half the ultimate resistance of the soil, q_u. Matlock also investigated the effect of repetitive loading on the load-deformation relationship. Solutions are also available for the case when the coefficient of subgrade reaction increases linearly with depth (Reese & Matlock, 1956).

It is suggested that a coefficient of subgrade reaction given by $k_h\ d=80\ s_u$ can be used for short-term static loading, and $k_h\ d=20\ s_u$ for long-term static loading (see Section 6.2.5). These values correspond to $E=E_u=120\ s_u$ for short-term (undrained) loading and $E=E'=30\ s_u$ for long-term (drained) loading. Load tests on short piles are required if a more accurate value for $k_h\ d$ is desired.

6.2.7.3 Ultimate resistance of piles

Failure of a laterally loaded single pile or of a pile group occurs either when the piles rotate as a unit in the soil *(soil failure)* or when the moment resistance of the piles is exceeded and one or several plastic hinges develop along the pile *(pile failure)*.

The ultimate strength of a laterally loaded pile in soft clay depends primarily on the ultimate lateral resistance of the soil close to the ground surface. On the basis of a simplified theoretical analysis, Reese (1958) suggested that the ultimate lateral resistance increases from about 2 s_u at the ground surface to about 12 s_u at a depth of about three pile diameters. When the soil flows around the laterally loaded pile, the maximum soil reaction can vary between 8.3 s_u and 11.4 s_u (Brinch Hansen, 1948). Test data indicate generally that the ultimate lateral resistance can vary between 7.5 s_u (Jalil, 1967) and 11.0 s_u (McClelland & Focht, 1958; Reese et al, 1975).

The probable earth pressure distribution along a laterally loaded short pile in soft clay (Fig. 6.17a) is shown in Fig. 6.17b together with that proposed by Broms (1964) for the analysis of the ultimate lateral resistance (Fig. 6.17c). In order to simplify the calculations, it is suggested that the lateral earth pressure should be taken as zero down to a depth that corresponds to 1.5 d, and below this point should be assumed to be 9 s_u. Test data indicate that this assumed lateral earth pressure is conservative. The shear force along the base

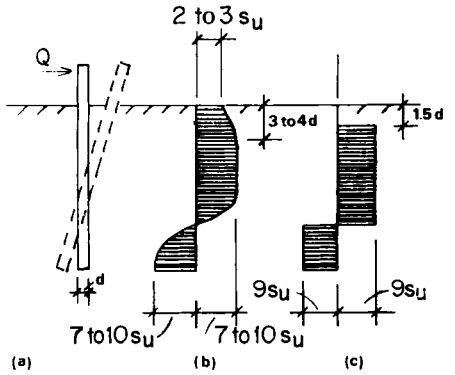

Fig. 6.17. Lateral resistance of a short pile in soft clay: (a) pile displacement, (b) probable earth pressure distribution, (c) ultimate distribution for analysis (Broms, 1964).

of the pile becomes important for very short piles in overconsolidated clays, as pointed out by Kerisel & Adam (1967) and Broms (1965), but for soft clays this effect is small.

Reese et al (1975) analysed the ultimate lateral resistance of the soil close to the ground surface by considering the forces acting on a wedge of soil in front of the loaded pile. They proposed that the ultimate lateral resistance of the soil at depth z can be calculated from:

$$q_u = 2\, s_u + \gamma' z + 2.83\, s_u \frac{z}{d} \tag{6.19}$$

where γ' is the submerged unit weight of the soil; the adhesion along the sides of the pile was neglected in the analysis. For a square pile in soft clay, when the adhesion is equal to the undrained shear strength of the soil, the first term in Eq. 6.19 is equal to 4 s_u instead of 2 s_u. Matlock (1970) suggested that for a circular pile this term should be 3 s_u.

Reese et al (1975) introduced the empirical correction factors $\bar{A}$ and $\bar{B}$ to account for the differences between measured and calculated ultimate lateral resistances for static and cyclic loading respectively; they proposed that $\bar{A}=0.6$ and $\bar{B}=0.3$. For large depths ($z>3d$), Reese et al suggested, therefore, that $q_u=6.6\ s_u$ ($0.6\times 11\ s_u$) for static loading, and $q_u=3.3\ s_u$ ($0.3\times 11\ s_u$) for cyclic loading. For static loads, the relatively simple design method proposed by Broms (1964) can be used. For cyclic loading, the deterioration in the shear strength has to be considered, and it is suggested that a reduced undrained shear strength be used equal to 70% of the undisturbed shear strength of the soil.

An axial load generally has only a small effect on the lateral deflections, as long as the axial load is less than about 10% of the buckling load. For partially embedded piles, the axial load can have a large effect on the bearing capacity

(Davidsson & Robinson, 1965). For very soft clays, the axial load can by important, since the critical load is low. In cases when the axial load is 40% of the critical load, the increase in the maximum lateral deflection due to the axial load will be 67% if the soil has the same properties as an ideally elastic medium.

Donovan (1959) reported that the lateral resistance of a pile group corresponds to that of the individual piles when the spacing of the piles exceeds four pile diameters. When the spacing is less than two pile diameters, the soil enclosed by the piles in the group follows the piles, and the pile group behaves as a unit. The lateral resistance per pile decreases with increasing number of piles in the direction of the applied load.

For soft clay, the lateral resistance can be increased by placing a layer of sand or gravel around the piles, as illustrated in Fig. 6.18a. This method is especially effective for pile groups subjected to cyclic loading, since the sand or gravel fill works itself down around the piles and increases their effective diameter (Broms, 1972a; Hellström, 1968). The lateral resistance is also increased if the piles are provided with wings, as shown in Fig. 6.18b. For a concrete or a timber pile, the wings can be attached to a separate collar which is driven or pushed down into the soil before driving the pile. The upper part of a pile can also be enlarged to increase the lateral resistance (Fig. 6.18c). In cases where piles extend unsupported above the ground surface, the enlarged section also increases the buckling strength of the unsupported parts of the piles.

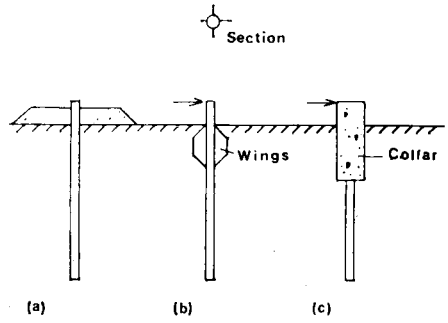

Fig. 6.18. Methods of increasing the lateral resistance of piles.

6.2.8 Embankment piles

Timber piles are extensively used in Sweden to increase the stability of slopes or embankments constructed on soft clay, as illustrated in Fig. 6.19. Slender wooden poles have also been used (Asplund, 1939), and these are very efficient because of their large surface area. The stability of a slope is increased by the skin resistance along the piles, as well as by the lateral resistance of the piles along the failure surface. When the piles are driven through a clay layer

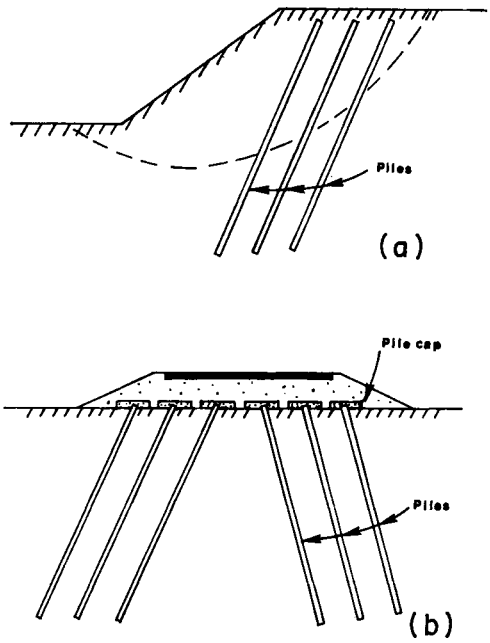

Fig. 6.19. Stabilization with piles: (a) slopes, (b) embankments.

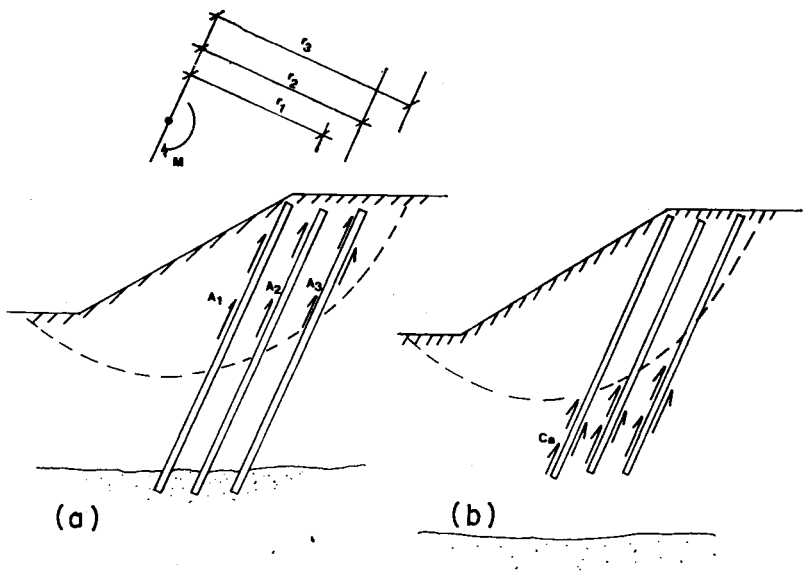

Fig. 6.20. Modes of failure of slopes stabilized with piles: (a) thin clay stratum, (b) thick clay stratum.

to bedrock or to a layer of sand, gravel or till, the skin resistance along the top part of the piles, above a circular failure surface, governs the restraining moment, as calculated by the relationship (Fig. 6.20a):

$$M_{re} = c_a (A_1 r_1 + A_2 r_2 + \ldots\ldots) \tag{6.20}$$

where c_a is the adhesion, A_1, A_2 . . . are the surface areas of the piles above the failure surface and r_1, r_2 . . . are the distances from each pile axis to the assumed centre of rotation. Where the thickness of the layer is large and the piles are relatively short and are not driven to a firm layer, the skin resistance along the lower parts of the piles will govern, as illustrated in Fig. 6.20b.

Piles also increase embankment stability through dowel action at the slip surface, and calculation methods have been published, for example, by Brinch Hansen (1948) and Broms (1972c). The lateral resistance is normally not utilized, because of the risk of progressive failure. The peak resistance of the soil might be exceeded along the pile surface before the lateral resistance of the piles has been mobilized, since the deformation required to mobilize the shear strength of a soft clay is small, especially if the sensitivity is high.

The driving of the timber piles reduces temporarily the average shear strength of the clay because of the disturbance during driving. The reduction is normally 20 to 30%, but a reduction of up to 50% has been observed, and this decrease in shear strength has in several cases initiated slides. Remoulding of the clay by pile driving and reduction in its shear strength can be reduced by pre-coring.

The sequence of pile driving is also important. The present practice in Sweden is to drive first the piles located closest to the slope and to proceed away from it. The piles which are driven first reduce the lateral displacement of the soil down the slope.

In Sweden and Finland, piles are used extensively to support road embankments on soft clays (Kjellman, 1940). The use of embankment piles in Thailand has been described by Eide (1968), and some relevant embankment failures in Sweden have been described by Andersson (1975). In Sweden, each pile is provided with a separate precast concrete cap (0.8×0.8 m to 1.5×1.5 m), the relationship shown in Fig. 6.21, which is mainly based on results from model tests (Swedish Road Board, 1974), being used to determine its size. In order to prevent failure of the embankment during the driving of piles, embankment piles are generally only used when the soil below the embankment has a low organic content, a sensitivity of less than 20, and a water content of less than 60%. The undrained shear strength of the clay should exceed 10 kPa to provide sufficient lateral resistance of the piles.

If the subsoil is soft, the design and construction of embankment piles have to be carried out with care. It is very important that the soil, the concrete caps and the piles act as a unit. The following requirements should be satisfied (Andersson, 1975):

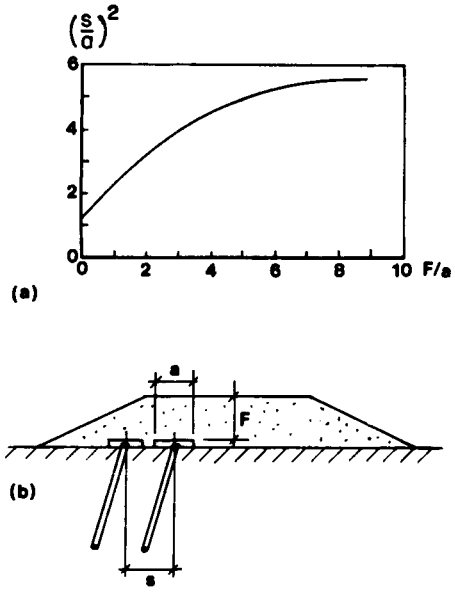

Fig. 6.21. Design of embankment piles (Swedish Road Board, 1974).

(1) The earth pressure from the embankment fill should be resisted by batter piles.

(2) The reaction forces of the batter piles should be taken by the structure (usually the embankment).

(3) The embankment fill between the concrete caps should remain in place (arching required).

(4) The fill should be placed so that piles and caps are not displaced: where there is a soft top layer, the pile heads must be fixed.

(5) The piles should be able to carry the weight of the embankment.

Precast concrete piles are normally used when the height of the embankment is relatively large (>2 to 3 m) since the maximum bearing capacity of the piles can then be utilized. Timber piles are used when the height is relatively small. The piles are generally designed to carry the total weight of the embankment. All piles below the toe of the embankment are usually inclined, the inclination being governed by the lateral earth pressure in the embankment. The stability of an embankment can be analyzed by assuming that failure takes place along a circular failure surface, and end effects are generally not considered. In most cases, rock-fill or well-graded sand and gravel are used in the embankment.

Smoltczyk (1966) described a case where a 7 m high railway embankment was supported on piles. The spacing of the piles was 1.3 m, and the pile caps covered 35% of the base area of the fill.

6.2.9 Soil investigation and control methods

6.2.9.1 Soil investigation

For pile design, information is required from field and laboratory tests about the strength and deformation properties of the soil, both above and below the foundation level. Information about the location of the ground water level and the properties of the soil above the foundation level is necessary for the assessment of difficulties to be encountered during driving or installation of the piles. It is important to evaluate accurately the decrease in shear strength of the soil between the piles caused by remoulding of the clay, since this decrease in shear strength can affect the stability.

In Europe, the length and bearing capacity of point bearing piles to be driven through a deep layer of soft clay are frequently evaluated from the results of dynamic and static penetration tests. Field vane tests or unconfined compression tests and fall-cone tests on undisturbed samples are generally required in order to evaluate the bearing capacity of floating piles in soft clay. Plate load tests and pressuremeter tests can also be used to estimate the shear strength and deformation properties of clays. Results from consolidation tests on undisturbed samples are used to determine the compressibility of the soil for settlement calculations. It is often difficult, however, to detect thin sand or silt seams present in the soil, and these will govern the time required for consolidation of the clay if they are continuous.

6.2.9.2 Load tests

Load tests are sometimes used to evaluate the ultimate bearing capacity of piles when the number of piles for a project is large and there is sufficient time available to carry out such tests. A few trial piles are often driven so that any difficulties during driving can be detected (e.g. stones or boulders in the soil, and vibrations and noise).

Long precast concrete piles are sometimes provided with a central pipe so that the piles can be inspected after driving; the straightness can be checked with a steel rod or pipe which is lowered down into the inspection pipe. Slope indicators are also used (Kallstenius & Bergau, 1961). The length and diameter of the steel rod or pipe are chosen such that the rod or pipe will not pass through the inspection pipe when the curvature of the pile axis is less than a certain predetermined value, e.g. 400 m. The allowable load is reduced if the rod cannot be lowered to the bottom. The reduction depends on the radius of the deformed pile, the angle change at the pile joints, whether the anticipated loading is of short or long duration, the shear strength of the surrounding soil, and the dimensions of the piles, as discussed in Section 6.2.5.

The most common pile load test is the 'maintained load' method (ML method) in which the load is increased in steps or increments. The load is not increased until the settlement rate is less than a given predetermined value, e.g. 0.05 mm/30 min. The maximum load is generally equal to twice

the design load, but in Great Britain the maximum load is usually only 1.5 times the design load (Butler & Morton, 1971). A pile test should generally statisfy certain requirements with respect to total settlement at the maximum load, total settlement at the working load and permanent set after unloading from the maximum applied load.

A load test can also be carried out at a constant rate of penetration, e.g. 0.5 mm/min. The CRP method is considerably faster than the ML method and is, therefore, cheaper (Fellenius, 1971b). An additional advantage is that test results are relatively easy to interpret.

The interpretation of an ML test is simplified if the length of each load increment is constant (e.g. 10 minutes). The creep during each load increment can then be plotted as a function of the applied load. The intersection between the two straight portions of the plot represents the *critical pile load* above which very large long-term displacements can be expected.

Neither the CRP method nor the ML method provides information on the long-term settlements of a single pile or a pile group. A separate settlement calculation is generally required. Sometimes, a marked peak point is obtained in CRP tests, but it is more common that the resistance to penetration of the pile increases gradually with penetration. When no peak load is obtained, the failure load is generally taken to be the resistance when the load-deformation relationship shows a marked change in slope. When the axial load continues to increase gradually with increasing deformation, different methods have been used to define the ultimate load. With the so-called '90% or 80%-methods', deformation at 90% or 80% of the ultimate load is assumed to be equal to half the deformation at the failure load (Hjeldnes, 1971). The ultimate bearing capacity has also been defined as the point where the tangents to the first and last parts of the load-deformation curve intersect. These methods, however, involve appreciable subjectivity in interpretation.

Frequently, a combination of the ML and CRP methods is used. The load is first applied in increments (ML method) up to the critical pile load, beyond which the pile is loaded at a constant rate of penetration (CRP method). A test should not be terminated until the total deformation corresponds to at least 20% of the pile diameter or exceeds the deformation at the peak resistance by at least 20 mm. It is suggested that the deformation rate should be decreased beyond the peak point to one-tenth the standard rate for about five minutes, so that the effect of the loading rate on the ultimate strength can be evaluated.

It is sometimes important to separate the skin resistance of a pile from the point resistance, particularly for long piles ($L>20$ to 30 m), since only the point resistance and the friction resistance along the lower part of the pile can often be utilized to carry the applied load. Strain gauges and load cells are sometimes used to separate skin resistance and point resistance. Van Weele (1957) suggested a separation method based on the total applied load and the elastic component of the deformation.

Tell-tales have been used in Sweden to separate skin resistance and point resistance in long piles by the provision of a central inspection pipe (Broms & Hellman, 1968), to the inside of which a separate steel tube is attached about 3 m from the bottom of the pile. Inside the steel tube is a central rod which extends to the bottom of the pile. With this system, it is possible to measure separately the compression of the bottom 3 m of the pile as well as the total compression of the pile. From these two measurements, the portion of the total load transferred to the pile tip can be evaluated. In the interpretation of the test data, it is usually assumed that the pile is initially stress free, and for soft clays, this simplification is generally acceptable (Fellenius & Broms, 1969), At low load levels, the applied load is carried mainly by skin friction. When the axial load is increased, a larger portion is transferred to the pile point. For a 50 m long, 0.3×0.3 m precast concrete pile driven through soft clay of 30 kPa average shear strength, it was reported by Fellenius & Broms (1969) that the load had to exceed approximately 1.8 MN before any load was transferred to the pile point.

It is important to note that, during load tests, the applied load should be measured accurately by means of a separate load cell inserted between the hydraulic jack and the pile head, since the friction in the jack can be as large as 5 to 10% of the total load.

6.3 RAFT FOUNDATIONS

Buildings normally have one or two, and sometimes more, basement floors. The number is mainly restricted by the stability of the excavation for the basement. If the ground water elevation is above the foundation level, either the ground water-table has to be lowered or the basement has to be made watertight. The latter alternative should be chosen in areas with soft clay, since a lowering of the water-table can cause regional subsidence, which can result in damage to water and sewer lines, harmful settlements of existing buildings, and decay of timber piles. Consequently, raft or piled-raft foundations of reinforced concrete are very common in areas with soft clay. The ground water pressure acting on the raft can be ultilized to carry part of the weight of the building, and the number of piles can then be reduced. Piles may not be required if the raft foundation is placed at such a depth that the weight of the excavated soil is equal to the weight of the building, even when the subsoil consists of very soft clay.

6.3.1 Raft-soil interaction

6.3.1.1 Methods of analysis

An important aspect in the analysis of raft foundations is soil-structure interaction. This interaction, however, is very complicated, and very few

in-situ measurements have been made to check the accuracy of existing theories. Problems arise mainly from the inhomogeneity and complexity of soil which make it extremely difficult to develop a simple but realistic constitutive model describing soil behaviour under general loading conditions. The distribution of contact stresses below a raft depends upon the stiffness of the raft, the deformation characteristics of the soil and the stiffness of the superstructure. Such factors as (1) the deformations caused by consolidation, creep and plastic flow, (2) the formation of plastic hinges in the superstructure and yield lines in the raft, and (3) the generation of cracks in the structure, gradually alter the relative stiffness of the subsoil, the raft and the superstructure, as well as the stress distribution below the raft, in an almost unpredictable way. The stress changes in the soil during construction are of decisive importance for the performance of the completed foundation.

Because of the many factors involved, it is not possible to analyse completely satisfactorily a raft foundation. The advent of the finite element method (FEM), however, has made possible much more realistic analytical solutions to this and similar problems. The main restrictions are the storage capacity of the computers, the costs of the computation, and the evaluation of the parameters used in the analysis. The answers obtained, of course, can never be more reliable than the input parameters, and full-scale studies with complete descriptions of the soil and structure are highly desirable to check whether refinements in the analytical methods are justified. Approximate methods of analysis, based on more or less simplified soil models in combination with sound engineering judgement, will always be important: detailed analyses based on more precise soil models and on the finite element method are needed only for the more complicated cases.

The number of published cases concerned with raft foundation analysis is very large; the reader should see the reference lists given by Gorbunov-Possadov & Davydov (1973), Hain (1975) and Lee (1975). Only the most important recent contributions are mentioned in this chapter, and an attempt is made to highlight what are believed to be the most important factors in the design and construction of raft foundations.

6.3.1.2 The Winkler model

Before the 'computer age', the calculation of contact stresses was based on the assumption that the soil could be replaced by a series of independent elastic springs (the *Winkler model* of 1867) and that, consequently, no shear stresses could be taken by the subsoil nor induced in the soil by the loading. As a result, the contact stresses were evenly distributed for a symmetrically loaded infinitely rigid slab as well as for a uniformly loaded flexible slab. A common way to correct this result is to increase the spring constant near the edges of the slab. The application of horizontal springs in the soil model is another common method to adapt as closely as possible the model to real soil

behaviour. Even if these refinements are adopted, however, a difficulty and serious drawback of the method is that the spring constant (the *modulus of subgrade reaction*) is a function not only of the stress-strain characteristics of the soil but also of the dimensions and stiffnesses of the raft slab and the superstructure. In spite of the obvious drawbacks, however, the elastic spring model, even in its simplest form, has become very popular and is extensively used, even in finite element analyses.

The concept, use and evaluation of the modulus of subgrade reaction was fully discussed by Terzaghi (1955). The results obtained from its use are quite acceptable under loading conditions where the basic assumptions hold as, for example, at the centre of a large foundation slab. Near the edges, where the shear stresses induced in the soil become important, the results are unreliable unless the modulus of subgrade reaction is varied in the manner mentioned above.

6.3.1.3 The elastic base model

Another very common assumption, probably first applied in the USSR by Gersevanov in 1934 (c.f. Gorbunov-Possadov & Davidov, 1973), is to consider the subsoil as a linearly elastic, weight-less, isotropic and homogeneous half-space. Practical solutions for various structural systems and loading conditions based on this soil model have been developed both in the USSR and in Western Europe.

In the USSR, tables and diagrams for the application of the elastic supporting soil hypothesis were worked out mainly by Gorbunov-Possadov and Simvulidi. This approach is widely used, as indicated fairly recently by Gorbunov-Possadov & Davydov (1973). According to a list published by Meshcheryakow in 1957 (c.f. Gorbunov-Possadov & Davidov, 1973), 620 papers on the design of structures on elastic supporting soil were published in the USSR between 1917 and 1967. The elastic base method is not generally accepted outside the USSR, although interest in its use is increasing. In Germany, for example, its use is now recommended for the analysis of soil-structure interaction problems (DIN 4010). The results of raft foundation analyses based upon the Winkler model and the elastic base model, however, are sometimes dramatically different, as shown in Fig. 6.22 (Lee, 1975).

The elastic base theory has recently been refined (c.f. Hain, 1975). For example, solutions are available which take account of an increasing soil deformation modulus with depth; such an increase is common in practice. Furthermore, soil with different elastic properties in the horizontal and vertical directions, as well as layered soils and soils with finite thickness, can be analysed.

In order to make the soil model resemble the real conditions as closely as possible, the subsoil is often divided into layers, each with finite thickness and constant deformation properties. If the analysis is based on the finite element method, the superstructure and foundation are represented by conventional

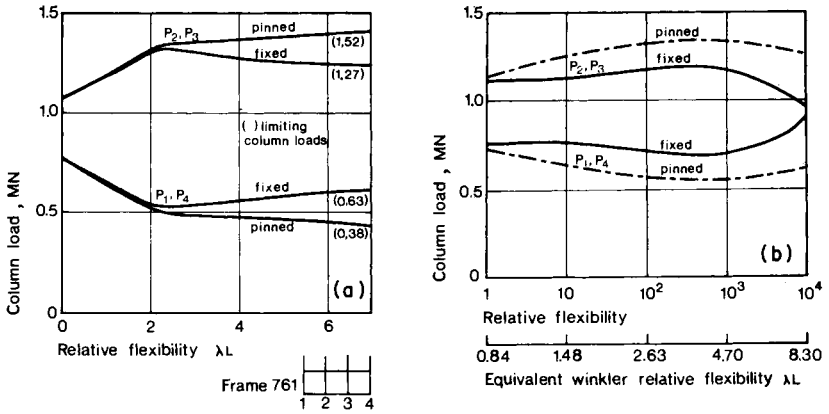

Fig. 6.22. Influence of raft flexibility on column loads for a three-bay multi-storey plane frame with columns 'pinned' or 'fixed' to a raft foundation using: (a) Winkler model, (b) elastic base model. ($E_s = 1.27 \times 10^3$ kPa and $K = 479$ kPa with $\lambda L = \sqrt[4]{KL^4/4E_rI_r}$; $L =$ length and $E_rI_r =$ flexual rigidity of raft) (Lee, 1975).

finite plate and beam elements and the subsoil by surface elements. The settlements are obtained by the summation of the vertical deformations of each layer. The stress distribution in the soil is usually calculated from the Boussinesq theory, but Fröhlich's theory has also been used (Beigler, 1976). Fröhlich's theory has some advantages since, by choosing a proper value of the stress concentration factor, it is possible to take into account the weight of the soil, the friction between the slab and the subsoil, the increase in modulus of elasticity of the subsoil, firm upper or lower layers, etc. Two recent examples of this approach are described below.

Beigler (1976) used Fröhlich's stress theory and the tangent modulus determined by oedometer tests to calculate the vertical displacement of the nodes. The oedometer modulus is defined by:

$$M = m\,\sigma_r \left(\frac{\sigma_v'}{\sigma_r}\right)^{1-\beta} \tag{6.21}$$

where m and β are soil parameters obtained from conventional oedometer tests, σ_v' is the effective vertical stress and σ_r is a reference stress (100 kPa). For normally consolidated clay, it can be assumed that $\beta = 0$.

Wood & Larnach (1975) used in their soil model the incremental tangent modulus approach described by Duncan & Chang (1970). The stress distribution in the subsoil was calculated from Boussinesq's theory, while the elastic tangent modulus was defined (slightly rewritten) by:

$$E = \left[1 - \frac{R_f(1\text{-}\sin\varphi')\,(\sigma_1 - \sigma_3)}{2c'\cos\varphi' + 2\sigma_3'\sin\varphi'}\right] m_e\,\sigma_r \left(\frac{\sigma_3'}{\sigma_r}\right)^{1-\beta_e} \tag{6.22}$$

where m_e, β_e and R_f are soil parameters obtained from conventional drained triaxial tests, c' and φ' are the effective strength parameters obtained from conventional triaxial tests, σ_1 and σ_3 are the major and minor principal stresses, and σ_r is a reference stress (100 kPa).

The layered elastic base model will not yield acceptable results at high shear stresses in the soil, since neither Boussinesq's nor Fröhlich's stress theories describe the real stress distribution with sufficient accuracy.

6.3.1.4 The elastic-plastic base model

Although the introduction of elastic base models is an improvement over the Winkler model, the analytical results obtained from their use may not be accurate enough. For example, the weight of the soil and the friction between the slab and the soil are not usually considered; neither are the stress concentrations at the edges of the slab, which are often so large that the shear strength of the soil is exceeded. These factors cause a redistribution of the contact pressure towards the centre of the slab but, for example, the models used by Beigler (1976) and Wood & Larnach (1975) are inadequate to predict the growth of possible plastic zones in the soil as the shear stresses approach the ultimate strength of the soil. The decrease in modulus of elasticity with increasing stress level, as assumed by Wood & Larnach, changes only the stiffness ratio of the raft/subsoil and does not reflect the local softening at the edges of the raft.

Hain (1975) suggested a simplified iteration method termed *excess strength cut-off analysis*, where the nodal forces of the surface elements which exceed the ultimate bearing pressure are deleted from the analysis. The reactions at these nodes are held constant at their ultimate values, and excess reactions are redistributed to adjacent, unyielded areas. This transfer of load is carried out in steps until no further yielding occurs.

A similar approach has recently been suggested by Beigler (1976), in which the contact stresses are calculated by Boussinesq's theory but are not allowed to exceed the bearing pressure of the soil (Fig. 6.23). The stresses that exceed the bearing capacity of the soil are transferred to adjacent unyielded areas in the same way as in Hain's method. Beigler showed that the contact stresses under a point or line load on a raft can be approximated by a cone or triangle of base radius r, given by:

$$r = 1.3\ K_r < 0.5\ b \tag{6.23}$$

where $K_r = t\sqrt[3]{E_r/E_s}$, and E_r is the modulus of elasticity of the raft, E_s is the modulus of elasticity of the subsoil, t is the thickness of the raft, and b is the width of the raft.

The peak contact pressure, q_1, is thus $3Q/\pi r^2$ for a point load Q, and P/r for a line load P. If the load is applied at a distance a from the edge which is less

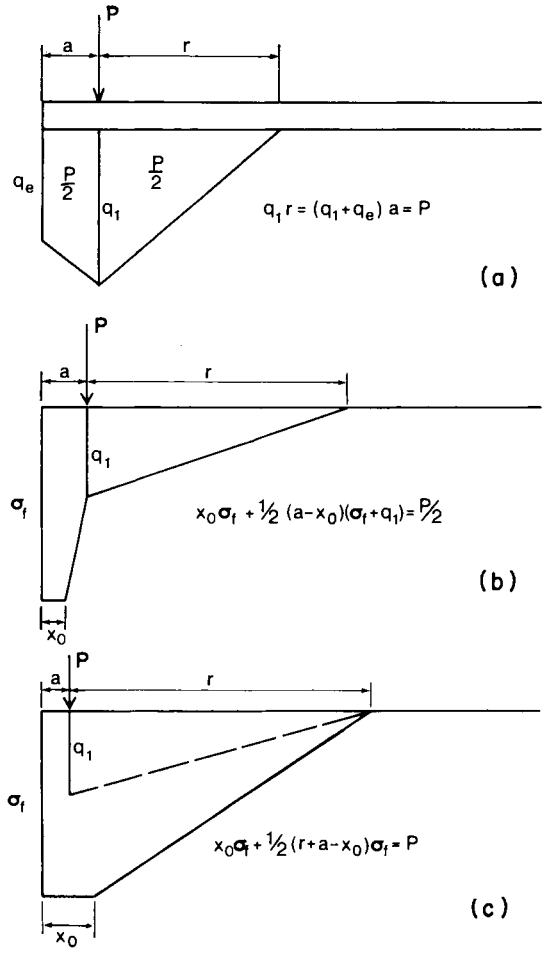

Fig. 6.23. Different cases of stress distribution at raft-soil interface caused by a line load P applied near the edge of the raft. (Beigler, 1976).

than r, the edge pressure, q_e, can be estimated from:

$$q_e = q_1 \frac{r-a}{a} \tag{6.24}$$

as shown in Fig. 6.23a. The contact pressure cannot exceed the bearing capacity, σ_f, of the soil, however, and if this is exceeded, the pressure has to be redistributed in the way described above.

For a line load $P<2a\ \sigma_f$, the contact pressure will be equal to σ_f over a distance x_o from the edge, where:

$$x_o = \frac{P-a(\sigma_f+q_1)}{\sigma_f-q_1} \tag{6.25}$$

The contact pressure then decreases linearly to q_1 below the line load and linearly to zero at a distance r from the load (Fig. 6.23b). If $P > 2a\ \sigma_f$, the distance x_o up to which the contact pressure is equal to σ_f can be calculated from:

$$x_o = \frac{2P - \sigma_f\,(a+r)}{q_f} \tag{6.26}$$

From x_o, the pressure decreases linearly to zero at a distance $(a+r)$ from the edge (Fig. 6.23c). The contact pressure due to point loads near the edge can be calculated in a similar way.

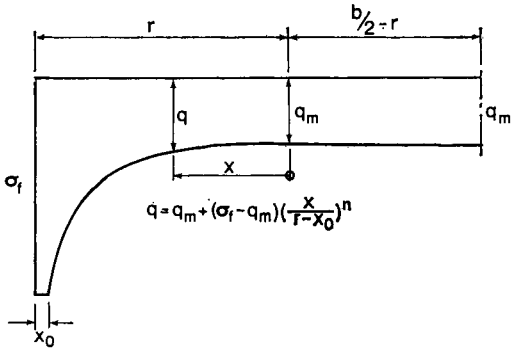

Fig. 6.24. Stress distribution at raft-soil interface under a uniformily distributed load (Beigler, 1976).

For a uniformly distributed load, p, on the raft, the contact pressure distribution follows the relation shown in Fig. 6.24, namely:

$$q_x = q_m + (\sigma_f - q_m)\left(\frac{x}{r - x_o}\right)^n \tag{6.27}$$

where σ_f is the ultimate bearing pressure of the soil at raft level, and:

$$n = 2.8\ \sigma_f/p,$$

$$q_m = p(1 - m_1)\,(1 + m_2)$$

$$m_1 = \frac{0.363}{1 - 0.05\,\dfrac{b}{1.3K_r} + 0.4\left(\dfrac{b}{1.3K_r}\right)^3},$$

$$m_2 = 0.3\ (p/\sigma_f)\ (m_1/0.363),$$

r and K_r are as defined in Eq. 6.23,

$$x_0 = \left\{\frac{(n+1)\,[m_1 - m_2\,(1-m_1)]}{2n(\sigma_f - q_m)/p} - \frac{r}{nb}\right\} b \geqslant 0;\ \text{(if negative, } x_0 = 0\text{)},$$

and x is the distance coordinate from the centre-line of the raft or, if $1.3K_r < b/2$, the distance from a point located $1.3K_r$ from the edge of the raft.

If no yielding occurs, i.e. $x_0 = 0$, the edge pressure will be:

$$q_e = q_m + p(n+1)\,[m_1 - m_2\,(1-m_1)]\,b/2r \tag{6.28}$$

The superstructure, raft and soil can also be treated as parts of a continuous system. The soil is then divided into a number of rectangular, trapezoidal or triangular elements in a two-dimensional analysis, and into prismatic or tetrahedral elements in a three-dimensional analysis. Artificial boundary conditions are imposed. From the stress-strain characteristics of the structure and the soil elements, a 'true' solution can, in theory, always be obtained but, in practice, the method is restricted by the limitations of the memory of the computer and the costs of the calculations. These practical difficulties are combined with the difficulties of evaluating reliable physical parameters and other factors for the soil elements (initial in-situ stress conditions, long-term effects, yielding, etc) and the superstructure (cracking, etc); these are probably the main reasons why it is so difficult to find examples in the literature of this advanced use of the finite element method. Most of the work has been done in the USA and in Great Britain. Some relevant papers were presented at the Conference on the Settlement of Structures held in Cambridge, England in 1974 (see, for example, Crowser et al, 1974; King & Chandresekaran, 1974; Naylor & Hooper, 1974). In the foreseeable future, the finite element method will be used mainly to solve some significant typical examples rather than to assist in daily design work.

6.3.2 Piled rafts

Pile supported raft foundations are even more complicated than unpiled rafts. The piles either act as 'reinforcement' in the soil under the raft (friction, or floating piles) or as point supports (end-bearing piles). In the latter case, a free space between the base of the raft and the subsoil must be taken into account. This free space can be caused by settlement from the disturbance during pile driving or by a lowering of the ground water-table (see Section 6.2.6). The design of the raft must then be based on the assumption that the piles act as upward 'column loads' and that the soil between the piles is not in contact with the raft. The same conditions will prevail also for friction piles in soft sensitive clays. Only when the clay is insensitive or when the spacing of the piles is large, can direct contact between the base of the raft and the soil be guaranteed. The load distribution, therefore, can only be estimated.

Obviously, a rigorous analysis of the load distribution under piled raft foundations is not possible. Even in the case where the soil is assumed to behave as a homogeneous elastic half-space, the problem is difficult to solve, since the number of equations which have to be solved is very large, and sufficiently large computers are not yet available.

Hain (1975) summarized the methods which have been developed to analyse piled raft foundations. These methods are based on the assumption that the soil behaves as a homogeneous and ideally elastic material. The most interesting of these methods is the one used by Hain. The first step in the analysis is performed as if the soil were elastic. In the succeeding steps, the calculations are corrected so that the pile load does not exceed the ultimate bearing capacity of the piles. If this happens, then in the next calculation cycle the excess pile loads are kept constant at the ultimate value, and the compatibility condition between the piles in question and the raft is deleted from the analysis *(load cut-off method)*. The analysis is continued until all pile loads are less than or equal to the ultimate bearing capacity. The results of a raft-pile load analysis (uniformly loaded square raft of length l and width $b=50\,d$; 36 piles with diameter d and length $100\,d$; pile spacing$=8.33\,d$), based on the assumption that the piles are incompressible, are given in Fig. 6.25 (Fig. 6.25a has been obtained without correction, and Fig. 6.25b with correction according to the

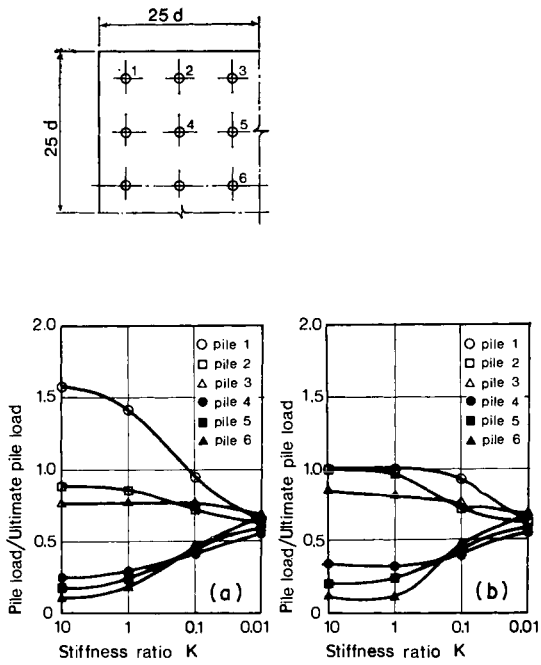

Fig. 6.25. Results of a raft-pile analysis carried out by Hain (1975): (a) elastic analysis, (b) load cut-off analysis.

load cut-off method). The stiffness ratio, K, in Fig. 6.25 is defined by the relationship:

$$K = \frac{4E_r b\, t^3\, (1-\upsilon_s^2)}{3\pi E_s l^4} \tag{6.29}$$

where E_r is the modulus of elasticity of the raft, t is the thickness of the slab, E_s is the modulus of elasticity of the soil, and υ_s is Poisson's ratio of the soil. Poisson's ratio of the raft material was taken as 0.15.

Measurements of contact stresses at the raft-soil interface and of pile loads published by Hooper (1973) showed good agreement between measured values and the elastic FEM analysis. The significance of these results should not be overemphasized, however, since the pile-soil system studied was particularly well suited for an elastic analysis (bored piles in a deep deposit of stiff London clay). Unfortunately, no similar field studies have been made for driven piles in soft clay, although such studies are highly desirable. The results of computer analyses can be misleading and dangerous in that they give an impression of accuracy. Such results should be used with the utmost care, therefore, and only after a thorough analysis has been made of the validity, or lack of validity, of the input parameters.

The distributions of compressions with depth which have been measured below the base of piled raft foundations in soft highly sensitive clays (Hansbo et al, 1973) seem to justify analysis on the basis of the following assumptions:

(1) No contact stresses exist at the raft-soil interface.
(2) The piles behave as columns with lengths equal to two-thirds of the total pile lengths.
(3) The soil-column interface is located at a depth equal to two-thirds of the pile length.

The analysis can be carried out by the finite element method. The raft is represented by finite bending elements, the piles by concentrated upward spring loads at the base of the raft, and the subsoil at a depth equal to two-thirds of the pile length by surface elements. In the case of a uniform soil, the approximate solution suggested by Beigler (Section 6.3.1.4) can also be used.

6.3.3 Practical aspects of rafts

Raft foundations in soft clay are generally placed below the ground water-table, and water-tight concrete is normally used. The word *water-tight* can be misleading, in that it gives the impression that no transport of water takes place through the concrete, whereas the coefficient of permeability of water-tight concrete with a water/cement ratio of 0.5 to 0.6 varies between 0.03×10^{-10} and 0.9×10^{-10} m/sec, which is nearly of the same magnitude as the permeability of the underlaying clay. The drop in pore pressure through the concrete

slab causes an upward flow of pore water and a decrease in the pore pressure in the clay. In the case of heated basements, the evaporation of the water also contributes to a reduction in the pore pressure. If the water balance below the raft is not maintained, therefore, the pore pressure in the clay will decrease and will be followed by consolidation settlements. For clays with a low permeability, a continuous layer of coarse-grained material (sand or gravel) should therefore be placed between the bottom concrete slab and the clay, and the water pressure in this layer should be the same as the original pore water pressure in the underlying clay.

Bottom heave caused by unloading during excavation for the basement of a building can sometimes cause problems, particularly when the factor of safety against failure by bottom heave is low, and it is often necessary to excavate and cast the concrete slab in strips to maintain a sufficiently high factor of safety. For piled rafts, bottom heave of excavations can be almost completely eliminated if the piles are installed prior to excavation (Hansbo et al, 1973).

6.3.4 Concluding remarks

The restraint from the embedment of the raft and/or piles is usually not considered in a raft analysis. This simplification can lead to serious errors, particularly for dynamically loaded structures (e.g. high towers subjected to wind forces) or for stability analysis, where the results can be on the unsafe side.

Other important factors, hitherto ignored, are the tensile cracks that are always present in concrete structures, unless prestressed, and the time of construction. As regards the influence of long-term effects (e.g. stress redistribution at the soil/raft interface because of consolidation of clay, creep of concrete and plastic yielding), only brief treatments can be found in the literature.

The stress changes in the subsoil caused by an excavation are very seldom taken into account. As previously pointed out, these stress changes are important, since the modulus of deformation at unloading and reloading can differ considerably from the modulus of deformation at loading.

In raft-pile analysis, it has to be observed that most of the driven piles are initially in tension because of ground heave caused by the driving of adjacent piles. On the other hand, the piles driven last will be in compression. Because of the reconsolidation of the clay, all the piles will soon be loaded in compression. This factor will probably only have a minor effect on the behaviour of the constructed raft and, therefore, can usually be neglected.

The importance of the stiffness of the raft and the superstructure is not always realized in the design of floating foundations. Thus, it is sometimes assumed that "when the weight of the structure is equal to the weight of the soil excavated, there will be no change in the stresses in the soils beneath the structure and therefore little settlement" (Sowers, 1962). This is only the case

when the superstructure (including the raft and any piles) has the same stiffness as the excavated soil, and only conventional cylindrical oil or water tanks with steel bottom plates have such a low stiffness.

The finite element method will no doubt be used more and more to take into account the superstructure-raft-soil interaction as improved computers are developed. It is necessary, therefore, that every possible effort be made to study and adapt the input parameters as closely as possible to the real values. The old methods, based on the Winkler model and the homogeneous elastic half-space model, will most probably be replaced by other methods which are more representative of the real in-situ soil behaviour. Due regard must be paid to the stress change and the stress level during and after the construction period.

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Chapter 7

CONSOLIDATION AND SETTLEMENT OF SOFT CLAY

by

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Chapter 7

CONSOLIDATION AND SETTLEMENT OF SOFT CLAY

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7.1 INTRODUCTION

All soils are compressible, in that they undergo volume changes when they are subjected to changes in the stresses applied to them. The resulting compressions can be particularly large when the drainage is not impeded, but their magnitudes are of engineering significance only when reference is made to the deformations which are tolerable for a given type of structure. The magnitude and rate of the deformation depend on the type of soil and on the nature of the applied loads. While settlement problems have occupied civil engineers for over a century, a rational quantitative approach has only been available since the introduction of Terzaghi's consolidation theory (Terzaghi, 1923), which is considered by many to be the birth of modern soil mechanics. Since then, a great number of contributions have been made. All of these have attempted to improve the capability of predicting the magnitude and rate of settlement and the rate of excess pore pressure dissipation by introducing more refined soil models and less restricted assumptions on the parameters describing these models. The availability of electronic computing facilities has relatively recently created a boom in the application of numerical techniques, among which the finite element method has proved to be a most versatile and useful tool. Methods to determine the relevant soil parameters in the laboratory and in the field have also been improved, although the progress in this respect has not kept pace with the analytical techniques. Most recently, attempts have also been made to employ probabilistic techniques, because one of the inherent difficulties in achieving more accurate predictions of the settlement process lies in the variability of the soil parameters.

In spite of all these improvements, prediction of settlement has remained more an art than a routine procedure. While it is possible today to predict the magnitude of total or final settlement within 10 to 20% in many cases (Akagi, 1979), the capability of predicting time-settlement relationships has remained rather poor. It is therefore not surprising that Terzaghi's original consolidation theory (occasionally with some refinements), together with one-dimensional compression models, is still widely used by practising engineers, even in cases where the loading conditions are far from one-dimensional.

The literature on settlement and, in particular, on consolidation theory is immense. A search of the data base of the Asian Information Center for Geotechnical Engineering (AGE) using the keywords *consolidation* or *consolidation test* combined with *theory* or *theoretical* yields no less than 331 titles for the period 1970 to 1979. Very little of this information, however, has found its way into design practice.

Within the scope of this review, the settlement behaviour of embankments is given more weight than other forms of loading. This topic has occupied the Authors over the past few years in the Bangkok area and is of particular relevance in areas of recent marine clays. Mathematical details of consolidation theories will not be treated here, as comprehensive reviews on this subject are available (Lee et al, 1968; Scott & Ko, 1969; Lee & Valliapan, 1974; Schlosser, 1973; and others). Some selected examples of recent developments will, however, be summarized. Also covered only briefly are modern numerical methods; for these the reader may refer to Desai (1975), Christian (1977) Schiffman & Arya (1977) and Murray (1978). Previous reviews of settlement analysis were given by Seed (1965), Ladd (1973), Simons (1974), Perloff (1975), Poulos (1975), Hirst (1976) and Feda (1978).

Emphasis is given here to a more detailed and comparative treatment of the more commonly employed methods of settlement analysis and the evaluation of the appropriate soil parameters.

7.2 COMPONENTS OF SETTLEMENT

7.2.1 Definitions

When a distributed load (or pressure) from a structure is applied to a soft soil stratum, the following three components of settlement are commonly distinguished:

(1) *Immediate settlement* (also called *initial* or *undrained* settlement), which takes place immediately upon load application and, if the soil is saturated, deformation is at constant volume caused by the shear strains beneath the loaded area. Little drainage takes place when the clay has a low permeability. Under the centre-line of the load, the vertical compression is accompanied by lateral expansion. Bjerrum (1972) stated that this component should be identified more correctly as *lateral yield.*

(2) *Consolidation settlement*, which arises because the hydraulic gradient of excess pore pressure set up by the applied load causes the water to drain out from the soil while the stress increment is transferred to the soil skeleton. This is a time-dependent process and produces mainly volume change, but shear deformations are also involved, leading to further settlement.

(3) *Secondary compression settlement* (often also termed *drained creep*), the main part of which takes place after essentially complete dissipation of excess pore water pressures, i.e. at practically constant effective stresses. In practical cases, it is often assumed that secondary compression does not start until after primary consolidation is completed.

There is at present no universal agreement on how to separate consolidation into its primary and secondary components. Opinions also differ regarding the exact mechanism of secondary compression. For example, Barden (1969) characterized primary consolidation as that stage during which compression is dominated and retarded by the rate of pore pressure dissipation, and which tends to follow the classical h^2 scaling law (consolidation time proportional to the square of drainage path length, h). Secondary compression on the other hand is dominated and retarded by viscous creep behaviour, and the process is independent of the drainage path length.

Mesri & Godlewski (1977) voiced the opinion that there should be an interrelationship between compressibility with respect to effective stress and compressibility with respect to time, and that the mechanisms of volume change (e.g. displacement, deformation and aggregation of particles, distortion of adsorbed water films, changes in double-layer thickness, etc) during secondary compression should be basically the same as during primary consolidation. This view implies that creep deformations are not controlled by viscous flow of water.

Bjerrum (1967) proposed a subdivision of the time-settlement relationship into instant and delayed compression. The first component represents the settlement which would result if the excess pore pressures set up by a suddenly applied load could dissipate instantaneously with load application, while the second component is the settlement then developing at constant effective stress. This model essentially disregards the effect of water in the voids. Bjerrum argued that this form of division is most suitable (at least conceptually) to describe the behaviour of the soil structure with respect to effective stress.

Further definitions of terms were proposed by Leonards (1977). He suggested the use of the term *primary consolidation* for the consolidation of that part of the strain versus effective stress curve that is independent of the rate at which the effective stress is applied, i.e. $d\sigma_v'/dt$, while the term *secondary consolidation* should be employed for the part that is dependent on $d\sigma_v'/dt$. The shape of the compression—time curve varies markedly with the magnitude of the load increment ratio. Finally, the term *secondary compression* should be reserved for the volume reduction taking place at more or less constant effective stress.

Another definition, provided by Hawley & Borin (1973), makes use of a state path followed by the soil element during the consolidation process in the void ratio—effective stress plane, and is particularly applicable to 'thick' layers. *Primary consolidation* is then defined as the process during which all

soil elements in a soil mass follow a unique state path, while *secondary consolidation* is characterized by different state paths followed by the elements at different depths in the soil layer or by a unique state path which, however, is much steeper ($\partial e/\partial \sigma_v' \to \infty$) than that obtained from high strain rate tests.

7.2.2 One-dimensional analysis versus two- and three-dimensional approaches

If the thickness of the compressible layer is small relative to the width of a foundation or embankment, strains are essentially one-dimensional, and no immediate settlement occurs; methods of settlement analysis based on one-dimensional models should then give adequate results. In cases where the conditions are clearly three-dimensional (e.g. an isolated foundation on a deep soil layer), one-dimensional analysis will underestimate the total final settlement by neglecting the contribution of the immediate settlement. Davis & Poulos (1968) carried out an analysis of a circular footing on the surface of a two-phase soil (Figs. 7.1 & 7.2) and demonstrated that:

(1) Errors when using one-dimensional conditions rapidly increase with layer depth only for drained Poisson's ratio $\upsilon' > 0.3$, otherwise the ratio of settlement evaluated from an oedometer test to total final settlement is close to unity (Fig. 7.1).

(2) The immediate settlement contributes an increasing proportion to the total final settlement as the relative depth of the soil increases or υ' increases (Fig. 7.2).

Thus, the relative importance of the immediate settlement as part of the total final settlement strongly depends on the value of υ', but also on the geometry of the problem (Davis & Poulos, 1968). Figures 7.1 & 7.2 may also serve as a general guide to the conditions under which a three-dimensional analysis should be selected.

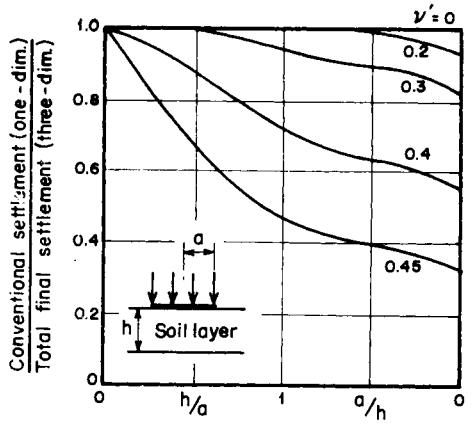


Fig. 7.1. Errors in the conventional one-dimensional approach for total final settlement (Davis & Poulos, 1968).

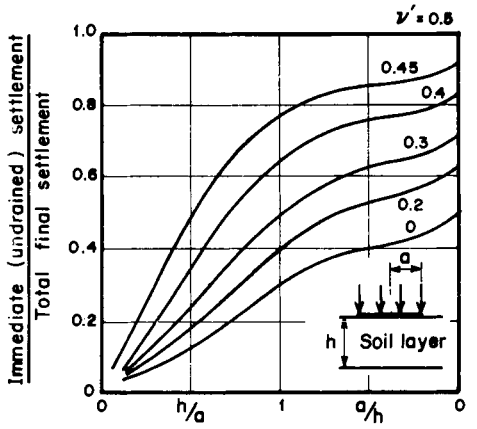


Fig. 7.2. Relative importance of immediate settlement (Davis & Poulos, 1968).

Two- and three-dimensional loading conditions can also produce significant creep movements, both undrained (occurring during the consolidation process) and drained (after dissipation of excess pore pressures). The most significant effect of two- and three-dimensional conditions, however, is on the *rate* of consolidation which, as a result of lateral drainage, can be greatly increased.

7.3 METHODS OF PREDICTING MAGNITUDE OF SETTLEMENT

7.3.1 Principal steps in a settlement analysis

Until the advent of modern numerical methods, a settlement analysis consisted in general of two main steps, namely: (1) an assessment of the stress increment in the compressible layer by the surface load, and (2) the evaluation of the settlement caused by these increments using an appropriate stress-strain relationship (e.g. from oedometer tests) of the soil. Although still widely used, this procedure is no longer the only one available to the designer. The finite element method permits us to evaluate the development of stresses and strains at any point in the soil layer as the surface loads are applied. In addition, it can handle complicated geometry, loading conditions and complex material properties. With this procedure, however, the initial stresses in the ground (i.e. before load application) must be known.

In Table 7.1, the necessary information required for a settlement prediction is summarized. Ladd et al (1977) stated that a prediction capability consists of three components: (1) a model to describe soil behaviour, (2) suitable methods to evaluate the required soil parameters, and (3) computational procedures for applying the model to practical problems. Until now, it has not been possible to describe soil response by a single model for all loading and drainage conditions.

Table 7.1. Components of a settlement analysis (Lambe, 1964, 1967b)

Determination of subsoil section

(1) Vertical and lateral extent of soils; location of compressible soils, drainage surfaces and any special boundary conditions.

(2) Variation of initial pore pressure with depth.

Stress analysis

(2) Initial effective stress versus depth.

(2) Magnitude, distribution and time rate of application of surface load, including any shear stress between ground surface and applied load.

(3) Stress distribution theory compatible with boundary conditions; effect of rigid boundaries or layers.

(4) Variation of σ_1, σ_2, and σ_3 with consolidation; influence of arching, change in Poisson's ratio.

Selection of soil parameters (m_v, C_c, C_r, C_α, σ'_{vc}, k, E_u, E', υ', K_o, A, c_v)*

(1) Representativeness of samples tested.

(2) Sample disturbance.

(3) Environmental factors.

(4) Testing technique.

Estimation of settlement and pore pressures

(1) Method of analysis.

(2) Rotation of principal planes.

(3) Variation of m_v, k, c_v with consolidation.

(4) Secondary compression.

* Definitions of these parameters are given later in the text.

7.3.2 Local yielding during initial settlement

When during undrained loading of a soil the shear stress at some point of the soil mass is relatively high compared to the shear strength, local yield (contained plastic flow) commences. Immediate settlements are usually predicted from linear elastic theory, but the major limitation of this approach is that stress redistribution and strains after local yield has started can no longer be evaluated. D'Appolonia et al (1971) developed from finite element analysis a simplified method to account for local yield under footings when estimating immediate settlement. The method requires three soil parameters: (1) the elastic undrained Young's modulus, E_u, (2) the ultimate bearing stress, q_{ult}, and (3) the initial shear stress ratio, f. The latter is defined as:

$$f = \frac{\sigma'_{vo} - \sigma'_{ho}}{2 s_u} = \frac{1 - K_o}{2\, s_u / \sigma'_{vo}} \tag{7.1}$$

where σ'_{vo} and σ'_{ho} are the initial vertical and horizontal effective stresses respectively, K_o is the coefficient of earth pressure at rest, and s_u is the undrained shear strength of the soil.

Curves giving a settlement ratio, $S_R = \rho_{ie}/\rho_i$, as a function of the applied stress ratio, q/q_{ult}, the initial shear stress ratio, f, and three values of h/B (h=layer depth, and B=foundation width) are shown in Fig. 7.3. The settlement ratio, S_R, expresses the ratio of elastic initial settlement, ρ_{ie} (without considering local yield) to the actual initial settlement, ρ_i (including yield). The figure also indicates the procedure to follow for estimating ρ_i from the given curves. It should be noted, however, that Fig. 7.3 was derived for a strip load on a homogeneous soil layer (constant E_u, s_u and K_o) and an isotropic shear strength. The effect of strength anisotropy is not too important for anisotropic shear strength ratios between 0.6 and 1.0, and if q_{ult} is known accurately. This also applies to the influence of strength nonhomogeneity (e.g. strength increasing with depth), and both effects appear to be small compared to the influence of f, to which S_R is very sensitive. In normally consolidated clays, f ranges from about 0.6 to 0.75, and local yielding first occurs at a factor of safety of 4 to 6 (Davis & Poulos, 1968; Ladd et al, 1977).

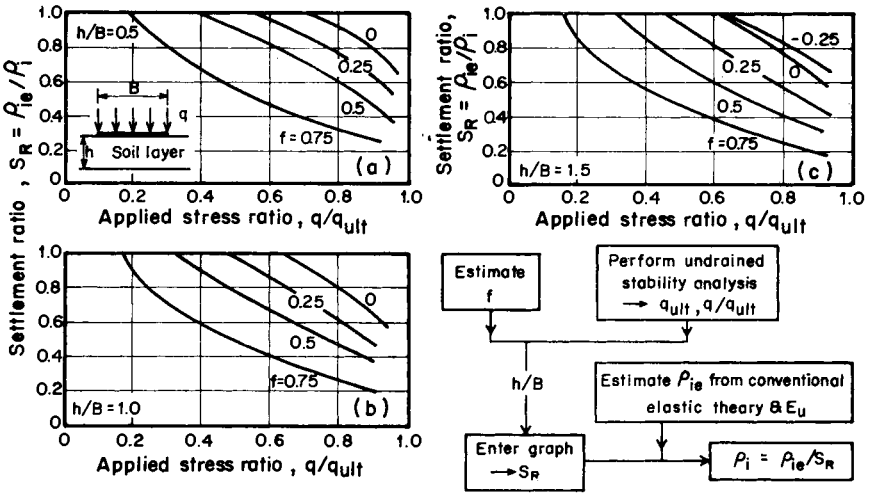


Fig. 7.3. Relationship between settlement ratio and applied stress ratio for strip foundation on homogeneous, isotropic, elastic layer (D'Appolonia et al, 1971).

For embankments ($h/B<0.5$), it appears that local yielding starts to become important when q/q_{ult} exceeds about 0.5 (assuming f=0.5). Tavenas et al (1974) concluded from observations on a test fill at St Alban, Quebec, that settlements deviate from linear elastic behaviour as soon as the stress level exceeds 60% of the failure stress at one location in the foundation. This limit is reached approximately when the fill height has attained 50% of the height at which failure occurs.

7.3.3 Conventional one-dimensional method

Under one-dimensional strain conditions, the immediate settlement, ρ_i, is zero, and the total final settlement, ρ_T, for a soil profile consisting of n layers can be expressed as:

$$\rho_T = \rho_{oed} = \sum_{i=1}^{n} (m_v \cdot \Delta\sigma'_v \cdot \Delta z)_i \tag{7.2}$$

where Δz is the initial layer thickness, $\Delta\sigma'_v$ is the increase in vertical stress at the centre of the layer due to the foundation load, ρ_{oed} is the settlement measured in an oedometer test, and m_v is the coefficient of volume compressibility, given by:

$$m_v = \frac{\Delta\varepsilon_v}{\Delta\sigma'_v} = \frac{\Delta e}{\Delta\sigma'_v(1+e_o)} \tag{7.3}$$

In Eq. 7.3, e_o is the initial void ratio, and ε_v is the vertical strain. This form of solution allows for different soil properties in different layers, Δz_i.

For the general case involving both compression and recompression (i.e. the loading of a soil from the overconsolidated to the normally consolidated state), Eq. 7.2 can be written in terms of the compression and recompression indices, C_c and C_r, as:

$$\rho_T = \sum_{i=1}^{n} \left(\frac{C_r}{1+e_o} \Delta z \log \frac{\sigma'_{vc}}{\sigma'_{vo}} + \frac{C_c}{1+e_c} \Delta z \log \frac{\sigma'_{vf}}{\sigma'_{vc}} \right)_i \tag{7.4}$$

where σ'_{vo} is the initial vertical effective stress (or for field loading, the vertical effective overburden pressure), σ'_{vf} is the final vertical stress, σ'_{vc} is the preconsolidation pressure (σ'_{vm} in case of true precompression from overburden), and e_c is the void ratio corresponding to the intersection of recompression and virgin lines ($e_c \approx e_o$). The terms used in Eq. 7.4 are explained in Fig. 7.4; note that the symbol σ'_{vc} is usually used in connection with a quasi-preconsolidation pressure (Bjerrum, 1972, 1973) while the symbol σ'_{vm} is reserved for preconsolidation caused by a previous maximum overburden pressure (see also Section 7.6.2.1). It should be pointed out here that this method of analysis is one-dimensional only with respect to the integration of strains, whereas the vertical stress increment, $\Delta\sigma'_v$, is usually obtained from three-dimensional elastic stress distribution theory.

In the conventional approach, secondary compression is sometimes taken into account by using the Buisman-Koppejan method. Buisman (1936) established that the settlement of a compressible layer due to secondary compression varies linearly with the logarithm of time and is independent of the layer thickness, while Koppejan (1948) added that the relative settlement is a linear function of the logarithm of the applied load. The secondary

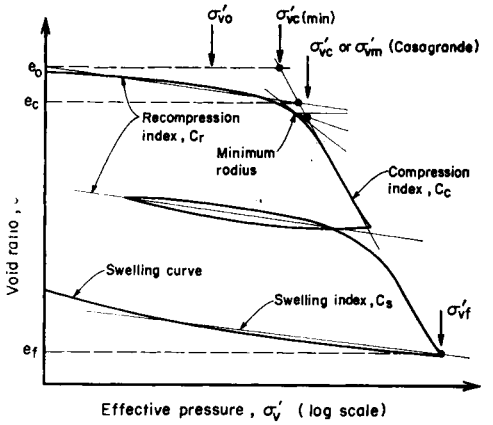


Fig. 7.4. Notation and terminology used for oedometer compression curves.

settlement after time $t=t_p$ (end of primary consolidation) can then be written for a normally consolidated soil as:

$$\Delta\rho_s(t) = 2.3\left(\frac{1}{C_1}+\frac{1}{C_2}\log t_s\right)\log\left(\frac{\sigma'_{vo}+\Delta\sigma'_v}{\sigma'_{vo}}\right)\Delta z \tag{7.5}$$

where C_1 and C_2 are coefficients which can be determined from long-term oedometer tests, and t_s is the time for which secondary compression is to be computed. When compared with Terzaghi's theory (Eq. 7.4), it can be seen that, for the normally consolidated case ($\sigma'_{vf}=\sigma'_{vo}+\Delta\sigma'_v$), $2.3/C_1=C_c/(1+e_o)$. This procedure has been recommended in French civil engineering practice for embankments on soft ground (Bourges et al, 1971; Magnan at al, 1979b).

A simpler approach is used in U.S. practice. Raymond & Wahls (1976) recommended the application of the expression:

$$\rho_s = \sum_{i=1}^{n}\left(\frac{C_\alpha}{1+e_o}\Delta z\right)_i \log\frac{t_s}{t_p} \tag{7.6}$$

for the computation of the secondary compression settlement. C_α is the coefficient of secondary compression, defined here as the change in void ratio per unit change in logarithm of time after the end of primary consolidation (see Section 7.6.4.1). Regarding the selection of the initial time from which all time values have to be measured, it is recommended that the instant at which the load is applied be used in the case of instantaneous load application, while for loads which increase approximately linearly with time, the time at which one-half of the total load has been applied should be used as the initial time. This procedure is also satisfactory for irregularly applied loads, as long as the construction time, t_c, is less than 20% of t_p.

7.3.4 Skempton-Bjerrum method

Realizing the three-dimensional nature of most settlements, Skempton & Bjerrum (1957) developed a method in which the final primary consolidation settlement, ρ_c, was calculated from three-dimensional pore pressures obtained under undrained axisymmetric (triaxial) stress conditions, as:

$$\rho_c = \sum_{i=1}^{n} (m_v \Delta u \Delta z)_i \tag{7.7}$$

in which, for saturated soils:

$$\Delta u = \Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3) \tag{7.8}$$

(whereas in the one-dimensional consolidation theory, $\Delta u = \Delta\sigma_v = \Delta\sigma_1$). $\Delta\sigma_1$ and $\Delta\sigma_3$ are the major and minor principal stress increments, and A is Skempton's pore pressure coefficient (Skempton, 1954). With reference to consolidation settlement computed from oedometer tests, this can be written as:

$$\rho_c = \mu\, \rho_{oed} \tag{7.9}$$

The correction factor μ is a function of the pore pressure coefficient A and the geometry of the problem (Fig. 7.5). In addition, the immediate settlement was calculated from elastic displacement theory. The total final settlement is then:

$$\rho_T = \rho_i + \mu\, \rho_{oed} \tag{7.10}$$

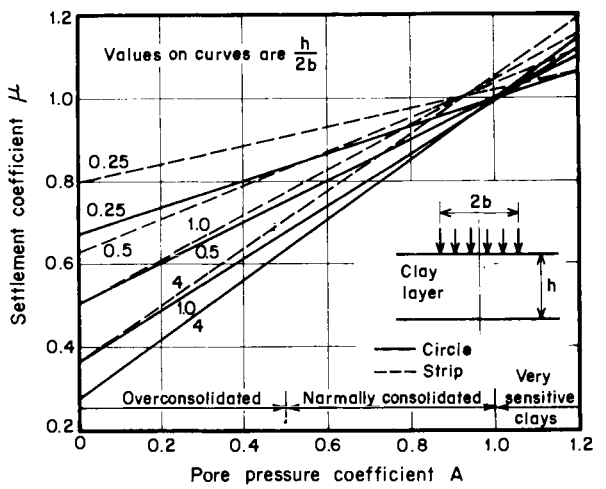


Fig. 7.5. Settlement coefficient of Skempton & Bjerrum versus pore pressure coefficient for circular and strip footings (modified by Scott, 1963).

This method is in a way illogical, since it combines pore pressure values obtained from triaxial type of stress conditions with the one-dimensional consolidation equation (Eq. 7.2). Also, Eq. 7.8 for Δu is strictly valid for triaxial stress conditions only.

The Skempton-Bjerrum method also tacitly assumes that the distribution of total stress imposed by the foundation load remains unchanged during the consolidation process, regardless of the change in Poisson's ratio from its undrained to its drained value.

7.3.5 Stress path method

7.3.5.1 General principle

A more general method which can better take into account the effects of lateral deformation on vertical settlement was described by Lambe (1964, 1967a) and Lambe & Marr (1979). In this procedure, the effective stress paths of selected elements are estimated and simulated as closely as possible in laboratory tests. Figure 7.6 shows the principle in its simplest form for a load which has been applied rapidly compared to the rate of consolidation. Point A represents in-situ conditions, and path AB represents loading along an undrained stress path (no volume change). The axial strain between points A and B corresponds to the immediate settlement. Path BC corresponds to consolidation (dissipation of excess pore pressure at constant deviator stress). The corresponding volumetric strain can be obtained from an oedometer test (stress path AD).

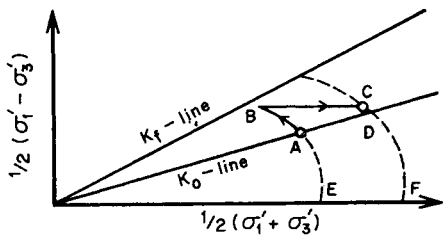


Fig. 7.6. Illustration of stress path method (K_f-line = failure envelope).

Two types of stress path technique can be employed to predict vertical strains: (1) method using stress-strain contours, and (2) method using the results of a laboratory test in which the actual field stress path has been duplicated.

7.3.5.2 Technique using stress-strain contours

This method consists of the following steps:

(1) From a series of consolidated undrained triaxial tests, a family of stress-strain contours is plotted, like that shown for weathered Bangkok clay in Fig. 7.7.

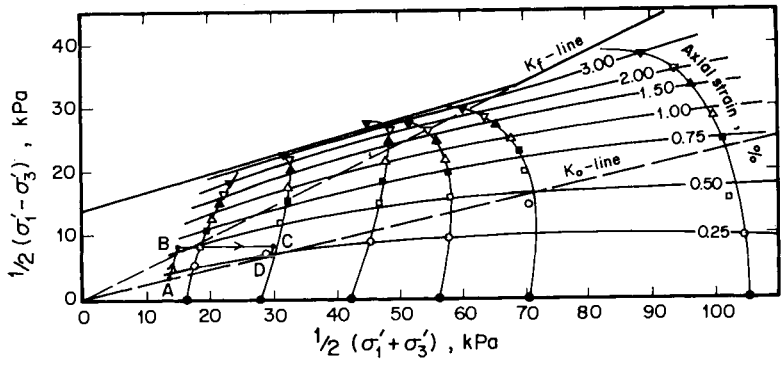


Fig. 7.7. Effective stress paths and strain contours for weathered Bangkok clay from Bangpli (depth 2.5 m).

(2) The estimated effective stress path in the field is superimposed on the stress-strain contours, such as *ABC* in Fig. 7.7.

(3) The vertical strain due to undrained loading, ε_{1u}, can then be obtained directly from the axial strain contours, while the volumetric strain occurring during consolidation is derived from a conventional oedometer test (stress path *AD*, Fig. 7.6). Note that along the stress paths *BC* and *AD*, the volumetric strains are identical because *DC* lies on an undrained stress path. Under zero lateral strain, the relationship between the consolidation component of axial strain, ε_{1c}, and the volumetric strain, ε_v^*, is given by (Lambe, 1964; Šuklje, 1969):

$$\frac{\varepsilon_{1c}}{\varepsilon_v^*} = \frac{1+K_o-2\,K\,K_o}{(1-K_o)\,(1+2K)} \tag{7.11}$$

where K_o is the coefficient of earth pressure at rest, and K is the ratio of the horizontal to the vertical effective stress increment ($K=\Delta\sigma_h'/\Delta\sigma_v'$). (For a horizontal stress path $\varepsilon_{1c}\approx\varepsilon_v^*/3$.)

(4) The total vertical strain is then the sum of the undrained and the consolidation components ($\varepsilon_1=\varepsilon_{1u}+\varepsilon_{1c}$), and when multiplied by the thickness of the layer under consideration, the total final settlement is obtained.

7.3.5.3 Procedure using laboratory tests to duplicate field effective stress path
This technique involves the following steps:

(1) A laboratory triaxial test duplicating the field effective stress path (both undrained and drained components) is performed, and the vertical strain measured.

(2) The settlement is computed by multiplying the measured vertical strain by the thickness of the corresponding layer.

This procedure should be used for final analysis, while the method using stress-strain contours is more suitable for preliminary work (Lambe, 1964). The stress path method requires exceptionally high standards of sampling and testing, which poses a serious limitation to its practical application.

7.3.6. Elastic method

7.3.6.1 General principle

This approach is basically a variation of the stress path method and was proposed by Davis & Poulos (1963, 1968), and in a similar form by Egorov et al (1957) and Kérisel & Quatre (1968) also. Corrections for the effect of local yield (Section 7.3.2) can also be incorporated. By obtaining appropriate elastic constants from laboratory tests, elastic theory is employed to calculate both immediate and consolidation settlement. The total final settlement, ρ_T, is expressed by:

$$\rho_T = \frac{\rho_{ie}}{S_R} + (\rho_{Te} - \rho_{ie}) = \rho_i + \rho_c \tag{7.12}$$

where ρ_{ie} and ρ_{Te} are the immediate and total final settlement respectively calculated from elastic theory, and S_R is the settlement ratio accounting for possible local yield under undrained conditions as derived by D'Appolonia et al (1971) and shown in Fig. 7.3.

The quantities, ρ_{ie} and ρ_{Te} may be obtained by either: (1) summation of vertical strains beneath the foundation, or (2) use of elastic displacement theory.

7.3.6.2 Summation of strains

This method is suitable for non-homogeneous or stratified deposits. The total settlement is given by:

$$\rho_{Te} = \sum_{i=1}^{n} (\Delta\varepsilon_v \, \Delta z)_i = \sum_{i=1}^{n} \frac{1}{E'} [\Delta\sigma_z - \upsilon'(\Delta\sigma_x + \Delta\sigma_y)]_i \, \Delta z_i \tag{7.13a}$$

and the immediate settlement by:

$$\rho_{ie} = \sum_{i=1}^{n} \frac{1}{E_u} [\Delta\sigma_z - \upsilon_u(\Delta\sigma_x + \Delta\sigma_y)]_i \, \Delta z_i \tag{7.13b}$$

where E', E_u, υ' & υ_u are the drained and undrained Young's moduli and Poisson's ratios respectively, and $\Delta\sigma_x$, $\Delta\sigma_y$ & $\Delta\sigma_z$ are the stress increments due to the foundation loading estimated from elastic stress distribution theory for the appropriate value of υ. For immediate settlement on saturated clay, υ_u is taken as 0.5.

Note that Eq. 7.13 is in its form identical to Eq. 7.2 ($\Delta\varepsilon_v = m_v \Delta\sigma_v'$), but the strain increment, $\Delta\varepsilon_v$, is evaluated by a three-dimensional relationship.

7.3.6.3 Elastic displacement theory

This procedure is recommended for cases in which the soil stratum is fairly homogeneous and where it is therefore possible to use average values for E' and E_u. The equations are of the form:

$$\rho_{Te} = \frac{q\,B\,I_\rho}{E'} \tag{7.14a}$$

and

$$\rho_{ie} = \frac{q\,B\,I_\rho}{E_u} \tag{7.14b}$$

where q is the applied foundation pressure, B is some convenient dimension of the foundation (e.g. width), and I_ρ is an influence factor for settlement which can be obtained from elastic displacement theory as a function of Poisson's ratio (υ' or υ_u) and the geometry of the problem (foundation shape and layer thickness). Solutions in the form of graphs for I_ρ for various types of loading have been given by Giroud (1973) and Poulos & Davis (1974). Figure 7.8 shows a graph from which the immediate elastic settlement under an embankment fill on a single soil layer can conveniently be obtained.

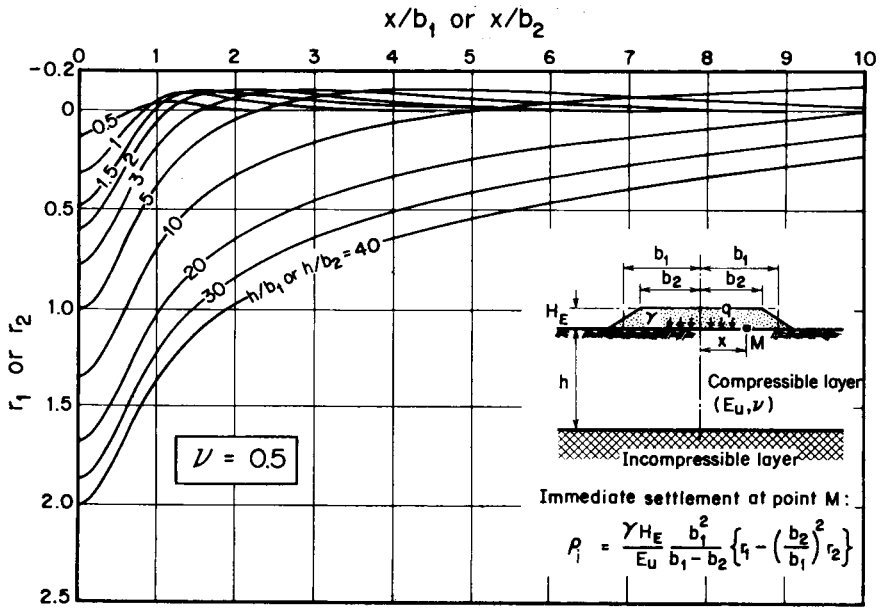


Fig. 7.8. Graph for calculating immediate settlement under embankment load (Giroud, 1973).

The success of the elastic method depends mainly on the values selected for the elastic constants. Since soil does not behave as an ideal elastic material, the elastic parameters depend on the initial stress state and the level of shear stress. The parameters must therefore be evaluated over the correct range of initial effective stress and stress increments. Test procedures to determine E and υ are given in Section 7.6.7.

7.3.6.4 Creep settlements

A practical method to estimate secondary compression or creep settlement under three-dimensional loadings, and which may be considered as an extension to the stress path and elastic methods (Lambe, 1964; Davis & Poulos, 1963, 1968), was proposed by Poulos et al (1976). This method is applicable only to situations in which undrained creep and local yield do not play an important part, i.e. to footings which have a relatively high factor of safety.

The procedure involves the following steps:

(1) The distribution of both the vertical stress and radial stress down the vertical axis beneath a footing is determined from elastic theory using the drained Poisson's ratio.

(2) Samples from various depths are reconsolidated in the laboratory under in-situ stresses (before load application).

(3) Under drainage conditions which best fit the field conditions, the samples are subjected to the calculated vertical and radial stress increments which would occur at the respective depths due to load application.

The axial creep rate is then computed from the axial strain—log time curve. Care must be taken not to include any strains which may arise from initial consolidation of the soil sample. Integration of the axial strain rates over the thickness of the layer gives then the creep rate of the footing. An approximate estimate of the complete time—settlement curve is also possible. Calculated data agreed well ($\pm 25\%$) with measured values from model footing tests on kaolin.

7.3.7 Bjerrum's method

Bjerrum (1972, 1973) emphasized the importance of the effect of the preconsolidation pressure, σ'_{vc}, (or p_c) in settlement calculations. For consolidation settlement, Bjerrum (1972) proposed:

$$\rho_{c1} = \mu_{oc} \sum_{i=1}^{n} (m_v \, \Delta\sigma'_v \, \Delta z)_i \tag{7.15a}$$

for cases in which $\Delta\sigma'_v < (\sigma'_{vc} - \sigma'_{vo})$. If $\Delta\sigma'_v$ exceeds σ'_{vc}, then the settlement should be computed from two components:

$$\rho_{c1} = \mu_{oc} \sum_{i=1}^{n} [m_v (\sigma'_{vc} - \sigma'_{vo}) \Delta z]_i \tag{7.15b}$$

$$\rho_{c2} = \mu_{cp} \sum_{i=1}^{n} \left(\frac{C_c}{1+e_c} \log \frac{\sigma'_{vo} + \Delta\sigma'_v}{\sigma'_{vc}} \Delta z \right)_i \tag{7.15c}$$

where μ_{oc} is the Skempton-Bjerrum μ-factor of Fig. 7.5 in the overconsolidated range, i.e. from σ'_{vo} to σ'_{vc}, and μ_{cp} corresponds to loadings in the 'virgin' or normally consolidated range.

To estimate secondary compression settlements, Bjerrum, inspired by Taylor's time lines for secondary compression (Taylor, 1942), proposed a unique stress-strain relationship as illustrated in Fig. 7.9. This diagram represents the equilibrium values of the void ratio for different values of effective overburden pressures at a specific time of sustained loading. Usually, it is difficult to construct such a diagram unless the approximate age of a deposit and long-term settlement records are available. It is possible, however, to estimate an upper limit of secondary compression from (see Fig. 7.9):

$$\rho_s = \sum_{i=1}^{n} \left(\frac{C_c}{1+e_o} \log \frac{\sigma'_{vc}}{\sigma'_{vo}} \Delta z \right)_i \text{ for } \Delta\sigma'_v > (\sigma'_{vc} - \sigma'_{vo}) \tag{7.16a}$$

This corresponds to a secondary compression which would develop over a time period equal to that over which the clay has developed its σ'_{vc} value. If the load increment, $\Delta\sigma'_v$ is smaller than $(\sigma'_{vc} - \sigma'_{vo})$, the settlement will be:

$$\rho_s = \sum_{i=1}^{n} \left(\frac{C_c}{1+e_o} \log \frac{\sigma'_{vo} + \Delta\sigma'_v}{\sigma'_{vo}} \Delta z \right)_i \text{ for } \Delta\sigma'_v < (\sigma'_{vc} - \sigma'_{vo}) \tag{7.16b}$$

A successful application of the concepts given in Fig. 7.9 to the settlement analysis of buildings, also beyond the time of pore pressure dissipation, has been given by Foss (1969). In this study, it was observed that the settlements were comparatively large when the load increment did not exceed σ'_{vc}. With a true overconsolidation (e.g. by a previous load), such a behaviour would not be expected, and this phenomenon was attributed to the reduction in σ'_{vc} with time because of secondary (or delayed) compression. According to the concept displayed in Fig. 7.9, the (quasi) preconsolidation pressure, σ'_{vc}, should decrease and finally 'vanish' when the consolidation occurs over a long enough period of time. Leonards (1972, 1977) strongly disagreed with this concept and argued that laboratory test results would not show this phenomenon. Leonards asserts that, in his experience, a properly assessed σ'_{vc}-value from carefully run oedometer tests gives good agreement with measured settlements.

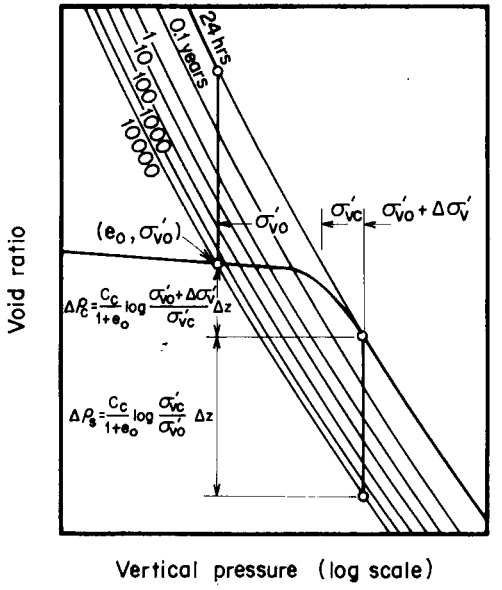


Fig. 7.9. Principle of Bjerrum's method for evaluating primary and secondary settlements (Bjerrum, 1972, 1973).

7.3.8 Critical state models for use in settlement predictions

7.3.8.1 The modified Cam-clay model

The critical state theory was developed as a general stress-strain theory for normally and lightly overconsolidated clays which are treated as an isotropic, elasto-plastic, strain-hardening material. Previous influences of delayed consolidation and time-hardening can also be taken into account. The soil is assumed to possess a yield locus and to have a flow rule which satisfies the normality condition (i.e. the plastic strain increment vector is everywhere normal to the yield locus). For details of this theory, the reader is referred to the publications by Roscoe & Schofield (1963), Roscoe et al (1963), Schofield & Wroth (1968), Roscoe & Burland (1968) and Atkinson & Bransby (1978). The original Cam-clay model, introduced by Roscoe & Schofield (1963), was later modified by Roscoe & Burland (1968), who employed a new expression for the work dissipated during a stress increment to form the *Modified Cam-clay model.* It has been demonstrated that this revised model gives somewhat better predictions of strains in laboratory tests (Roscoe & Burland, 1968; Balasubramaniam & Chaudhry, 1978).

The yield locus forms a boundary of stress states such that the strains for stress paths wholly within this boundary are recoverable and small. Stress states crossing the yield locus will cause large irrecoverable plastic strains. Experimental evidence for the existence of a yield locus has been provided not only for resedimented clay but also for natural clays, such as Leda clay

(Mitchell, 1970), Belfast estuarine clay (Crooks & Graham, 1976) and Bangkok clay (Balasubramaniam & Hwang, 1980).

The concept of *yield* also leads to somewhat different definitions of the terms *normally consolidated* and *overconsolidated* (Burland, 1971). A normally consolidated clay is one that is in a state of yield such that a small increase in stress gives rise to significant irrecoverable strains, whereas an overconsolidated material undergoes relatively small and approximately recoverable strains. The preconsolidation pressure obtained in an oedometer test, for example, represents a yield stress in this sense. Delayed compression increases the yield stress.

The pertinent equations derived by Roscoe & Burland (1968) for the Modified Cam-Clay model will now be given.

For axial symmetry ($\sigma_2=\sigma_3$), the yield locus is defined by the equation:

$$p'_{oy}=p'\left(1+\frac{(q/p')^2}{M^2}\right) \tag{7.17}$$

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 (see Fig. 7.10); p'_{oy} can be obtained experimentally from an isotropic consolidation test; the quantities p' and q are defined as:

$$\begin{aligned} p' &= \tfrac{1}{3}(\sigma'_1+2\sigma'_3) \\ q &= \sigma_1-\sigma_3=\sigma'_1-\sigma'_3 \end{aligned} \tag{7.18}$$

M is the value of the stress ratio q/p' at failure and, for a normally consolidated clay with zero cohesion intercept, is related to the effective angle of friction, φ', by:

$$M=\left(\frac{q}{p'}\right)_f=\frac{6\sin\varphi'}{3-\sin\varphi'} \tag{7.19}$$

The total volumetric strain increment, δv, resulting from an increment of stress causing yield is the sum of a recoverable component and an irrecoverable component. The recoverable component caused by an increment in mean normal stress, $\delta p'$, is:

$$\delta v^r=\frac{\mathrm{K}}{(1+e)\,p'}\,\delta p' \tag{7.20}$$

and the irrecoverable component is:

$$\delta v^p=\frac{\lambda-\mathrm{K}}{1+e}\left[\frac{2\,q/p'}{M^2+(q/p')^2}\left(\delta q-\frac{q}{p'}\,\delta p'\right)+\delta p'\right]\frac{1}{p'} \tag{7.21}$$

The total volumetric strain increment is then:

$$\delta v=\frac{1}{1+e}\left[(\lambda-\mathrm{K})\frac{2\,q/p'}{M^2+(q/p')^2}\left(\delta q-\frac{q}{p'}\,\delta p'\right)\frac{1}{p'}+\lambda\frac{\delta p'}{p'}\right] \tag{7.22}$$

The parameters λ and K are the slopes of the isotropic consolidation line and swelling line respectively. For all practical purposes, they can be expressed as:

$$\lambda = 0.434\ C_c \qquad K = 0.434\ C_s \tag{7.23}$$

where C_s is the swelling index ($\approx C_r$) obtained from an oedometer test.

For q/p'=constant or $\delta(q/p')=0$ (one-dimensional compression), Eq. 7.22 leads to the Terzaghi equation:

$$\left(\frac{\lambda}{1+e}\right)\frac{\delta p'}{p'} = \left(\frac{0.434\ C_c}{1+e}\right)\frac{\delta\sigma_v'}{\sigma_v'} = \delta\varepsilon_v \tag{7.24}$$

where $\delta\varepsilon_v$ is the increment of vertical strain. When we put $\delta v = 0$ in Eq. 7.22, the equations for undrained stress paths or constant void ratio contours can be obtained.

For the shear strain increment, it is assumed that there is no recoverable component, i.e. $\delta\varepsilon^r=0$ and $\delta\varepsilon=\delta\varepsilon^p$. The increment of plastic shear strain is made up of two components:

$$\delta\varepsilon^p = (\delta\varepsilon^p)_{v^p} + (\delta\varepsilon^p)_{q/p'} \tag{7.25a}$$

or:

$$\delta\varepsilon^p = \left(\frac{d\varepsilon^p}{d(q/p')}\right)_{v^p}\delta\left(\frac{q}{p'}\right) + \left(\frac{d\varepsilon^p}{dv^p}\right)_{q/p'}\delta v^p \tag{7.25b}$$

The first component, $[d\varepsilon^p/d(q/p')]_{v^p}$ represents the plastic shear distortion which is caused by the shift of the constant-q yield locus for a state path below the state boundary surface (see Roscoe & Burland (1968) for a definition of these terms). This component is independent of the plastic volume change and is only a function of the stress ratio, q/p'. Experimentally, it can be determined from undrained triaxial tests by establishing a relationship between q/p' and $(\varepsilon)_{v^p}$. The second component in Eq. 7.25 arises from the shear distortion caused by a state path on the state boundary surface when the yield locus is shifted. During yield, the plastic strain increment ratio is uniquely related to the stress ratio (q/p') by the flow rule:

$$\left(\frac{d\varepsilon}{dv^p}\right)_{q/p'} = \frac{2\,(q/p')}{M^2-(q/p')^2} \tag{7.26}$$

Combining Eq. 7.26 with Eq. 7.21 gives:

$$(\delta\varepsilon^p)_{q/p'} = \left(\frac{\lambda-K}{1+e}\right)\left(\frac{2\,q/p'}{M^2-(q/p')^2}\right)\left[\left(\frac{2\,q/p'}{M^2+(q/p')^2}\right)\left(\delta q - \frac{q}{p'}\delta p'\right) + \delta p'\right]\frac{1}{p'} \tag{7.27}$$

Roscoe & Burland (1968) also showed how the plastic strain increments can be approximately expressed for the special case of plane strain by making the simplifying assumption that $K=0$, i.e. $\delta v^r=0$ and, hence $\delta v=\delta v^p$. It should be noted that Eqs. 7.22 & 7.27 are only applicable for stress changes which cause the soil to yield.

7.3.8.2 Application to settlement predictions

Figure 7.10 depicts a possible stress path, *ABCD,* for an embankment loading. In this path, *AB* represents the undrained effective stress path below the yield locus (i.e. in the elastic range). It is described by $\delta v^r=0$ in Eq. 7.20. The theory assumes this portion to be vertical, although this is not in agreement with experiments. The section *BC* corresponds to the undrained stress path *above* the current yield locus, and is described by $\delta v^p=0$. Finally, the portion *CD* represents consolidation.

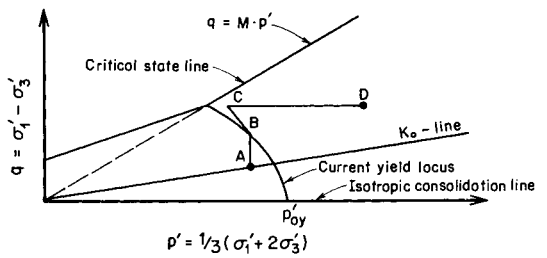


Fig. 7.10. Possible effective stress path for embankment loading predicted by critical state theory.

The immediate settlement is obtained from:

$$\rho_i = \sum_{i=1}^{n} (\delta\varepsilon^p \,.\, \Delta z)_i \tag{7.28}$$

and the consolidation settlement from:

$$\rho_c = \sum_{i=1}^{n} (\delta\varepsilon_{1c}\, \Delta z)_i \tag{7.29}$$

where:

$$\delta\varepsilon_{1c} = (\delta\varepsilon^p)_{q/p'} + \tfrac{1}{3}\,\delta v \tag{7.30}$$

Burland (1969, 1971) gave examples of the practical application of the theory for predicting pore pressures under embankments (with consideration of local yield) as well as vertical and horizontal deformations by using readily available field and laboratory test data. The theory can also estimate the gain in strength during consolidation.

From the equations presented above it can be seen that the Cam-clay model only requires three parameters, namely C_c, C_s and φ', which can readily be

determined from laboratory tests. It appears, however, that in practice, the selection of parameters from tests is not an easy task. This applies particularly to C_s values obtained from oedometer tests. Predicted consolidation settlements by means of the Cam-clay model and the simple one-dimensional oedometer method have been found to be in good agreement for factors of safety greater than three (Burland, 1969; Ohta & Hata, 1973).

When the critical state model is applied with a conventional type of settlement analysis, a previous knowledge of the stresses is required. With the introduction of the FE method, this separation of steps is no longer required, and the critical state model has been employed as a constitutive relationship in such analyses. Formulations of this kind have been made, for example, by Zienkiewicz & Naylor (1971) and Ohta et al (1975). Wroth & Simpson (1972) and Wroth (1977) have successfully used the model to estimate the deformations and pore pressures beneath embankments on soft clay.

7.3.8.3 Further developments of critical state models

A model which can be considered an extension of the elasto-plastic Cam-clay theory of Roscoe & Schofield (1963), but which can account for anisotropy, was developed by Japanese researchers at Kyoto University (Ohta & Hata, 1973; Ohta et al, 1975; Ohta & Wroth, 1976). This model was further improved by incorporating dilatancy (i.e. volume change under constant p') and rheological behaviour (i.e. effects of secondary compression) by Sekiguchi & Ohta (1977). A revised Sekiguchi-Ohta model has recently been presented in which the strain increments for an anisotropically consolidated clay subjected to a stress reorientation could be found (Ohta & Sekiguchi, 1979). Based on this model, and using a finite element technique, Ohta et al (1980) predicted the vertical and horizontal deformations beneath an embankment constructed on a soft ground with transverse surface reinforcement (in this case by steel strips). A fair agreement with the measured data was obtained.

7.3.9 Finite element method

The basis of the finite element method is the representation of a body or a structure by an assemblage called *finite elements*. These elements are interconnected at nodal points, and the solutions are obtained in terms of displacements at these nodal points and average stresses of the elements. The method formulates first a stiffness matrix for an individual element, making use of the constitutive (stress-strain) relationship of the material. Individual stiffness matrices are then assembled to a global stiffness matrix for the entire soil mass with specified boundaries. This stiffness matrix then relates the nodal displacements to the load vector. Such a procedure, therefore, can take into account various types of stress-strain behaviour, non-homogeneity, irregular geometry and complex boundary conditions, as well as time-dependent loading (e.g. simulation of construction sequence).

Among the material models which describe stress-strain behaviour, the linear elastic one is of the simplest form, and this gives useful results when the elastic parameters are evaluated over the appropriate stress range (see Section 7.3.6). Other models commonly used are: (1) the bilinear elastic model (D'Appolonia & Lambe, 1970), (2) the elasto-plastic model (Davis & Booker, 1973), (3) hyperbolic relationships (Kondner, 1963; Duncan & Chang, 1970), and (4) the critical state model (see Section 7.3.8).

Except for the linear elastic model, all other models need the shear strength of the soil, and they also need to know the effects of the initial geostatic stresses. Thus, estimates of the in-situ K_o value must be made. In order to obtain the initial settlement, the FE analysis is carried out assuming completely undrained conditions and using undrained stress-strain deformation properties. The final settlement is obtained from an analysis employing drained deformation properties and effective stresses.

7.3.10 Probabilistic methods of settlement prediction

Failures to accurately predict magnitudes of settlement, even when using advanced methods, are partly caused by the variability of the soil parameters inherent in all natural soil deposits. By employing techniques of probabilistic analysis, the engineer can, together with his own judgement and experience, at least make a statement on the reliability of a prediction. A probabilistic approach can provide a systematic insight into the range of uncertainty which may be expected for a particular type of settlement problem.

Uncertainties in settlement analysis arise from the three sources: (1) variability of the soil parameters, (2) magnitude and distribution of imposed loads, and (3) uncertainties in the assumptions made in the mathematical model describing stress distribution and deformation. In most probabilistic procedures (Elnaggar & Krizek, 1970; Kay & Krizek, 1971; Corotis et al, 1975; Krizek et al, 1977), quantities representing compressibility and applied loads are considered as random variables (or functions of random variables), which have a certain probability distribution (e.g. normal, log normal, etc) derived from the available data. These random variables are then introduced into a deterministic settlement formula, and the distribution of the settlement variables is obtained. In the following, a method described by Sivandran (1979) and Balasubramaniam et al (1979) following a similar approach will be given.

Equation 7.4 may be written in the form:

$$R = K_1 L_1 + K_2 L_2 \tag{7.31a}$$

where R is the settlement ratio ($R = \rho_T/\Delta z$), and:

$$\left.\begin{aligned} K_1 &= \frac{C_r}{1+e_o}; \quad L_1 = \log \frac{\sigma'_{vo} + \Delta\sigma'_v}{\sigma'_{vo}} \\ K_2 &= 0 \quad ; \quad L_2 = 0 \end{aligned}\right\} \text{for } (\sigma'_{vo} + \Delta\sigma'_v) < \sigma'_{vc} \tag{7.31b}$$

$$\left.\begin{aligned} K_1 &= \frac{C_r}{1+e_c}; \quad L_1 = \log\frac{\sigma'_{vc}}{\sigma'_{vo}} \\ K_2 &= \frac{C_c}{1+e_c}; \quad L_2 = \log\frac{\sigma'_{vo}+\Delta\sigma'_v}{\sigma'_{vc}} \end{aligned}\right\} \text{for } (\sigma'_{vo}+\Delta\sigma'_v) > \sigma'_{vc} \tag{7.31c}$$

and e_o and e_c are the void ratios as shown in Fig. 7.4.

The settlement ratio, R, is a function of the compressibility variables K_1 & K_2 and of the variables L_1 & L_2 expressing the applied loads. These variables are in turn functions of the independent variables C_r and/or C_c, e_o and/or e_c, σ'_{vo}, σ'_{vc} and $\Delta\sigma'_v$. Provided the settlement model is correct, uncertainties in the magnitude of the computed settlements will arise from the randomness of these parameters. Probability distributions for all variables listed above can be derived from experimental data, i.e. laboratory and/or field tests, provided these are available in sufficiently large numbers. These probability distributions are then used in Eq. 7.31 to obtain the probability distribution, $f_R(R)$, for the settlement ratio R.

For a given relationship:

$$R = F(x_1, x_2, \ldots x_n) \tag{7.32}$$

where $x_1, x_2 \ldots \ldots x_n$ are random variables with probability distribution functions $f_1(x_1), f_2(x_2), \ldots f_n(x_n)$, it is possible to determine the associated probability distribution for R as $f_R(R)$ in a closed form (=method of derived distribution). These closed forms can be solved completely only for specific functions of $F, f_1, f_2, \ldots f_n$, but it is possible, by using a numerical technique such as the Monte Carlo method (Warner & Kabaila, 1968), to produce effective solutions to most problems.

By means of the Monte Carlo method, random numbers can be generated on a computer, and it is possible to sample from any specified distribution for each of the parameters in Eq. 7.31. If a set of such sampled variables is substituted into Eq. 7.31, a value for R is obtained. Repetition of this simulation procedure a large number of times will then yield the associated probability distribution $f_R(R)$ for the settlement ratio R. The function $f_R(R)$ is unique for a given set of parameter distribution functions. The accuracy to which it can be determined depends on the number of simulations. Balasubramaniam et al (1979) found that 1000 simulations were necessary to obtain an acceptable function for $f_R(R)$.

7.3.11 Methods using in-situ tests

7.3.11.1 Settlement estimation from pressuremeter tests

The pressuremeter test produces a volume, V, (or hole diameter) versus pressure, p^*, relationship from which various parameters can be derived, such

as the pressuremeter modulus, E_M, (which is the slope ($\Delta V/\Delta p^*$) of the linear portion of the pressuremeter curve) and the yield or creep pressure, p_y^*, (which is the pressure at the end of the linear range). The information obtained from this test has been used to predict the settlement of foundations (see, for example, Ménard & Rousseau, 1962; Ménard, 1965, 1971, 1975). Ménard based the development of his settlement formula on the assumption that settlement was caused mainly by the soil contained within a semi-spherical zone which has the same diameter as the foundation. The derivation makes use of the theory of elasticity, and is therefore appropriate only for loading cases in which the preconsolidation pressure, σ'_{vc}, is not exceeded. There is a similarity between the e—log σ'_v curve from an oedometer test for pressures below σ'_{vc} and the pressuremeter curve for pressures below p_y^*, in that they both give an approximately linear relationship. A good correlation between p_y^*, and σ'_{vc} has been found by Mori & Tajima (1964) and Lukas & LeClerc de Bussy (1976). Mori (1965) concluded that, in fact, σ'_{vc} corresponds to p_y^* if p_y^* is determined in terms of effective stress. (Comparison arises also with the yield stress, p'_{oy} defined in Section 7.3.8.1).

According to Menard (1965, 1971, 1975), the settlement, ρ, of a foundation of half-width b, exerting a uniform net pressure q, can be expressed as:

$$\rho = \frac{1+\upsilon}{3E_M} q\, b_o \left(\lambda_2 \frac{b}{b_o}\right)^{\alpha} + \frac{\alpha\, \lambda_3}{4.5E_M} q\, b \tag{7.33}$$

where E_M is the pressuremeter modulus, υ is Poisson's ratio (taken as $\frac{1}{3}$), b_o is a reference half-width equal to 300mm, λ_2 & λ_3 are empirical coefficients that depend on the shape of the foundation, and α is an empirical coefficient which depends on the type and behaviour of the soil and which accounts for the non-elastic characteristics of the soil. The value of α is 1.0 for peat, $\frac{2}{3}$ for clay, $\frac{1}{2}$ for silt and $\frac{1}{3}$ for sand and gravel.

Equation 7.33 consists of two components; the first represents the deviatoric contribution, and the second the isotropic or spherical compression portion equivalent to consolidation settlement. The equation was experimentally calibrated by numerous full-scale loading tests on foundations of various shapes and in different types of soil. For layered soils, procedures are available to compute an equivalent modulus (Ménard, 1975, Baguelin et al, 1978). Since the pressuremeter modulus is only valid in the range $p^* < p_y^*$ or $(\sigma'_{vo} + \Delta\sigma_v) < \sigma'_{vc}$, which corresponds about to a factor of safety > 3, Eq. 7.33 should be applied only to cases which fall into this range. The method has been used extensively in France, and satisfactory results have been obtained (Ménard, 1971, Bru et al, 1973).

For isolated foundations, predictions made from pressuremeter tests appear to be much superior to those made from oedometer tests. For successful predictions, however, it is required that the equipment and procedures for the

pressuremeter test are identical to those used by Ménard. It is obvious that the best results are obtained for cases in which the foundations and soil types are similar to those for which Eq. 7.33 was developed, and this would exclude embankments on shallow very soft soil deposits (Tavenas, 1975b). French engineers (Amar et al, 1977), however, have also reported case histories of embankments and rafts on very soft soils in which the pressuremeter gave satisfactory results. In Fig. 7.11, some comparisons between predicted and observed settlements of embankments and rafts are shown, both for the pressuremeter and the oedometer method. Only in one case does the oedometer method give a better prediction.

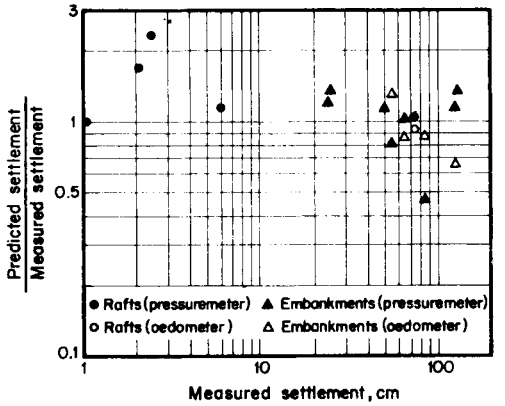


Fig. 7.11. Comparison between measured and predicted settlements of rafts and embankments using the results of oedometer and pressuremeter tests (Amar et al, 1977).

7.3.11.2 Predictions from static cone penetration tests

Although the cone penetration test is a procedure which measures strength rather than deformation, numerous relationships have been proposed between the static cone resistance, q_c, and compressibility. A good summary of available formulae was given by Mitchell & Gardner (1975), but most of these are applicable to cohesionless soils rather than to soft clay. All methods for soft clay relate the constrained modulus, M_D, which is the inverse of the compressibility, m_v, to the cone resistance, q_c, by:

$$M_D = \frac{1}{m_v} = \alpha_c \, q_c \tag{7.34}$$

This method was originally proposed by Buisman in 1940 (Bachelier & Parez, 1965) and subsequently pursued by Sanglerat at the Ecole Centrale de Lyon.

In the normally consolidated range, the parameter α_c is related to C_c by (Mitchell & Gardner, 1975):

$$\alpha_c = \frac{2.3\ (1+e_o)}{C_c}\ \frac{\sigma'_{vo}}{q_c} \tag{7.35}$$

For overconsolidated soils σ'_{vo} is replaced by σ'_{vc}, and e_o by e_c. Settlements can be computed from (see Eq. 7.2):

$$\rho_T = \sum_{i=1}^{n} (m_v \,.\, \Delta\sigma'_v \,.\, \Delta z)_i = \sum_{i=1}^{n} \left(\frac{\Delta\sigma'_v\ \Delta z}{\alpha_c\, q_c}\right)_i \tag{7.36}$$

For α_c the following values have been proposed:

(1) Bachelier & Parez (1965):

$3.3 < \alpha_c < 5.75$, for normally consolidated clays

(2) Sanglerat (1972) and Sanglerat et al (1972):

$3 < \alpha_c < 8$, for low plasticity clays (CL) and $q_c < 700$ kPa

$2 < \alpha_c < 6$, for highly plastic clays and loams (MH, OH) and $q_c < 2000$ kPa

$$\left.\begin{array}{r} 1.5 < \alpha_c < 4 \\ 1 < \alpha_c < 1.5 \\ 0.4 < \alpha_c < 1 \end{array}\right\} \begin{array}{l}\text{for peat and highly} \\ \text{organic clays } (OH) \\ \text{and } q_c < 700 \text{ kPa}\end{array} \left\{\begin{array}{r} 50 < w_n < 100 \\ 100 < w_n < 200 \\ w_n > 200 \end{array}\right.$$

Correlations given by Gielly et al (1969) are of similar magnitude.

Case histories showing successful predictions by this method have been reported in the book by Sanglerat (1972), which also gives many more details about the method. Sanglerat et al (1977) applied the method to a fill over soft clay. Meigh & Corbett (1969) showed that, within a given deposit of soft clay, the relationship between M_D and q_c can be remarkably consistent. In spite of these successful applications reported in the literature, however, the method can only be considered as a rough estimate for settlements .

7.4 METHODS FOR PREDICTING RATE OF ONE-DIMENSIONAL CONSOLIDATION SETTLEMENT

7.4.1 Classification of approaches to analyze consolidation problems

For the prediction of the *magnitude* of consolidation settlement, only a knowledge of the vertical strains is required, which in the case of one-dimensional strain does not need to rely on any particular theory. It is only necessary to assess the change in void ratio for a particular applied stress from a laboratory determined stress-strain relationship. The *rate* of consolidation settlement, on the other hand, involves the development of effective stress in the soil as water is squeezed out. In order to predict the rate at which this occurs,

it is necessary to establish a material model describing the flow of water through the soil skeleton (e.g. Darcy's law), the relationship between permeability and void ratio, and the soil skeleton stress-strain relationship. The separation of a settlement analysis into magnitude and rate of settlement is in a way an artificial one, but this has to be made in one-dimensional analysis where there is only a single vertical component of displacement. In fact, the two phenomena are inherently coupled, and in a truly three-dimensional theory this interrelationship is automatically taken care of (Schiffman et al, 1969).

Since Terzaghi formulated his consolidation theory in 1923, many extensions have been made to account for most of the rather unrealistic assumptions made in the original theory. Today, consolidation theory has reached an advanced stage of development, and solutions are now available for most practical problems, either in closed analytical form (and presented in the form of convenient graphs), or by numerical techniques in the form of general-purpose computer programs.

On the basis of the extensions which have been made to the original Terzaghi theory, a classification of presently available solutions can be made with respect to the following characteristics:

(1) Flow and deformation conditions: one-dimensional, two-dimensional (plane strain) or three-dimensional.
(2) Loading conditions: constant loading, construction type loading or arbitrary loading.
(3) Soil profile: homogeneous or inhomogeneous, isotropic or anisotropic, single layer or multi-layer.
(4) Soil skeleton stress-strain relationship: linear or nonlinear and with or without creep, and formulated either in terms of stresses or strains.
(5) Variation of soil parameters relevant to consolidation (e.g. permeability, coefficient of consolidation, compressibility) with void ratio or effective stress.
(6) Magnitude of deformation of consolidating layer compared to its initial thickness: infinitesimal strain or finite strain theories.
(7) Compressibility of pore fluid: incompressible or compressible.
(8) Method of solution: analytical closed-form solutions or numerical solutions by, for example, finite difference or finite element techniques.

In the following sections, Terzaghi's theory and its assumptions are first briefly recalled, and then selected examples of practical interest are given of improvements which have been made to the original theory. Some developments have only refined Terzaghi's theory by solving a more generalized form of the differential equation of consolidation, while others have established entirely new governing consolidation equations based on realistic stress-strain-time models to yield compression-time and pore pressure-time relationships.

7.4.2 Terzaghi's theory and its deficiencies

The classical one-dimensional Terzaghi theory is based on the assumptions (Barden, 1968; Tavenas et al, 1979): (1) one-dimensional deformation of clay layer, (2) one-dimensional drainage following Darcy's law for any hydraulic gradient, (3) homogeneous soil which is fully saturated, (4) incompressibility of soil grains and pore fluid, (5) constant compressibility and permeability, (6) linear relationship between effective stress and void ratio (volume) of soil, (7) infinitesimal one-dimensional strains and flow velocities, and (8) no structural viscosity or secondary compression of the soil, i.e. the time lag of compression is caused only by the low permeability of the clay.

By combining the relationship for the continuity of fluid flow with the stress-strain (effective stress-void ratio) law of the soil skeleton and assuming constant total stress, Terzaghi's well-known consolidation equation can be derived (see for example Lambe & Whitman, 1969). It has the form:

$$\frac{\partial u_e}{\partial t} = c_v \frac{\partial u_e^2}{\partial z^2} \tag{7.37}$$

where u_e is the excess pore pressure, z is the space coordinate starting at the top of the consolidating layer, t is the time, and c_v is the coefficient of consolidation. The solution of this diffusion type equation can be written as a Fourier series:

$$u_e(z,t) = \frac{4}{\pi} \sum_{m=0}^{\infty} \frac{1}{2m+1} \sin\left[\frac{(2m+1)\pi}{2h} z\right] \exp\left[-(2m+1)^2 \pi^2 \frac{T_v}{4}\right] \tag{7.38}$$

where h is the length of drainage path, and T_v is a dimensionless time factor given by:

$$T_v = \frac{c_v t}{h^2} \tag{7.39}$$

The degree of consolidation settlement, U_s, which in this case is identical to the degree of pore pressure dissipation, U_p, is given by:

$$U_s = \frac{\rho_c(t)}{\rho_c} = 1 - \sum_{m=0}^{\infty} \frac{8}{(2m+1)^2 \pi^2} \exp\left[-(2m+1)^2 \pi^2 \frac{T_v}{4}\right] \tag{7.40}$$

where $\rho_c(t)$ is the settlement at time t, and ρ_c is the final consolidation settlement ($\rho_c = \rho_c(t=\infty)$). Equation 7.40 follows directly from:

$$U_s = U_p = 1 - \frac{\int u_e \, dz}{\int u_{eo} \, dz} \tag{7.41}$$

where u_{eo} is the initial excess pore pressure distribution.

Solutions to consolidation problems are commonly presented in the form of graphs of U_s (or U_p) versus a time factor T_v (which is not necessarily identical with Eq. 7.39). From such graphs, the settlement at any time t can then conveniently be obtained from:

$$\rho_c(t) = U_s \rho_c \tag{7.42}$$

Most of Terzaghi's assumptions are unrealistic or are not satisfied in practical applications. Thus, real loadings are time-dependent, soils are inhomogeneous (e.g. layered), soil properties vary nonlinearly with respect to effective stress, and strains are occasionally large. In spite of this, the original Terzaghi theory is widely used because it is simple to apply and because uncertainties in the results more commonly result from ill-defined stratigraphy, soil properties and loading conditions than from simplifications in the analysis.

In laboratory tests, deviations from the Terzaghi theory have long been recognized. Typically, these are (Newland & Allely, 1960; Leonards & Girault, 1961; Barden, 1969; Hawley & Borin, 1973):

(1) Settlements do not approach an ultimate value (i.e. they exhibit the phenomenon of secondary compression).
(2) The ratio of secondary to primary settlement increases with decreasing load increment ratio and also with increasing maximum drainage distance.
(3) The rate of dissipation of mid-plane pore pressure increases with a reduction in load increment ratio during the early stages of the consolidation process.
(4) Prolonged load increment duration causes an increase in the ratio of secondary to primary settlement and an initially rapid dissipation of mid-plane pore pressure in the succeeding load increment.

If, however, some of the unrealistic assumptions can be removed from the classical analysis, then acceptable predictions can often be achieved (Olson & Ladd, 1979).

7.4.3 Improved models of one-dimensional consolidation

7.4.3.1 Time-dependent loading

An important practical case is construction-type loading. For example, for an embankment, the load application is first linear with time and then remains constant; this may be repeated when further fill stages are added. Convenient solutions in graph form have been produced, for example, by Schiffman (1958) and recently by Olson (1977). Olson retained all the assumptions of Terzaghi's basic theory (Section 7.4.2) except instantaneous loading; Fig. 7.12 shows curves of time factor, T_v, versus degree of consolidation for different construction time factors, T_c. The time factors are defined as:

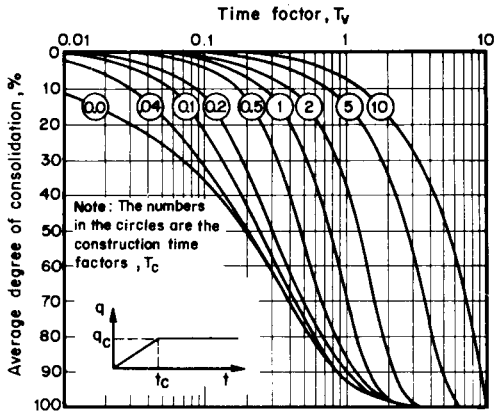


Fig. 7.12. Degree of consolidation versus time factor curves for one-dimensional vertical flow and single ramp loading (Olson, 1977).

$$T_v = \frac{c_v t}{h^2} \quad ; \quad T_c = \frac{c_v t_c}{h^2} \tag{7.43}$$

where t is the time since load application, t_c is the construction time, and h is the drainage distance.

An example of how to use the graph in Fig. 7.12 is given in Table 7.2. The following data were used: consolidation of a 3 m thick, doubly-drained layer with c_v=0.0045 m^2/day and an ultimate settlement, ρ_c=254 mm, subjected to a single ramp loading with a construction time of 30 days. This gives a construction time factor, T_c, of 0.060. Interpolation between the curves in Fig. 7.12 gives a degree of consolidation after, for example, 100 days (T_v=0.200) of 47%. If now a second ramp loading of 6 m is added, say between days 50 and 100, then according to Olson, the combined solution can be obtained by superposition of each loading step. Table 7.2 shows all the calculations and also includes a step of instantaneous unloading of 3 m of fill at day 200. The construction time factors, T_c (0.060, 0.100 & 0.000) and the

Table 7.2. Example of consolidation under construction loading (Olson, 1977)

First ramp loading (T_c=0.060; ρ_c=254 mm)				Second ramp loading (T_c=0.100; ρ_c=508 mm)				Instantaneous unloading (T_c=0.000, ρ_c= −254 mm)				
t_1 (days)	T_1	U_1 (%)	$\rho_{c1}(t)$ (mm)	t_2 (days)	T_2	U_2 (%)	$\rho_{c2}(t)$ (mm)	t_3 (days)	T_3	U_3 (%)	$\rho_{c3}(t)$ (mm)	$\rho_{ctot}(t)$ (mm)
10	0.020	4	10.2									10.2
30	0.060	17	43.2									43.2
50	0.100	27	68.6	0	0	0	0					68.6
100	0.200	47	119.4	50	0.100	21	106.7					226.1
200	0.400	68	172.7	150	0.300	57	289.6	0	0	0	0	462.3
400	0.800	89	226.1	350	0.700	84	426.7	200	0.400	70	−177.8	475.0

ultimate settlements, ρ_c (254, 508 &-254 mm) are also indicated. These calculations can also be carried out by means of a table for T_v and T_c published by Schiffman (1960).

7.4.3.2 Nonlinear stress-strain relationships

Terzaghi formulated a linear stress-strain relationship as:

$$\frac{de}{d\sigma'_v} = -a_v = -\frac{0.434\,C_c}{\sigma'_v} \tag{7.44}$$

where a_v is the coefficient of compressibility. More realistic nonlinear relationships have been employed by Davis & Raymond (1965), Gibson et al (1967), Hawley & Borin (1973) and Mesri & Rokhsar (1974), usually expressing the void ratio as a function of the logarithm of effective stress, such as:

$$e_o - e = -C_c \log \frac{\sigma'_v}{\sigma'_{vo}} \tag{7.45}$$

However, only by means of numerical methods can the actual e—log σ'_v curve be directly incorporated into a solution.

Olson & Ladd (1979) investigated the sensitivity of time-settlement curves to various nonlinear e—log σ'_v relationships. They applied a constant stress of 240 kPa to a 1.5 m thick doubly-draining layer with constant c_v of 0.0046 m^2/day and an average initial effective stress of 135 kPa. Three different types of e—log σ'_v curve were used, as shown in Fig. 7.13, namely: (1) constant coefficient of compressibility, a_v (corresponding to Terzaghi's classical theory), (2) constant compression index, C_c, and (3) a relationship corresponding to an overconsolidated clay with a preconsolidation pressure of 290 kPa. The resulting time-settlement curves are shown in Fig. 7.13, and these indicate that the overconsolidated clay significantly retards the settlement process.

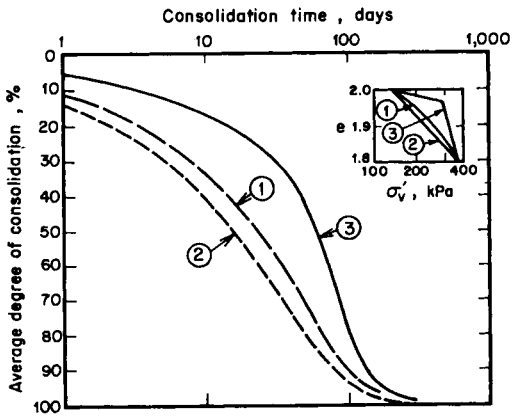


Fig. 7.13. Time-settlement curves for three cases involving identical soil properties except for shape of compression curve (Olson & Ladd, 1979).

While in the linear Terzaghi theory the degree of consolidation settlement is identical with the average degree of pore pressure dissipation, this is no longer the case when nonlinear stress-strain relationships are involved. For example, with the nonlinear stress-strain law adopted by Davis & Raymond (1965), the degree of settlement coincides with Terzaghi's theory, whereas the degree of maximum pore pressure dissipation is a function of the load increment ratio.

7.4.3.3 Variable soil parameters

Closed-form solutions with variable permeability and/or compressibility are restricted to analytically simple variations of these parameters. Schiffman & Gibson (1964) presented consolidation time-settlement curves where either the coefficient of permeability, k, or the coefficient of volume compressibility, m_v, had a polynomial variation with depth. Another frequently employed expression is the linear logarithm of coefficient of permeability versus void ratio relationship (Poskitt, 1969; Hawley & Borin, 1973; Mesri & Rokhsar, 1974) which can be expressed as:

$$\Delta e = C_k \, \Delta \log k \tag{7.46}$$

where C_k is a permeability index.

Poskitt (1969) used a perturbation method for the solution of the governing equations of his one-dimensional strain theory, and showed that it contained the theory of Davis & Raymond (1965) as a special case. Poskitt's theory can take into account not only variable compressibility and permeability but also finite strains. Creep effects, however, are ignored. The perturbation solution clearly demonstrates the departures from Terzaghi's theory in the pore pressure and settlement versus time relationships as a result of the variation in compressibility and permeability. Poskitt further showed that the well-known h^2 scaling law (h=drainage path length) still applied, inspite of the nonlinearity of the model.

More realistic solutions can be obtained by direct numerical techniques, but generalized degree of consolidation settlement versus time factor curves are no longer possible. Olson & Ladd (1979) illustrated the effect of a variable coefficient of consolidation, c_v. The variable c_v solution (c_v varying as a function of effective stress) properly accounts for the formation of a somewhat less pervious skin near the drainage boundaries during the earlier stages of consolidation. The skin develops as a result of higher effective stresses and thus decreased void ratio and permeability near the drainage boundaries. This leads to a retardation of the consolidation process.

7.4.3.4 Validity of Darcy's law

A theory considering a nonlinear variation of rate of pore water flow, v, with hydraulic gradient, i, in the form:

$$v = k\, i^n \tag{7.47}$$

was developed by Hansbo (1960) and applied to sand drain problems. Hansbo observed that, at small gradients, an exponent $n > 1$ better characterized experimental data obtained from an undisturbed Swedish soft clay, while the original Darcy equation with $n=1$ fitted the data well when the hydraulic gradient had exceeded a certain definite limit. The deviations were explained by the presence of mobile soil particles which clog the flow channels. Loosely connected particles in the soil fabric could in fact be verified by electron photomicrographs (Hansbo, 1973). Another factor believed to account for the deviations in Darcy type flow, namely non-Newtonian liquid behaviour, is now considered unlikely, as many of the effects originally interpreted as unusual water characteristics can be ascribed to experimental errors (Olsen, 1965; Mitchell, 1976).

Elnaggar et al (1973) introduced a four-parameter flow velocity-gradient relationship which, by proper choice of the parameters, is capable of characterizing much of the published experimental data. They found, for example, that with their improved theory, the time rate of excess pore pressure dissipation is considerably less than from the conventional Terzaghi theory. Their results showed further that the classical similitude (h^2 scaling law) was no longer satisfied, with a thin layer consolidating much faster than a thick one. They recommended, therefore, that corrections be made to laboratory test data for thickness, in addition to the extrapolation made by means of the h^2 law, before they are used for field predictions.

7.4.3.5 Finite strains

In the conventional Terzaghi theory, strains are assumed to be small or, in a mathematical sense, infinitesimal. This means that the drainage distance, h, remains constant during consolidation. While such an assumption is justified for buildings where only small settlements can be tolerated, it might lead to considerable errors in the case of embankments on highly compressible soils.

Recently, various publications on the subject of finite strain consolidation have appeared (Gibson et al, 1967; Mesri & Rokhsar, 1974; De Simone & Viggiani, 1976; Monte & Krizek, 1976; Krause, 1977; Lee & Sills, 1979). The formulation of a finite strain consolidation theory requires an equation of equilibrium for the soil mass and the fluid flow, and a continuity balance relationship (Gibson et al, 1967). Schiffman (1980) demonstrated that linear and nonlinear infinitesimal theories, e.g. those given by Schiffman & Gibson (1964) or by Davis & Raymond (1965), are special cases of the finite strain theory.

Olson & Ladd (1979) used finite difference techniques to illustrate the effects of large and nonuniform strains. A highly compressible soil layer, linearly elastic and with a constant coefficient of consolidation, was subjected to instantly applied loads which would produce ultimate settlements of 1%,

10%, 50% and 80% of the original layer thickness. In Fig. 7.14a, the time-settlement curves computed by the finite difference technique and taking into account the changing layer thickness are compared with the curve obtained from the exact (Fourier-series) solution (Eq. 7.40) for small strains. For ultimate settlements of less than 10%, the influence of finite strains is negligibly small for practical purposes.

An improvement proposed by Olson & Ladd (1979) to the classical small-strain theory can be made by defining a constant average value of h as:

$$h = h_o - \frac{\rho_T}{2N_D} \tag{7.48}$$

where N_D is the number of drainage boundaries, and ρ_T is the ultimate settlement. Figure 7.14b shows two time-settlement curves, one calculated from the exact Terzaghi solution but using a constant h according to Eq. 7.48, and the other from finite difference analysis for the case of a layer with double drainage and ρ_T=50% of the original layer thickness. Olson & Ladd (1979) stated further that other solutions indicate that Eq. 7.48 yields degrees of consolidation within 5% if $\rho_T/(2h)$ is less than 46%.

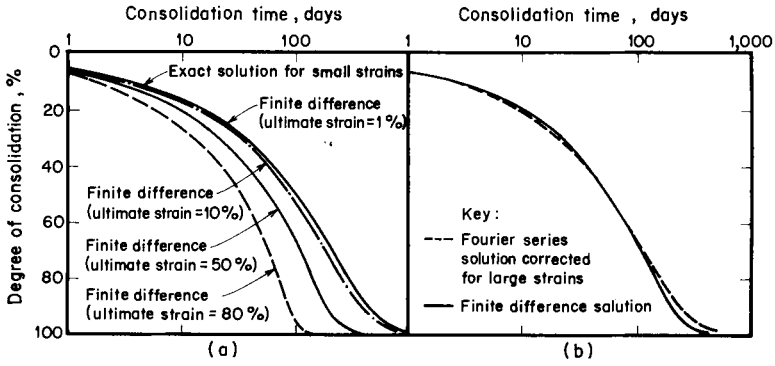


Fig. 7.14. (a) Effects of large strains on time-settlement curve, (b) comparison of finite difference time-settlement curve and curve calculated using Fourier series with simple correction for large strain (Eq. 7.48) (for this analysis ultimate strain was 50%) (Olson & Ladd, 1979).

In agreement with Fig. 7.14a, slower rates of consolidation when assuming small strains were also predicted with a one-dimensional consolidation model developed by Mesri & Rokhsar (1974) (Section 7.4.3.8).

7.4.3.6 Layered systems

Attempts to model layered soil systems by extending the classical Terzaghi theory have been made by Gray (1945), Horne (1964) and Schiffman & Stein (1970). These solutions, however, are complicated and are seldom utilized in

design. Direct numerical techniques which are more flexible are usually preferred. Palmer (c.f. Olson & Ladd, 1979) proposed that a layered system could be replaced by an 'equivalent' homogeneous layer by the following procedure. The c_v value of any layer, s, in the system is denoted by c_{vs}. Then, the equivalent layer thickness, h'_n, of any other layer n ($n \neq s$) of original thickness h_n is found from the transformation:

$$h'_n = h_n \left(\frac{c_{vs}}{c_{vn}}\right) \tag{7.49}$$

7.4.3.7 Submergence correction

When a fill settles on ground with a high water-table, a part of the fill may become immersed. A submergence correction can be applied to a settlement analysis to take into account the reduced total stress and the associated rebound of consolidated soil close to a drainage boundary. Olson & Ladd (1979) investigated the effect of submergence on the time-settlement relationship using a finite difference procedure for two embankments on a 6 m thick clay layer with double drainage and constant c_v. Figures 7.15a & 7.15b show the results for 0.3 m and 6 m high fills respectively. In each case, two analyses were performed, one with and the other without submergence correction, but both taking into account large finite strains and a nonlinear e—log σ'_v

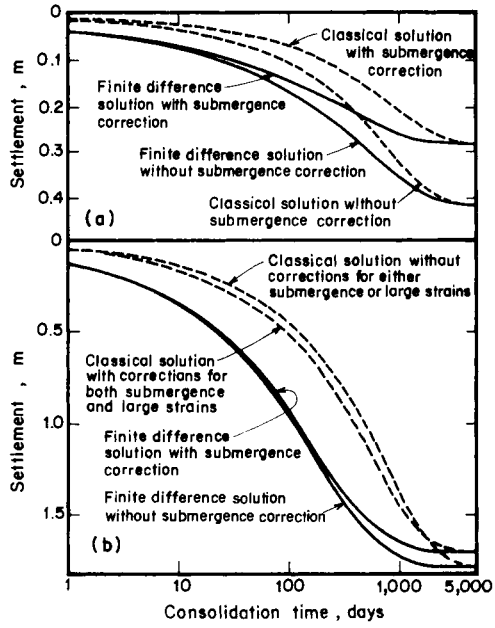


Fig. 7.15. Time-settlement curves for embankment on soft clay: (a) embankment height 0.3 m, (b) embankment height 6 m (Olson & Ladd, 1979).

relationship. For comparison, two conventional-type Terzaghi analyses are also shown, one with and the other without corrections for submergence and large strains (according to Eq. 7.48). It can be seen that, for the higher fill, the submergence correction relative to the total settlement is much smaller than for the lower fill. Thus, submergence corrections are of practical significance for low fills on highly compressible soils.

7.4.3.8 Time-dependent behaviour of soil skeleton

In order to account better for the deviations in the Terzaghi theory observed in laboratory consolidation tests (see Section 7.4.2), some more advanced one-dimensional theories have extended the soil skeleton stress-strain relationship to include time effects, e.g. strain rate, $\partial e/\partial t$. Several such models are now in existence (Garlanger, 1972; Hawley & Borin, 1973; Mesri & Rokhsar, 1974; Sekiguchi & Toriihara, 1976; Magnan et al, 1979a), and most of them are based on soil parameters which can readily be obtained from conventional oedometer tests. All these models, however, require a numerical treatment of the equations governing the consolidation process.

One of the first practical approaches, and one which has received considerable attention, was made by Garlanger (1972) who used Bjerrum's concept of Fig. 7.9. Bjerrum's model (Section 7.3.7) considers separate expressions for consolidation settlement and secondary compression. Garlanger's procedure, however, is able to take into account the combined effect of primary consolidation and secondary compression. The family of curves in Fig. 7.9 can mathematically be expressed as:

$$\frac{e}{e_o} = \left(\frac{\sigma'_{vc}}{\sigma'_{vo}}\right)^{-\bar{a}} \cdot \left(\frac{\sigma'_{vf}}{\sigma'_{vc}}\right)^{-\bar{b}} \cdot \left(\frac{t_i + t}{t_i}\right)^{-\bar{c}} \tag{7.50}$$

where $\bar{a}$ is the slope of the log e vs log σ'_v curve from σ'_{vo} to σ'_{vc}, $\bar{b}$ is the slope of the log e vs log σ'_v curve for $\sigma'_v > \sigma'_{vc}$, $\bar{c}$ is the slope of the log e vs log t curve, t_i is the time of the instant line (e.g. 24 hrs), e_o is the in-situ void ratio, and the other quantities are as defined previously. The continuity equation for the one-dimensional consolidation of a saturated clay layer can be written in the form:

$$\frac{\partial e}{\partial t} = \frac{k\,(1+e_o)}{\gamma_w}\,\frac{\partial^2 u_e}{\partial z^2} \tag{7.51}$$

where k is the permeability, γ_w is the unit weight of water, and u_e is the excess pore pressure. Garlanger further assumed (as in the Terzaghi theory) the validity of the effective stress equation and constant total stress:

$$\frac{\partial \sigma'_v}{\partial t} = -\frac{\partial u_e}{\partial t} \tag{7.52}$$

Finally, if the void ratio of an element is now a function of both effective stress and time, the total rate of change in void ratio, de/dt can be written as:

$$\frac{de}{dt} = \left(\frac{\partial e}{\partial \sigma_v'}\right)_t \frac{d\sigma_v'}{dt} + \left(\frac{\partial e}{\partial t}\right)_{\sigma_v'} \tag{7.53}$$

The terms $(\partial e/\partial \sigma_v')_t$ and $(\partial e/\partial t)_{\sigma_v'}$ can be obtained by differentiation of Eq. 7.50. Equations 7.51, 7.52, & 7.53 can then be solved simultaneously for given boundary conditions to yield the e—t and u_e—t relationships at every point in the consolidating layer. Integration of the e—t—z relationships gives the average degree of consolidation, or the settlement of the soil layer at any time. Garlanger (1972) employed a finite difference numerical procedure to solve this problem.

Along similar lines, Mesri & Rokhsar (1974) developed a more generalized theory which considers finite strain, variable compressibility and permeability during the consolidation process, as well as the effects of a preconsolidation pressure and secondary compression. The stress-strain-time law was also based on the Bjerrum concept (Fig. 7.9). It was solved together with a continuity equation based on Darcy's law and a permeability-void ratio relationship according to Eq. 7.46.

The dimensionless excess pore water pressure and void ratio versus time relationships were presented by Mesri & Rokhsar, with $\sigma_{vf}'/\sigma_{vo}'$, $\sigma_{vc}'/\sigma_{vo}'$, C_k/C_c, C_r/C_c and C_α/C_c as parameters. (For a definition of these quantities see Fig. 7.4, Eq. 7.46 and Eq. 7.6). Examples of these results are given in Figs. 7.16 & 7.17. In Fig. 7.16, the effect of the load increment ratio on a young clay ($\sigma_{vc}'=\sigma_{vo}'$) with secondary compression is illustrated, and it can be seen that, when C_α/C_c is relatively large and $\sigma_{vf}'/\sigma_{vo}'$ is small, the standard S-shape curve is no longer obtained, which is in agreement with observations from laboratory tests (Leonard & Girault, 1961). In Fig. 7.17, the response of an aged clay ($\sigma_{vc}' > \sigma_{vo}'$) with secondary compression to a variation in σ_{vc}', expressed as the ratio $R_c=(\sigma_{vc}'-\sigma_{vo}')/(\sigma_{vf}'-\sigma_{vo}')$, is presented. It can be noticed that, for a given C_r/C_c and $\sigma_{vf}'/\sigma_{vo}'$, the variation in R_c significantly influences the rate of pore pressure dissipation as well as the rate of compression.

The dimensionless quantities $\bar{\beta}$, μ_m and T in Figs. 7.16 & 7.17 are defined as:

$$\mu_m = \frac{u_{em}}{\Delta\sigma_v'} \tag{7.54a}$$

$$\bar{\beta} = \left(\frac{e_o - e}{e_o - e_f}\right)_{average} \tag{7.54b}$$

$$T = \frac{k_o(1+e_o)\,\sigma_{vo}'}{0.434\,\gamma_w\,C_c}\,\frac{t}{h_o^2} \tag{7.54c}$$

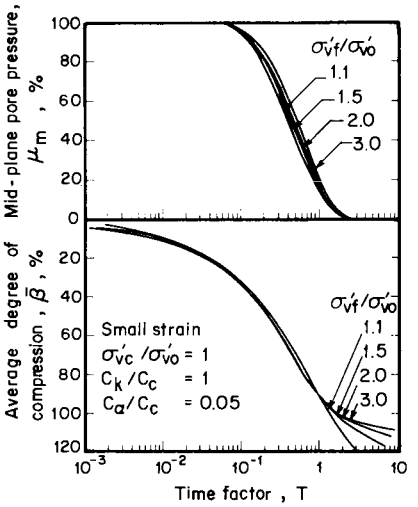


Fig. 7.16. Theoretical consolidation curves for young clays with secondary compression; $\sigma'_{vf}/\sigma'_{vo}$ variable (Mesri & Rokhsar, 1974).

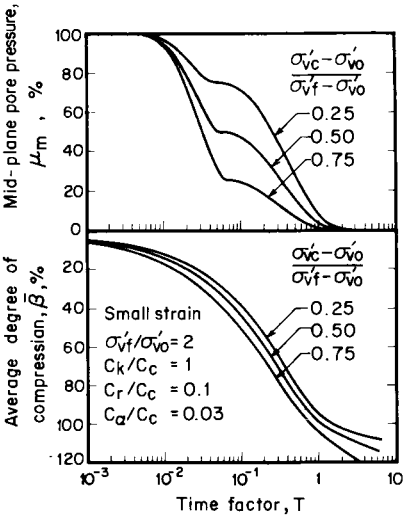


Fig. 7.17. Theoretical consolidation curves for overconsolidated or aged clays with secondary compression; critical pressure ratio, R_c, variable (Mesri & Rokhsar, 1974).

where u_{em} is the excess pore water pressure at the midplane, $\Delta\sigma'_v$ is the instantly applied pressure ($\sigma'_{vf} - \sigma'_{vo}$), e_f is the void ratio corresponding to σ'_{vf}, k_o is the initial permeability, and h_o is the initial drainage path length.

An elaborate one-dimensional consolidation model has also been developed by the Laboratoires des Ponts et Chaussées in France, in cooperation with Laval University in Quebec (Magnan et al, 1979a, 1979b; Tavenas et al, 1979). This model does not retain any of Terzaghi's original assumptions (Section 7.4.2), except for one-dimensional strain, and can in particular take into account layered soils, variations in e, c_v and k with σ'_v, secondary compression, unsaturated conditions, and compressible pore fluid. The model obeys a stress-strain-time law proposed by Bjerrum (1967, 1972, 1973) and Garlanger (1972) (Fig. 7.9), except that the constant-time lines are taken as equidistant, according to:

$$\Delta e = C_\alpha \, \Delta \log t \tag{7.55}$$

The permeability follows Darcy's law, with a variable vertical coefficient of permeability, k_v varying according to Eq. 7.46. The solution of the model is accomplished by means of a finite difference program, CONMULT *(Consolidation des Multicouches)*.

The application of the CONMULT model has permitted a systematic study of the influence of various factors (e.g. overconsolidation, creep) on the progress of pore pressure dissipation and settlement with time. Magnan et al

(1979a) demonstrated how creep retards the dissipation of pore pressures and increases the amplitude of settlement (i.e. it not only compensates for the decreased pore pressure dissipation but even accelerates the settlement of the soil layer). In agreement with Olson & Ladd (1977) (Section 7.4.3.3), Tavenas et al (1979) showed that the rapid consolidation close to drainage boundaries leads to the formation of a less pervious zone which retards the average consolidation and modifies the isochrones. They also showed that c_v varies enormously during consolidation of a normally consolidated clay, giving a slower settlement than when c_v is assumed to be constant.

The influence of preconsolidation can also be very important for cases where the load increment spans σ'_{vc}, as pointed out by Mesri & Choi (1979a). By substituting Eq. 7.52 into Eq. 7.53, the time rate of pore pressure dissipation for a time-independent void ratio, $(\partial e/\partial t)_{\sigma'_v}=0$, can be written as:

$$\frac{du_e}{dt} = \frac{de/dt}{\dfrac{0.434\ C_c}{\sigma'_v}} \tag{7.56}$$

This implies that the rate of excess pore pressure dissipation is expected to decrease when a load increment spans the preconsolidation pressure, because C_c increases suddenly when $\sigma'_v \geqslant \sigma'_{vc}$. Mesri & Choi (1979a) cited various field observations where the pore pressures after an initial sharp drop remained more or less stationary while settlement continued.

The calculations carried out with the CONMULT program by Tavenas et al (1979) revealed that the degree of settlement versus time relationship followed more or less the classical Terzaghi shape, while the degree of pore pressure dissipation versus time relationship exhibited a distinct flat section (Fig. 7.18), in keeping with the field observations of Mesri & Choi (1979a).

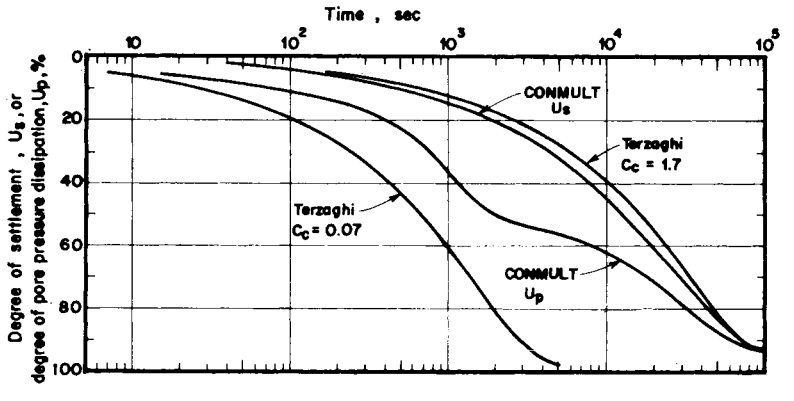


Fig. 7.18. Results of CONMULT program for a pressure increment spanning the preconsolidation pressure, and comparison with Terzaghi theory (Tavenas et al, 1979).

7.44. *Rheological models*

7.4.4.1 *Model concept*

The creep phenomenon and other deviations from the Terzaghi theory observed in the consolidation of clays have led to the development of rheological models. These are mechanical analogues of rheological constitutive behaviour, i.e. systems composed of elastic (springs), viscous (dashpots) and plastic (sliders) elements which are connected either in series or in parallel. The deformation characteristics of these elements can be either linear or nonlinear. These rheological models are helpful in giving a mechanical representation of postulated mechanisms of soil behaviour.

The first such rheological equation, although not given explicitly in the form of a mechanical analogue was presented in 1939 by Merchant working under the supervision of D.W. Taylor in an attempt to develop a mathematical theory for secondary compression. Merchant postulated the rate of secondary compression to be proportional to the amount of residual secondary compression. The mechanical analogue of the Taylor-Merchant model (Christie, 1964) is shown in Fig. 7.19a and consists of a linear spring in series with a Kelvin body whose spring and dashpot are also linear. The dashpot has the function of incorporating e as a function of σ_v' and t.

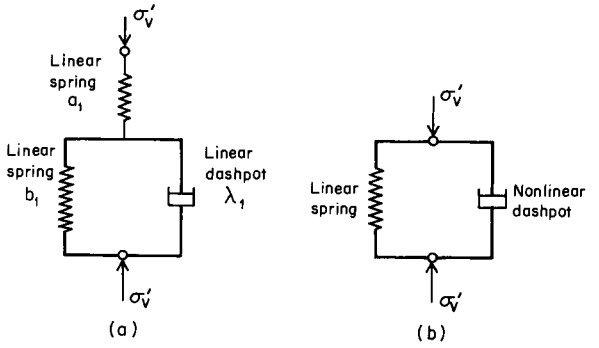


Fig. 7.19. Mechanical analogue of rheological constitutive behaviour: (a) Gibson & Lo model, (b) Barden model.

Further efforts to obtain improved predictions with experimental data have produced a large number of solutions derived from rheological models (Gibson & Lo, 1961; Lo, 1961; Wahls, 1962; Schiffman et al, 1964; Barden, 1965; Wu et al, 1966; Barden, 1969; Poskitt, 1971; Berry & Poskitt, 1972; and many others). All these models consider the consolidation and creep processes to occur simultaneously once the load increment has been applied. It can also occur that different assumed mechanisms of soil behaviour lead to similar rheological models. In such a case, only observations on a microscopic scale may decide which mechanism applies to a given soil (Barden, 1969).

An evaluation and description of the relative merits of each of the contributions made to soil rheology is beyond the scope of this review, and only two methods, namely that of Gibson & Lo (1961) and of Barden (1965, 1969) will be treated here in some more detail. Rheological models are not generally used in practice to predict settlement rates and pore water pressures, mainly because the parameters used in these theories are not widely known and sometimes difficult to measure.

7.4.4.2 Gibson & Lo model

The theory developed by Gibson & Lo (1961) can be considered as a direct extension of Terzaghi's theory. All of Terzaghi's assumptions are retained, but while Terzaghi's primary consolidation model can be represented by a single Hookean (linear) spring, Gibson & Lo (1961) connected the spring in series to a Kelvin body, like the model representing the Taylor-Merchant theory of Fig. 7.19a. In fact, Christie (1964) has shown that the mathematical equations developed by Merchant and by Gibson & Lo are identical. The model is characterized by the two spring constants (compressibilities) a_1 and b_1 and the dashpot constant λ_1 (or $1/\lambda_1$=viscosity of dashpot). The effective stress-strain relationship of the model is expressed by:

$$\varepsilon_v(\sigma_v' t) = a_1 \sigma_v'(t) + \int_o^t \sigma_v'(\tau) \exp\left[-\frac{\lambda_1}{b_1}(t-\delta)\right] d\tau \tag{7.57}$$

This equation can be written in terms of effective stress by making use of continuity of flow (Eq. 7.51) and $\Delta\sigma_v' = \Delta u$.

For settlement calculations, it is convenient to define the following dimensionless parameters:

$$M_1 = \frac{a_1+b_1}{a_1};\ N_1 = \frac{\lambda_1 h^2}{b_1 \theta};\ T_{GL} = \frac{\theta t}{h^2} \tag{7.58}$$

where $\theta = k_v/(a_1 \gamma_w)$, h is the length of the drainage path, and k_v is the permeability for vertical flow. The degree of consolidation settlement, U_s, at time t is then expressed as a function of the above variables, by:

$$U_s(t) = U_s(M_1, N_1, T_{GL}, \theta) \tag{7.59}$$

The Gibson & Lo time factor, T_{GL}, and the coefficient of consolidation, θ, correspond to the Terzaghi quantities T_v and c_v respectively. Graphs giving U_s versus T_{GL} for various values of M_1 and N_1 have been given by Gibson & Lo (1961), Christie (1964) and Lo et al (1976). Figure 7.20 shows such a graph for $M_1 = 1.43$ (i.e. when secondary compression accounts for 43% of the primary settlement) and five values of N_1 from 0 to ∞.

The settlement $\rho_{cs}(t)$ after time t (combining both primary and secondary components) of a layer of initial thickness h subjected to an external load q, is then obtained from:

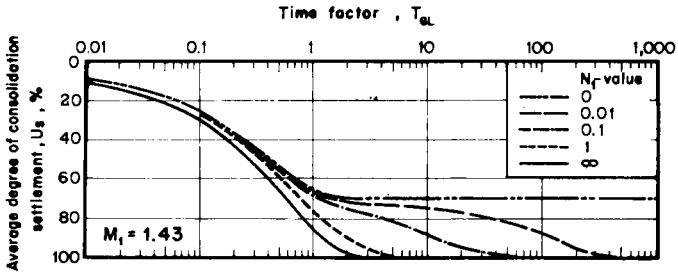


Fig. 7.20. Degree of settlement versus time factor curves obtained from the theory of Gibson & Lo, for $M_1=1.43$ and different values of N_1 (Christie, 1964).

$$\rho_{cs}(t) = (a_1+b_1)\, q\, h\, U_s(t) \tag{7.60a}$$

and

$$\rho_{cs} = (a_1+b_1)\, q\, h \tag{7.60b}$$

It should be noted here that the parameter N_1 is a function of the layer thickness, as well as of the soil parameters. Thus, the N_1 value for a thin oedometer sample (for which N_1 is typically between 0.01 and 0.1) and that for an actual soil layer of several metres thickness (N_1 between 1 and ∞) will be quite different. This implies that there exists no longer a similitude between the shapes of the time-settlement curves, and the h^2 scaling law becomes limited to the special case when $\lambda_1=0$ or ∞ (no secondary compression) and N_1 is independent of b_1.

In order to apply the theory to practical cases, the soil parameters a_1, b_1, λ_1 and θ must be determined from long-term oedometer tests. Two curves have to be plotted for each load increment; namely, one showing log $[d_f-d(t)]$ versus time, and the other giving $\rho_{cs}(t)$ versus $\sqrt{t}$. The quantities d_f and $d(t)$ are the dial readings corresponding to ρ_{cs} and $\rho_{cs}(t)$. Details of the procedure can be found in the original paper by Gibson & Lo (1961) and in that by Bourges et al (1971). These latter authors also presented a numerical example, and they recommended the method in French practice as an alternative to the Buisman-Koppejan procedure (Section 7.3.3).

Very few data can be found on the Gibson & Lo parameters in the literature. Figures 7.21, 7.22 & 7.23 show the variation of Gibson & Lo parameters a_1, b_1 and λ_1 with effective stress for Bangkok clay.

7.4.4.3 Barden model

Starting from a rheological equation proposed by Taylor (1942) which can be represented by a Kelvin body with a linear spring and dashpot (Fig. 7.19b), Barden (1965) introduced a nonlinear dashpot obeying a power law between viscous resistance (stress carried by the dashpot) and strain rate. In this model, the total stress is shared by three components, namely the intergranular force in the soil grains (represented by the spring), the viscous resistance in the water film surrounding the particles (represented by the dashpot), and the

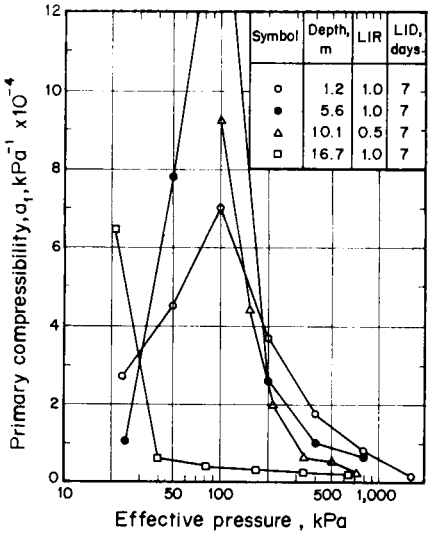


Fig. 7.21. Variation of primary compressibility with pressure for Bangkok clay from different depths.

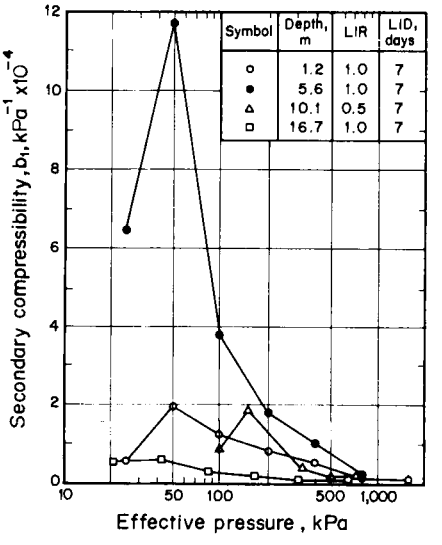


Fig. 7.22. Variation of secondary compressibility with pressure for Bangkok clay from different depths.

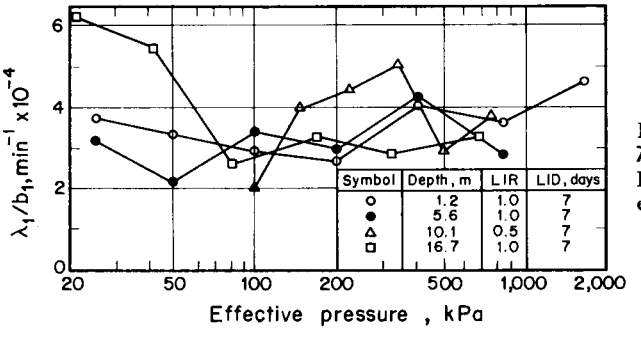


Fig. 7.23. Variation of ratio λ_1/b_1 with pressure for Bangkok clay from different depths.

excess pore pressure acting in the free pore water. The governing equations (see also Poskitt, 1967, for corrections made to Barden's paper) were solved by a finite difference procedure. Barden (1969) replaced the power law used for the dashpot by a relationship derived from rate process theory (Wu et al, 1966).

Barden's model is capable of realistically predicting the main departures from the Terzaghi theory, i.e. greater rate of pore pressure dissipation and creep effects during and after the dissipation of pore pressures. The dependence of the ratio of secondary to primary compression on load increment ratio and sample thickness could also be verified, and it was found that the coefficient of secondary compression, C_α, was within approximately $\pm 33\%$ of the mean, independent of load increment ratio, sample height and load increment duration. Parameters needed for Barden's model can be obtained from laboratory tests.

The two examples of rheological models discussed here have shown that this approach can provide excellent predictions of one-dimensional consolidation behaviour of soft clay. Today, however, rheological models may be considered as obsolete, or at most as of research interest only. The availability of powerful finite difference programs, such as CONMULT, for which input data from standard laboratory tests can directly be used, are preferred if the engineer decides to go beyond the conventional Terzaghi analysis in a more important project.

7.4.5 Graphical evaluation of settlement records (Asaoka's method)

A new and practical approach by which to estimate final total settlement and settlement rates from settlement data obtained during a certain time period was proposed by Asaoka (1978). He showed that the partial differential equation of consolidation expressed in terms of vertical (volumetric) strain as derived by Mikasa (1963):

$$c_v \frac{\partial^2 \varepsilon_v}{\partial z^2} = \frac{\partial \varepsilon_v}{\partial t} \tag{7.61}$$

can be approximated by an ordinary differential equation in series form, thus:

$$\rho + a_1 \frac{d\rho}{dt} + a_2 \frac{d^2\rho}{dt^2} + \ldots\ldots + a_n \frac{d^n\rho}{dt^n} + \ldots = b \tag{7.62}$$

where ρ represents the consolidation settlement (previously denoted by $\rho_c(t)$), and $a_1, a_2, \ldots\ldots a_n$ and b are constant coefficients which depend on the coefficient of consolidation and the boundary conditions of the soil layer. Asaoka's procedure utilizes the available settlement observations to identify these unknown coefficients, and can then predict future settlements based on the assessed parameters.

For the classical case of one-dimensional consolidation of a homogeneous layer with single drainage, Eq. 7.62 becomes (Magnan & Mieussens, 1980):

$$\rho + \frac{5}{12}\frac{h^2}{c_v}\frac{d\rho}{dt} + \ldots\ldots\ldots = h \,.\, \varepsilon_{vf} \tag{7.63}$$

where ε_{vf} is the final vertical strain at the top of the layer ($z = 0$) of thickness h.

A discretization of the time-settlement relationship $\rho(t)$ can be made as follows:

$$t_j = j \,.\, \Delta t; \quad j = 1,2,3, \ldots\ldots, \text{ and } \Delta t = \text{constant}$$

$$\rho_j = \rho(t_j) = \text{settlement at time } t_j \tag{7.64}$$

and Eq. 7.62 can be written in the form of an nth order recurrence relationship:

$$\rho_j = \beta_o + \sum_{i=1}^{n} \beta_i \, \rho_{j-i} \tag{7.65}$$

For a first order approximation ($n=1$), which is usually sufficient for most practical cases, Eqs. 7.62 & 7.65 reduce to:

$$\rho + a_1 \frac{d\rho}{dt} = b, \text{ with } a_1 = \frac{5}{12}\frac{h^2}{c_v} \tag{7.66}$$

$$\rho_j = \beta_o + \beta_1 \, \rho_{j-1} \tag{7.67}$$

Under stationary boundary conditions, the solution of Eq. 7.66 is:

$$\rho(t) = \rho_\infty - (\rho_\infty - \rho_o) \exp\left(-\frac{t}{a_1}\right) \tag{7.68}$$

where ρ_o and ρ_∞ are the initial and final settlements of the layer.

The boundary condition at $t=\infty$ is $\rho_j = \rho_{j-i} = \rho_\infty$, and substitution of this into Eq. 7.67 yields:

$$\rho_\infty = \frac{\beta_o}{1-\beta_1} \tag{7.69}$$

Thus, the recurrence relation, Eq. 7.67, becomes:

$$\rho_j = \frac{\beta_o}{1-\beta_1} - \left(\frac{\beta_o}{1-\beta_1} - \rho_o\right)(\beta_1)^j \tag{7.70}$$

Comparison of Eq. 7.70 with Eq. 7.68 shows that the coefficient β_1 is given by the relation:

$$ln\, \beta_1 = -\frac{\Delta t}{a_1} = -\frac{12}{5}\frac{c_v}{h^2}\Delta t \tag{7.71}$$

This is only valid if $\left|\frac{\Delta t}{a_1}\right| < 1$

Magnan & Deroy (1980) showed that, a_1 can be obtained much simpler from a first order approximation of the Fourier series solution of Terzaghi's equation (Eq. 7.40). This approach yields $a_1 = 4h^2/(\pi^2 c_v)$, which is almost the same as Asaoka's value, since $5/12 = 0.4167$ and $4/\pi^2 = 0.4053$. For values of $T_v > 0.05$, there exists an excellent agreement between the mathematical solution of Terzaghi's theory and Asaoka's first order approximation.

Asaoka suggested a graphical procedure as one possible way to solve the recurrence relation, Eq. 7.67. The usefulness of this procedure has been explored by the Laboratoire des Pont et Chausées in France, and promising results appear to have been obtained with settlement records of test embankments on soft clay deposits (Magnan & Mieussens, 1980; Magnan & Deroy, 1980).

The steps in the graphical procedure used by Magnan & Mieussens (1980) are as follows:

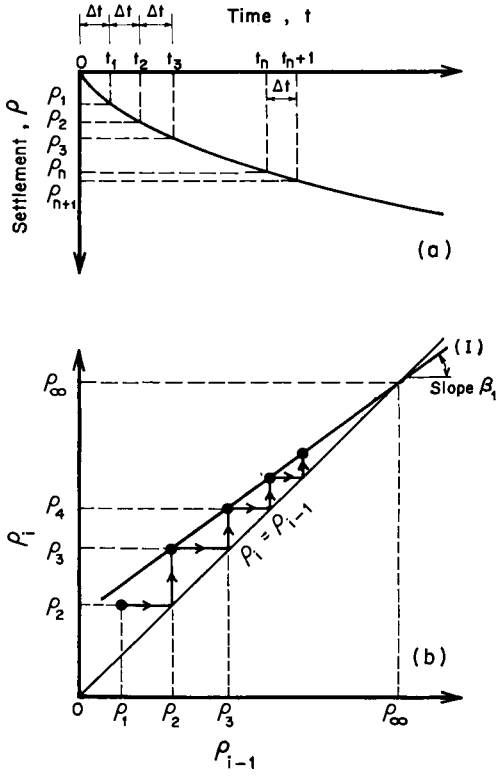


Fig. 7.24. Steps for the use of Asaoka's method: (a) partition of settlement record into equal time intervals, (b) plot of settlement values and fitting of straight line (Magnan & Deroy, 1980).

(1) The observed time-settlement curve plotted to an arithmetic scale is divided into equal time intervals, Δt, (usually Δt is between 30 and 100 days). The settlements ρ_1, ρ_2 . . . . . corresponding to the times t_1, t_2, ... are read off and tabulated (see Fig. 7.24a).

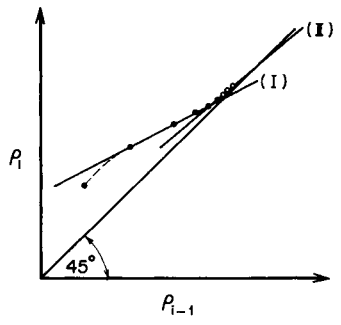
(2) The settlement values ρ_1, ρ_2 . . . . . are plotted as points (ρ_{i-1}, ρ_i) in a coordinate system with axes ρ_{i-1} and ρ_i, as shown in Fig. 7.24b. The 45° line $\rho_i = \rho_{i-1}$ is also drawn.

(3) A straight line (*I*) is fitted through the points. The point where this line intersects the 45° line gives the final consolidation settlement, ρ_∞ (or ρ_c). The slope β_1 is related to the coefficient of consolidation, c_v, by:

$$c_v = -\frac{5}{12} h^2 \frac{ln\, \beta_1}{\Delta t} \tag{7.72}$$

and therefore indicates the rate of settlement. The slope, β_1, depends on the time step, Δt, selected and decreases when Δt increases.

In some cases, it occurs that the data points are better fitted by two straight lines (*I*) and (*II*), as shown in Fig. 7.25. The second straight line, (*II*), corresponds to the tail of the settlement curve, and thus represents secondary compression. In the case of multi-stage loading, a straight line corresponding to each loading stage can be fitted. If the time between the load increments is large, a secondary compression line can also be drawn. Figure 7.26 shows a case of a load applied in three stages with secondary compression at the end of the third stage. In some rare cases, according to Magnan & Deroy (1980), it is difficult to define a straight line through the points.



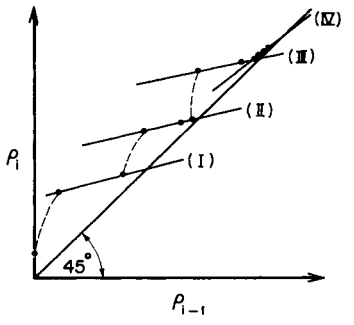


Fig. 7.25. *(left)* Asaoka's method for single-stage loading and secondary compression (Magnan & Deroy, 1980).

Fig. 7.26. *(right)* Asaoka's method for multi-stage loading and secondary compression occurring at the end of the third loading stage (Magnan & Deroy, 1980).

In the event of creep, a coefficient, λ_c, can be defined from line (*II*) by:

$$\rho(t) = \rho_\infty [1 - \exp(-\lambda_c t)] \tag{7.73}$$

and

$$\lambda_c = -\frac{ln\, \beta_1}{\Delta t} \tag{7.74}$$

Equation 7.73 represents an exponential creep law, which is different from the logarithm of time law usually employed for secondary compression (Eq. 7.6).

Asaoka's method can also be applied to sand drain problems. The graphical procedure in the form presented here is limited to single layers with one-way or two-way drainage. An extension of the general theory to multilayer systems has recently been accomplished (Asaoka & Matsuo, 1980).

An important feature of Asaoka's method is that it allows the separate determination of the coefficient of consolidation for vertical drainage, c_v, (or for radial drainage, c_r, in the case of sand drains) and the final value of settlement. In the conventional method, both the final settlement and c_v must be estimated by fitting the theoretical curve to the observed one, and final settlements cannot be determined accurately if the settlement record is not known from the beginning of consolidation. Furthermore, the sharp bend, characteristic for most settlement records, occurs around 90% primary consolidation, and when Terzaghi's solution is employed, no conclusions regarding c_v can be made unless the observation time is sufficiently long. With Asaoka's method, good predictions are usually possible after 60% consolidation has been achieved (Magnan & Deroy, 1980).

Examples of applications of Asaoka's method to embankment settlements have also been given by Magnan & Deroy (1980). They first determined c_v and ρ_∞, and then the settlement-time relationship was obtained by means of degree of consolidation values, U_s, calculated from the one-dimensional Terzaghi theory.

7.5 METHODS OF PREDICTING TWO- AND THREE-DIMENSIONAL CONSOLIDATION SETTLEMENTS

7.5.1 General considerations

As stated previously (Section 7.2.2), one-dimensional consolidation theory often underestimates the rate of settlement in field situations. In most cases, this can be attributed to horizontal dissipation of pore pressures. Such situations can be better analyzed by employing a consolidation theory which accounts for three-dimensional drainage and strain conditions.

There are two basic approaches commonly used for solving two- or three-dimensional consolidation problems. The first kind is the direct extension of Terzaghi's one-dimensional diffusion type theory proposed by Rendulic

(1936), and is often called the *Terzaghi-Rendulic pseudo-consolidation theory* or *diffusion theory*. This theory assumes that the total stress remains constant during consolidation, i.e. the internal volumetric components of total stress are assumed to have the same time history of behaviour as the applied volumetric stress components (Schiffman et al, 1969). During consolidation, however, pore pressure dissipation rates vary depending on the distance from the drainage boundaries, and since Young's modulus and Poisson's ratio change from their undrained to their drained values, differential strains occur, which in turn require an adjustment of the total stress to satisfy stress-strain compatibility. The pseudo-theory does not provide for such a coupling between the magnitude and the progress of settlement; it assumes that only the dissipation of the excess pore pressure contributes to the progress of settlement.

The second theory was derived directly from the theory of elasticity by Biot (Biot, 1941), and is commonly known as the *Biot theory*. It is mathematically much more complex than the pseudo-theory. However, the theory provides for a coupling between magnitude and progress of displacement, and at any point in the consolidating layer, there is continuous interaction between the dissipating excess pore pressure and the changing total stress. The original Biot theory, formulated first for a homogeneous, isotropic, fully saturated medium, has been refined repeatedly to account for compressible fluids and the recognition that the fluid velocity is relative to the solid velocity (Biot, 1955, 1956). However, the theory has remained limited to research problems in its application. With today's powerful numerical techniques, in particular the finite element method which can include nonlinear effects, an increased use of Biot's theory is to be expected.

In Table 7.3, the basic consolidation equations for the Terzaghi-Rendulic theory and the Biot theory are summarized. With the pseudo-theory, either the one-dimensional coefficients of consolidation for each space direction, c_x, c_y & c_z, can be used (or c_v & c_h for two-dimensional problems), or a single value of the coefficient can be adopted, which will differ with the number of space dimensions. The three coefficients of consolidation shown in Table 7.3 are related by:

$$c_1 = 2\,(1-\upsilon')\,c_2 = 3\left(\frac{1-\upsilon'}{1+\upsilon'}\right)c_3 \tag{7.75}$$

The equations for the Biot theory (see, for example, Lee et al, 1968, or Christian, 1977, for their derivation) contain an additional term representing the time derivative of the bulk or mean total stress. This term takes care of the changing total stress within the soil mass. For $\upsilon'=0.5$, i.e. undrained conditions, $c_1=c_2=c_3$, and the rate of change in mean total stress is zero. In other words, for this special case, the two theories are identical.

It should also be pointed out that, in two-and three-dimensional problems, the degree of consolidation settlement, U_s, is no longer equal to the average

Table 7.3. Equations for consolidation theories in one, two and three dimensions (Murray, 1978)

Number of space dimensions	Terzaghi-Rendulic pseudo-consolidation theory	Biot theory	Coefficient of consolidation
1	$\frac{\partial u_e}{\partial t} = c_1 \frac{\partial^2 u_e}{\partial z^2}$	$\frac{\partial u_e}{\partial t} = c_1 \frac{\partial^2 u_e}{\partial z^2}$	$c_1 = \frac{kE\,(1-\upsilon)}{\Upsilon_w(1+\upsilon)\,(1-2\upsilon)}$
2	$\frac{\partial u_e}{\partial t} = c_2 \left(\frac{\partial^2 u_e}{\partial x^2} + \frac{\partial^2 u_e}{\partial z^2}\right)$ or: $\frac{\partial u_e}{\partial t} = c_x \frac{\partial^2 u_e}{\partial x^2} + c_z \frac{\partial^2 u_e}{\partial z^2}$	$\frac{\partial u_e}{\partial t} = c_2 \left(\frac{\partial^2 u_e}{\partial x^2} + \frac{\partial^2 u_e}{\partial z^2}\right) + \frac{1}{2}\frac{\partial(\sigma_x+\sigma_z)}{\partial t}$	$c_2 = \frac{kE}{2\Upsilon_w(1-2\upsilon)\,(1+\upsilon)}$
3	$\frac{\partial u_e}{\partial t} = c_3 \left(\frac{\partial^2 u_e}{\partial x^2} + \frac{\partial^2 u_e}{\partial y^2} + \frac{\partial^2 u_e}{\partial z^2}\right)$ or: $\frac{\partial u_e}{\partial t} = c_x \frac{\partial^2 u_e}{\partial x^2} + c_y \frac{\partial^2 u_e}{\partial y^2} + c_z \frac{\partial^2 u_e}{\partial z^2}$	$\frac{\partial u_e}{\partial t} = c_3 \left(\frac{\partial^2 u_e}{\partial x^2} + \frac{\partial^2 u_e}{\partial y^2} + \frac{\partial^2 u_e}{\partial z^2}\right) + \frac{1}{3}\frac{\partial(\sigma_x+\sigma_y+\sigma_z)}{\partial t}$	$c_3 = \frac{kE}{3\Upsilon_w(1-2\upsilon)}$

Notation: υ = Poisson's ratio of soil skeleton, E = Young's modulus of soil skeleton.
$\sigma_x, \sigma_y, \sigma_z$ = total stress increments in x, y and z directions respectively at a point in the soil mass.
c_x, c_y, c_z = one-dimensional coefficients of consolidation in x, y and z directions respectively.

degree of pore pressure dissipation, U_p, since there will always be some stress redistribution. With the pseudo-theory, however, U_s can only be obtained by equating it to U_p (Davis & Poulos, 1972).

7.5.2 *Terzaghi-Rendulic theory*

Solutions for the three-dimensional flow of water but only one-dimensional vertical strains have been applied to various practical problems, since they are relatively simple to handle in a mathematical sense. A well-known example is Barron's sand drain theory (Barron, 1948).

Davis & Poulos (1972) obtained a series of curves for the average degree of pore pressure dissipation, U_p, by solving the diffusion equation by a finite difference technique. They obtained the initial excess pore pressure distribu-

tion from the relationship for an ideal elastic two-phase material of $u_{eo}=(\sigma_x+\sigma_y+\sigma_z)/3$, and the distribution of total stress for $\upsilon'=0.5$ from elastic theory for a layer on a rough rigid base.

Figures 7.27a & 7.27b show curves of U_p $(=U_s)$ versus time factor for a strip footing for various values of h/b (where h is the layer thickness, and b is the half-width of the strip load). The time factor is defined as $T_v=c_1t/h^2$, i.e. in terms of the one-dimensional coefficient of consolidation (Table 7.3). The hydraulic boundary conditions assume a permeable upper surface for both cases, while the base has been assumed as permeable in Fig. 7.27a and as impermeable in Fig. 7.27b. Davis & Poulos stated that these curves may also be applied to rigid footings with reasonable accuracy. It can be seen from Fig. 7.27 that, for strip footings (which roughly simulate embankment loads), the influence of three-dimensional flow only becomes important when the layer thickness is larger than the width of the strip, otherwise the deviation from the one-dimensional Terzaghi theory is less than 20%. Other results by Davis & Poulos (1972), however, show that for circular footings, the influence of the layer depth is much more pronounced.

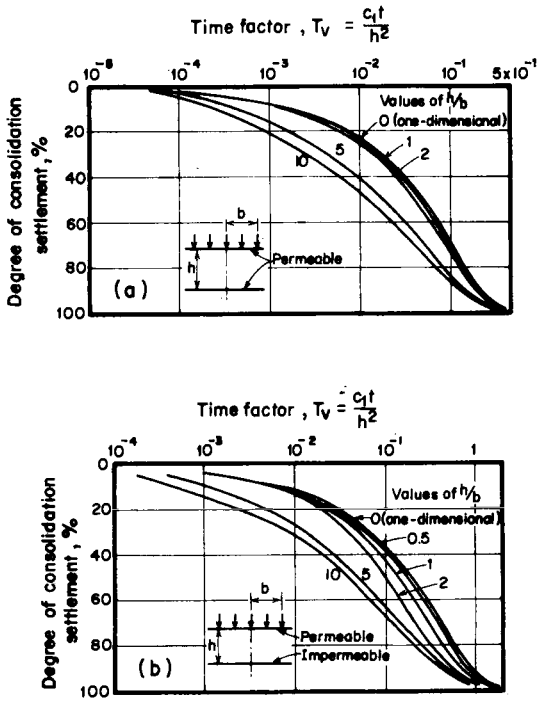


Fig. 7.27. Degree of consolidation settlement beneath the centre of a strip footing versus time factor: (a) permeable upper surface and permeable base, (b) permeable upper surface and impermeable base (Davis & Poulos, 1972).

A useful application of two-dimensional pseudo-theory to the design of road embankments was given by Dunn & Razouki (1974). Their analysis included the influence of embankment geometry, layer thickness and anisotropy of the soil, and the results were presented in a form suitable for design, as shown in the review by Murray (1978). Figure 7.28 illustrates the influence of drainage conditions (permeable or impermeable base), embankment shape (expressed by the ratio of the width at the top to that at the bottom of the embankment), and layer thickness, on two degrees of consolidation. The shape of the embankment does not seem to have a great effect, especially with a degree of consolidation of 90% and, for the conditions described, can thus be ignored.

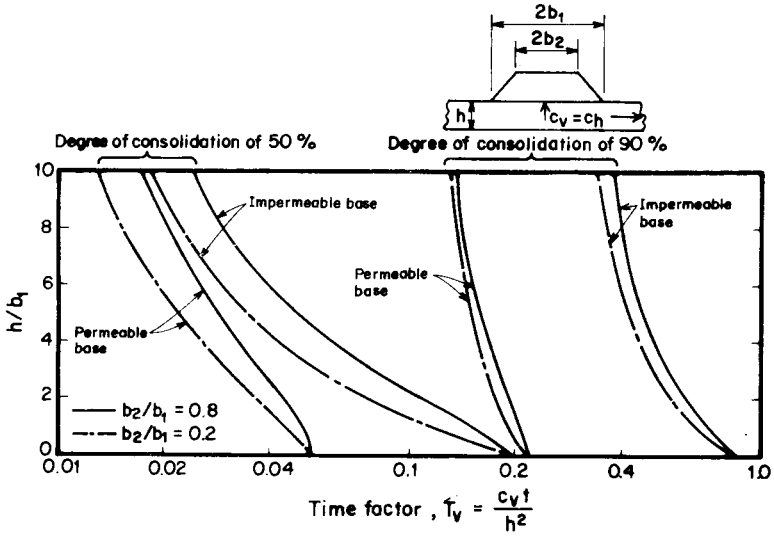


Fig. 7.28. Influence of embankment width on the progress of consolidation beneath a long embankment founded on a homogeneous, isotropic soil (Murray, 1978).

7.5.3 Biot theory

Solutions to the Biot theory in closed analytical form are limited because of the complex nature of the problem. Situations which have been solved are (Davis & Poulos, 1972): (1) uniformly loaded circle on a semi-infinite mass (De Josselin de Jong, 1957; McNamee & Gibson, 1960; Schiffman & Fungaroli, 1965), (2) uniformly loaded rectangle on a semi-infinite mass (Gibson & McNamee, 1957, 1963), (3) uniformly loaded strip on a semi-infinite mass (McNamee & Gibson, 1960; Schiffman et al, 1969), (4) uniformly loaded strip and circle on a finite layer (Gibson et al, 1970; Yamaguchi & Murakami, 1976), (5) sphere subjected to hydrostatic pressure (Cryer, 1963), and (6) concentrated and circular loading of a deep soil mass overlain by a permeable layer (Mandel, 1957, 1961).

More common now are solutions by means of a finite element formulation of the Biot equations (Sandhu & Wilson, 1969; Yokoo et al, 1971; Hwang et al, 1971; Ghaboussi & Wilson, 1973; Desai, 1975; and others), but general design charts cannot be established when the geometry of the problem loses its simplicity. Examples involving the predicted and observed behaviour of embankments using the Biot theory have been presented by Smith & Hobbs (1976) and by Shoji & Matsumoto (1976).

An interesting feature of the Biot theory is the so-called *Mandel-Cryer effect* or *stress transfer effect* (Mandel, 1957; Cryer, 1963). The effect indicates that the pore pressure in the consolidating layer can increase some time after load application and may reach values which are higher than the applied pressure. This phenomenon can be verified in laboratory experiments (Aboshi, 1955), but it is difficult to observe in the field because of the many factors that affect the accuracy of piezometer readings. A physical explanation of the phenomenon can be given by viewing an element of soil which is draining as one which is squeezing the adjacent element and thus producing an increase in total volumetric stress. The pressure increase starts near the drainage boundary and is then transferred to the interior of the soil layer. The magnitude of the effect increases with depth because of the growing zone of influence (Schiffman et al, 1969). If the surface of the layer is impervious, the effect is almost completely absent. The magnitude of the Mandel-Cryer effect also depends on Poisson's ratio; it is reduced when υ' increases. The Mandel-Cryer effect does not occur when diffusion theory is used.

Figure 7.29 shows results of a Biot-type analysis applied to the plane strain consolidation problem of a flexible strip load footing overlying a clay layer with an impermeable base, published by Yamaguchi & Murakami (1976). This figure may be compared with Fig. 7.27. For low values of Poisson's ratio, υ', a peculiar phenomenon can be noticed, namely that for larger values of b/h (i.e. embankment-type loading), the consolidation settlement is retarded as compared to the Terzaghi one-dimensional theory. Yamaguchi & Murakami related this phenomenon to the Mandel-Cryer effect, because the increase in pore pressure during the initial phase of consolidation would also affect the settlement, since in the Biot theory the soil skeleton deformation and pore pressure dissipation are interdependent. For $\upsilon'=0.4$, the effect is almost negligible (Fig. 7.29c) and disappears for $\upsilon'=0.5$.

A useful contribution to multi-dimensional consolidation theory was made by Christian et al (1972) based on a finite element Biot-type analysis. They presented a parametric study of the effects of load width, material constants and anisotropy for the plane-strain consolidation of a strip footing. They found that, for ratios of half-width to layer thickness, $b/h>1$, the times for specific degrees of settlement were virtually the same. This result is in agreement with that of Davis & Poulos (1972) (Figs. 7.27) obtained by the diffusion theory.

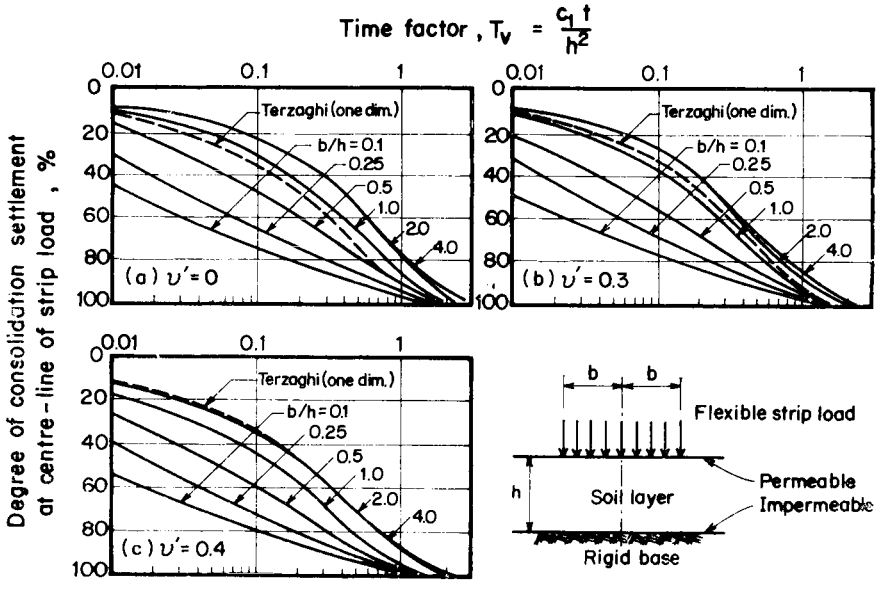


Fig. 7.29. Degree of settlement of a flexible strip load overlying a clay layer on an impervious base versus time factor: (a) $\upsilon'=0$, (b) $\upsilon'=0.3$, (c) $\upsilon'=0.4$ (Yamaguchi & Murakami, 1976).

Christian et al (1972) also showed that the time factor for 90% consolidation is about 0.75 for $b/h=1$ and υ' between 0.25 and 0.40; the time factor in this case was defined as $T_1=c_1t/h^2$, with c_1 given in Table 7.3. Since 0.75 is fairly close to 0.848 (the value corresponding to one-dimensional consolidation theory), these authors recommended the use of conventional one-dimensional theory for predicting the rates of consolidation settlement of strip loads on surface soil with a rough base when $b/h>1$. On the other hand, for $b/h<1$, Christian et al (1972) provided a table to obtain a corrected (reduced) time factor (Table 7.4).

Table 7.4. Effect of load width on time factor (Christian et al, 1972)

b/h	T_{10}	T_{50}	T_{75}	T_{90}
0.25	0.007	0.09	0.30	0.60
0.5	0.011	0.14	0.35	0.68
1	0.016	0.20	0.45	0.80
2	0.016	0.24	0.53	0.90
∞ (Terzaghi theory)	0.008	0.197	0.48	0.848

In order to evaluate the effects of permeability anisotropy, Christian et al (1972) defined a parameter α_k as:

$$\alpha_k = \frac{k_{eff}}{\sqrt{k_h k_v}} \tag{7.76}$$

where k_{eff} is an effective permeability which would cause the same relative amount of consolidation settlement in the isotropic case as that observed in the anisotropic case with permeabilities k_h and k_v in horizontal and vertical directions respectively.

In Fig. 7.30a, the ratio α_k is plotted versus k_h/k_v for 85% consolidation and various drainage conditions. In this graph, α_k never exceeds the value of 1.0, which means that flow in the horizontal direction is less effective than one would expect. On the other hand, Fig. 7.30b shows the situation for 50% consolidation. The accelerated consolidation is particularly pronounced for single drainage. This is because, during the early stages of consolidation, the excess pore pressures are concentrated directly under the load, and they dissipate rapidly by lateral flow when the horizontal permeability is high. At later times, the rate of consolidation decreases as the contribution of vertical drainage gains in significance.

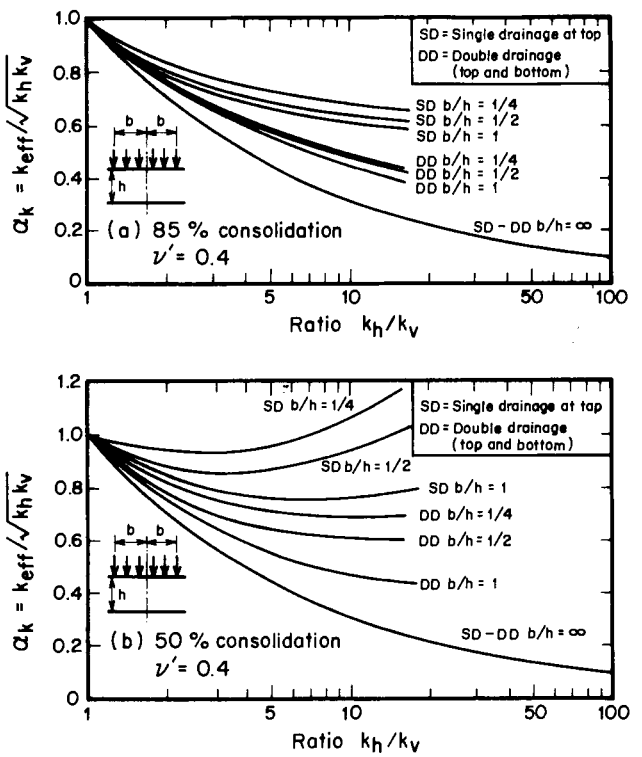


Fig. 7.30. Parameter α_k versus permeability ratio, k_h/k_v, at: (a) 85% consolidation, (b) 50% consolidation (Christian et al, 1972).

An example given by Christian et al (1972) shows how Fig. 7.30 and Table 7.4 can be used to correct conventional one-dimensional theory ($b/h=\infty$ in this case) for effects of anisotropy. Assume $b/h=1$, $k_h/k_v=10$ and double drainage, and determine the time for 90% consolidation, $t_{90}(2D)$. First, the time for 90% consolidation is obtained for one-dimensional theory, t_{90} $(1D)$ (i.e. from $T_{90}=0.848$). Then, this value is corrected for loading width by means of Table 7.4, and finally for anisotropy by means of Fig. 7.30a. Hence:

$$t_{90(2D)} = t_{90(1D)} \frac{T_{90} \text{ for } b/h=1}{T_{90} \text{ for } b/h=\infty} \frac{1}{a_k\sqrt{k_h/k_v}}$$

$$= t_{90(1D)} \cdot \frac{0.80}{0.848} \cdot \frac{1}{0.45\sqrt{10}} = 0.663\ t_{90(1D)}$$

7.5.4 Comparison between pseudo-theory and Biot theory

In order to compare the degree of consolidation settlement, U_s, obtained from the diffusion and Biot theories, it is most suitable to define the time factor in terms of the one-dimensional coefficient of consolidation, c_1 (see Table 7.3). For comparison of pore pressures, however, the time factor must be expressed in terms of c_2 (plane strain conditions) or c_3 (triaxial conditions) (Davis & Poulos, 1972). In Fig. 7.31, the effect of Poisson's ratio on the degree of settlement when using the Biot theory is again demonstrated, and a comparison is made with the solution obtained by Davis & Poulos (1972) employing the diffusion theory for the case of a strip footing on a finite layer. It can be seen that the prediction which is based on the diffusion theory lies between the curves based on Biot's theory calculated for the values of $\nu'=0$ and 0.45. The difference between the curves is not great, at least for the ratio $h/b=1.0$ for which the curves were calculated.

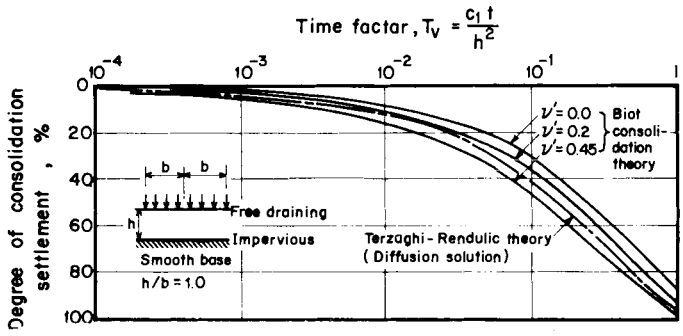


Fig. 7.31. Influence of Poisson's ratio on the progress of plane strain consolidation beneath the centre of a strip load (Murray, 1978).

Schiffman et al (1969) have clearly demonstrated that, from a theoretical point of view, the diffusion theory solution is in general not a valid approximation for two- and three-dimensional problems. For many practical cases, such as embankment-type loading with $b/h>1$, particularly in view of the uncertainties in the soil parameters selected in an analysis, the simple diffusion theory may be considered a sufficient approximation to find the settlement versus time relationship.

7.6 DETERMINATION OF SOIL PARAMETERS FOR SETTLEMENT ANALYSES

7.6.1 General characteristics and testing procedures

The preceding review on methods of predicting magnitude and rate of settlement has shown that different methods are also associated with different sets of soil parameters which cannot all be determined from the same type of test. For one-dimensional types of analysis, the oedometer test has been the primary test for many years to determine compressibility and consolidation parameters. The introduction of the stress path method and the use of elastic parameters (E_u, E', υ') in three-dimensional and finite element type analyses has lead to an increased application of the triaxial test. Since most analysis procedures require a knowledge of the initial in-situ stress conditions, various special tests have been divised to determine the coefficient of earth pressure at rest, K_o. Poulos (1975) reviewed various laboratory testing procedures to evaluate soil parameters in relation to the elastic type analysis of shallow foundations.

Among the oedometer techniques, the incremental load test (see e.g. Lambe, 1951) is the most common one in present-day practice. Other procedures which have been proposed are the controlled gradient test (Lowe et al, 1969), the constant-rate-of-loading test (Aboshi et al, 1970) and the constant-rate-of-strain (CRS) test (Smith & Wahls, 1969; Wissa et al, 1971; Sällfors, 1975). These improved testing techniques produce a continuous compression curve and enable a more accurate determination of compressibility and coefficient of consolidation. Their main advantage, however, is the greatly reduced testing time compared to the conventional incremental test. On the other hand, more sophisticated equipment is needed, and the continuous readings to be taken require an automated data acquisition system. Information on secondary compression can be obtained with the equipment used for the controlled gradient test but not with all types of equipment employed in the CRS test.

The idea of the controlled gradient test is to achieve uniform effective stress conditions and uniform rates of compression (possibly of similar magnitude as under field conditions) throughout the specimen by controlling the gradient of excess pore pressure. In incremental loading tests, the initial compression rates are much higher than in the field because of the high gradients of excess pore pressure near the drainage boundaries. In the constant-

rate-of-strain test, the vertical deformation, Δh, is proportional to rht, where r is the rate of strain and t is the time.

7.6.2 *Preconsolidation pressure*

7.6.2.1 *Mechanisms causing a preconsolidation pressure*

A clay whose in-situ stress, σ'_{vo}, is less than the *preconsolidation pressure*, σ'_{vc}, (other names used are *maximum past pressure, critical pressure* or *amount of precompression,* and as alternative notation, σ'_{vm} or p_c are often employed) is, regardless of the cause, called *overconsolidated;* if $\sigma'_{vo}=\sigma'_{vc}$, it is designated as *normally consolidated.* The ratio $\sigma'_{vc}/\sigma'_{vo}$ is known as the *overconsolidation ratio, OCR.*

The geological history a deposit of clay has passed through determines the amount of precompression. Various geological factors responsible for causing precompression have now been recognized, and they are listed in Table 7.5. It is good practice when selecting a σ'_{vc} value from the results of oedometer tests for design to try to identify the mechanism involved.

Table 7.5. Mechanisms that cause a preconsolidation pressure (Brumund et al, 1976)

Item	Remarks and references
Change in total stress due to:	
(1) Removal of overburden.	
(2) Past structures.	
(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.
(5) Desiccation due to plant life.	May have occurred during deposition.
Changes in soil structure due to:	
(1) Secondary compression (aging).*	Leonards & Ramiah (1959), Leonards & Altschaeffl (1964), Bjerrum (1967, 1972, 1973).
(2) Environmental changes, such as pH, temperature, salt concentration.	Lambe (1958a, 1958b).
(3) Chemical alterations due to: 'weathering', precipitation of cementing agents, ion exchange.	Bjerrum (1967).
Change of strain rate on loading	Lowe (1974).

* 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).

If overconsolidation is not caused by the removal of a previous load (i.e. no change in total stress), then the precompression effect is sometimes designated as *quasi-preconsolidation* (Leonards & Altschaeffl, 1964). A typical example of quasi-preconsolidation is the stiffening effect caused in normally consolidated soft clay by aging or secondary compression. In Fig. 7.9, compression curves corresponding to various times are shown, and it can be seen that when the 10,000 year old sample (e_o, σ'_{vo}) is loaded, it will exhibit a preconsolidation pressure of $\sigma'_{vc} > \sigma'_{vo}$. From the geometry of the time lines in Fig. 7.9, the following expression can readily be derived (Mesri & Choi, 1979b):

$$\frac{\sigma'_{vc}}{\sigma'_{vo}} = \left(\frac{t}{t_p}\right)^{C_\alpha/C_c} \tag{7.77}$$

where t_p is the time required for natural deposition and primary consolidation of the clay, and t is the age of the deposit.

7.6.2.2 Determination of preconsolidation pressure

A recommended procedure to determine σ'_{vc} from incremental oedometer tests has recently been published by the Transportation Research Board (Brumund et al, 1976). With e versus log σ'_v curves which exhibit a well-defineable break, Casagrande's method (Casagrande, 1936) is most widely employed in practice (see Fig. 7.4). Often, a range of probable σ'_{vc} values, rather than a single value, is defined, with the upper limit taken as the value found from the Casagrande construction, and the lower limit, $\sigma'_{vc(min)}$, as the point of intersection of a horizontal line through (e_o, σ'_{vo}) and the extension of the straight-line portion of the compression curve (also shown in Fig. 7.4). Casagrande's method is considered to yield satisfactory results with good quality samples and properly conducted tests (Brumund et al, 1976).

Other procedures for finding σ'_{vc} from an e—log σ'_v curve have been proposed by Burmister (1951) and Schmertmann (1955). However, both methods require a rebound and reloading cycle. Schmertmann's method is illustrated in Fig. 7.32 and can be useful for e—log σ'_v curves for which the definition of a minimum radius is difficult because of sample disturbance. The procedure consists of finding, by trial and error, the location of σ'_{vc}, such that the resulting void ratio reduction pattern (Δe plotted versus pressure) becomes symmetrical with respect to σ'_{vc}. The method also permits an assessment of sample disturbance and the construction of an in-situ compression curve. Schmertmann observed that the e—log σ'_v curves for samples of various degrees of disturbance merge together at a void ratio equal to approximately 40% of the initial void ratio, e_o, and that the swelling curves have essentially all the same slope regardless of the pressure at which the rebound starts.

A more recent procedure to determine σ'_{vc} utilizes the data of a constant-rate-of-strain test. If the constrained modulus, M_D, ($= 1/m_v = \Delta\sigma'_v/\Delta\varepsilon_v$) is plotted versus the vertical effective stress to an arithmetic scale (rather than e

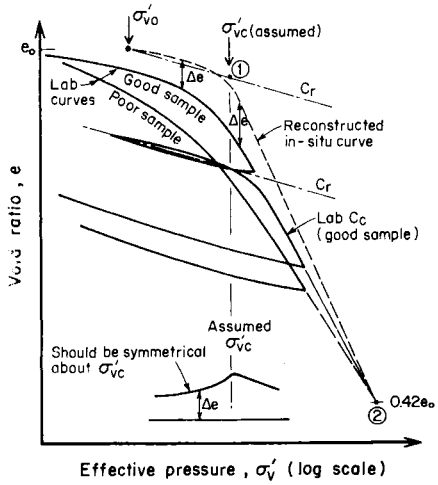


Fig. 7.32. Preconsolidation pressure and reconstruction of in-situ compression curve using Schmertmann's method (Schmertmann, 1955).

versus log σ'_v), a V-shaped curve with a sharp break is obtained; the break corresponds to σ'_{vc} (Wissa et al, 1971). This idea was originally proposed by Janbu (1969) and is probably superior to Casagrande's method. The V-shaped curve, however, is also generally observed with incremental oedometer tests, as shown by Stamatopoulos & Kotzias (1973, 1978). These authors defined a "specific constrained modulus" as the M_D-value corresponding to $\sigma'_{vo}+50$ kPa, which can be utilized directly for settlement calculations. Stamatopoulos & Kotzias (1973) further demonstrated that the M_D versus σ'_v plot is compatible with the e against log σ'_v relationship.

The position and shape of the e—log σ'_v curve depends also on the load increment ratio, *LIR*, and the load increment duration, *LID*. The load increment ratio, defined as the ratio of load increment to preceding consolidation pressure, is taken as unity in 'conventional' tests, which means that the pressure is doubled with each successive load increment. For very soft plastic and sensitive clays, in which the preconsolidation pressure was caused by a change in soil structure (Table 7.5), this procedure is not satisfactory, since the clay might undergo sudden collapse or squeeze out of the consolidometer, and σ'_{vc} will thereby be underestimated or totally obliterated. The influence of LIR on the magnitude of σ'_{vc} was first demonstrated by Leonards & Altschaeffl (1964) with tests on various normally consolidated clays. In order to enable an adequate definition of σ'_{vc}, a reduction in *LIR* in the vicinity of σ'_{vc} is necessary. Bjerrum (1973) proposed the use of increments of $\frac{1}{3}$ $(\sigma'_{vc}-\sigma'_{vo})$, while Brumund et al (1976) recommended load steps of 0.5 σ'_{vc}, 0.75 σ'_{vc}, 0.85 σ'_{vc}, 1.0 σ'_{vc}, 1.3 σ'_{vc} and 1.5 σ'_{vc}.

The load increment duration, *LID*, denotes the time allowed for consolidation prior to the application of the next load increment. In 'conventional'

tests, an *LID* of one day is used, but the time needed for primary consolidation of soft clay samples with the thicknesses usually employed in oedometer tests, is only about 5 to 100 minutes. Secondary compression, therefore, causes the e—log σ'_v curve to fall below the compression curve corresponding to the end of primary consolidation, thus reducing σ'_{vc} by as much as 10 to 20% (Ladd, 1973). This reduction has also been verified by Crawford (1964) with samples of Leda clay. In the case of constant-rate-of-strain tests, Vaid et al (1979) observed a decrease in σ'_{vc} with a reduction in imposed strain rate. This observation, as pointed out by Mesri & Choi (1979b), is analogous to the influence of *LID* in the incremental load test. For further details on recommended testing procedures, test equipment and data interpretation of incremental oedometer tests, the reader is referred to Brumund et al (1976).

7.6.3 Compression and recompression index

7.6.3.1 Definition and general characteristics

The compression index, C_c, and the recompression index, C_r, defined as $\Delta e/\Delta \log \sigma'_v$ in the virgin and recompression range respectively, are the parameters most frequently used to quantify the compressibility of a clay. In some European countries (e.g. Germany), it is more common to make use of the constrained modulus for settlement calculations. The compressibility of a saturated clay is a direct function of its void ratio, e_o, or of its water content, w_n, and it usually decreases with depth. This is illustrated in Fig. 7.33, where e—log σ'_v curves for Bangkok clay from four different depths corresponding to different strata are plotted. The shapes of the curves for the weathered, soft and medium clay are similar, whereas the curve for the stiff clay, which has a different geological age, is flat, resembling a recompression curve. Schmertmann (1955) demonstrated with samples from different depths of a post-glacial deposit of organic silty clay, that the position of the virgin portion of the e—log σ'_v curve represents essentially a unique property of the deposit and is independent of e_o and σ'_{vc}. Sample disturbance decreases the slope of the e—log σ'_v curve in the virgin region.

In Fig. 7.34, the variations in the compression index with effective pressure for the curves of Fig. 7.33 are shown. Except for the stiff clay, they all exhibit a peak in the vicinity of the preconsolidation pressure, the height of which is related to the sensitivity of the clay and to sample quality. With increasing consolidation pressure in the virgin range, the compression indices appear to approach a common value. For soft, normally consolidated or slightly overconsolidated clays, the compression index in the vicinity of σ'_{vc} is typically somewhat larger than 1.0, but it can reach values as high as 10 (e.g. Mexico City clay; Mesri et al, 1975).

7.6.3.2 Empirical correlations

Compression indices are frequently determined quantities in soil investigation programmes, and this makes them ideally suited for statistical treatment.

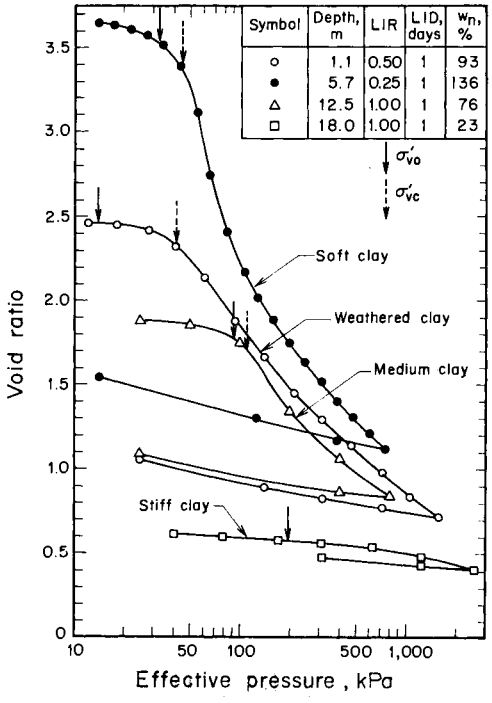


Fig. 7.33. Typical void ratio versus effective pressure curves for Bangkok clay from Bangpli.

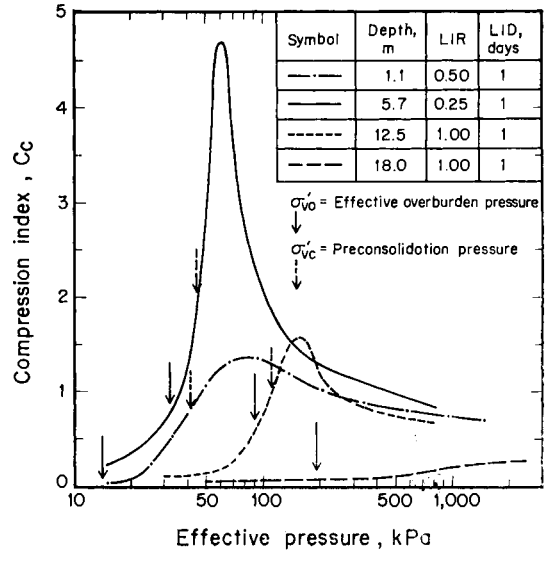


Fig. 7.34. Variation of compression index with effective pressure for Bangkok clay from Bangpli.

Numerous single and multiple correlations between C_c or the compression ratio CR (defined as $C_c/(1+e_o)$) and basic physical properties, such as natural water content, w_n, initial void ratio, e_o, and liquid limit, w_L, have been proposed (Nishida, 1956; Cozzolino, 1961; Terzaghi & Peck, 1967; Lambe & Whitman, 1969; Elnaggar & Krizek, 1970; Azzouz et al, 1976; Krizek et al, 1977), with the purpose of predicting C_c or CR from simple laboratory tests which are less costly than oedometer tests. A selection of such correlations is given in Table 7.6. Differences in the regression equations may be attributed to the use of different data sources from which these equations were established. Correlations of specific constrained modulus with dry density are also available (Stamatopoulos & Kotzias, 1973).

It has been shown (Azzouz et al, 1976; Krizek et al, 1977; Sivandran, 1979) that linear regression models involving more than one variable, or nonlinear models, are not justified, since they do not greatly improve the accuracy of the prediction (see also Table 7.6). Usually, the simple linear regression model with e_o can explain at least 80% of the variation of C_c or CR. If the area of applicability of the regression equation is narrowed down, this percentage even increases.

More advanced multivariate techniques, such as factor analysis (Wong & Brenner, 1979; Balasubramaniam et al, 1979) have also been carried out on groups of soil data obtained in the laboratory in connection with oedometer tests. By this technique, a grouping of variables into geotechnically meaningful categories or factors can be achieved.

A compression index equation, claimed to be universal, was recently developed by Rendon-Herrero (1980), based on the fact that soils with different e—log σ'_v curves but equal specific gravity, G_s, when plotted in terms of moisture content or void ratio versus dry density, γ_d, yield a unique relationship, namely the *zero-air-voids curve*, $\gamma_d = G_s\gamma_w/(1+e_o)$. It is then postulated that, for a given void ratio, the slope of any e—log σ'_v curve, C_c, is related to the slope of the zero-air-voids curve at that void ratio. Thus:

$$C_c = f\left(\frac{\gamma_w}{\gamma_d}\right) \tag{7.78}$$

where γ_w is the unit weight of water. This assumption implies that, at a specified void ratio, the slopes of all e—log σ'_v curves are equal. Evaluation of a large number of sets of consolidation test data (which included, however, only few soils with $C_c > 0.5$), showed that the best curve fit was of the form:

$$C_c = 0.48827\left(\frac{\gamma_w^2}{\gamma_d^2}\right)^{1.19167} \approx \tfrac{1}{2}\left(\frac{\gamma_w}{\gamma_d}\right)^{12/5} \tag{7.79}$$

Rendon-Herrero recommended that the calculation of γ_d for normally consolidated soils be based on a void ratio which is 95% of the 'initial' void ratio, e_o, determined on the soil specimen.

Table 7.6. Regression equations for compression index and compression ratio of clays

Regression equation	Correlation coefficient	No. of samples	Applicability	Reference
$C_c=0.007\ (w_L-7)$			Remoulded clays	Skempton (1944)
$C_c=1.15\ (e_o-0.35)$			All clays	Nishida (1956)
$C_c=0.256+0.43\ (e_o-0.84)$			Brazilian clays	Cozzolino (1961)
$C_c=0.0046\ (w_L-9)$			Brazilian clays	Cozzolino (1961)
$C_c=0.009\ (w_L-10)$			Normally consolidated clays	Terzaghi & Peck (1967)
$C_c=0.40\ (e_o-0.25)$	0.85	717	Clays from Greece and some parts of the United States	Azzouz et al (1976)
$C_c=0.01\ (w_n-5)$	0.79	717		
$C_c=0.006\ (w_L-9)$	0.59	678		
$C_c=0.37\ (e_o+0.003\ w_L-0.34)$	0.86	678		
$C_c=0.40\ (e_o+0.001\ w_n-0.25)$	0.85	717		
$C_c=0.37\ (e_o+0.003\ w_L+0.0004\ w_n-0.34)$	0.86	678		
$C_c=0.21+0.008\ w_L$	0.70	113	Weathered and soft Bangkok clay	Adikari (1977)
$C_c=0.22+0.29\ e_o$	0.77	113		
$C_c=0.20+0.008\ w_n$	0.77	113		
$C_c=0.20+0.008\ w_L+0.009\ e_o$	0.70	113		
$C_c=0.1882+0.3097\ e_c$	0.88		Soft Bangkok clay	Sivandran (1979)
$C_c=0.1509+0.3401\ e_c-0.0062\ e^2_c$	0.90			
$C_c=0.575\ e_o-0.241$	0.966		French clays	Vidalie (1977)
$C_c=0.0147\ w_n-0.213$	0.963			
$CR=0.0043\ w_n$			Marine clays of Southeast Asia	Cox (1968)
$CR=0.0045\ w_L$				
$CR=0.156\ e_o+0.0107\ (e_o<2)$	0.93	≈ 230	All clays	Elnaggar & Krizek (1970)
$CR=0.14\ (e_o+0.007)$	0.74	717	Clays from Greece and some parts of the United States	Azzouz et al (1976)
$CR=0.003\ (w_n+7)$	0.68	717		
$CR=0.002\ (w_L+9)$	0.53	678		
$CR=0.00566\ w_n-0.037$	0.81		Bangkok clays	Authors
$CR=0.00463\ w_L-0.013$	0.63			
$CR=0.0039\ w_n+0.013$ (for $w_n<100\%$)	0.86		French clays	Vidalie (1977)
$CR=0.403\ \log w_n-0.478$	0.86			

7.6.3.3 Probabilistic approach

The probabilistic treatment of a settlement problem (Section 7.3.10) requires the soil properties, such as the compression index, to be represented as a probability distribution (Krizek et al, 1977; Balasubramaniam et al, 1979). In Fig. 7.35, are shown the empirical frequency distributions of C_c values obtained from two narrow depth intervals, one each located in the weathered and soft Bangkok clay. A χ^2 goodness-of-fit test at a 5% significance level (see Benjamin & Cornell, 1970) was conducted to test the hypothesis of a normal distribution, and this indicated that the difference between the data and the theoretical model was not significant. This is in agreement with Krizek et al (1977), who also found the values of C_c to fit a normal distribution.

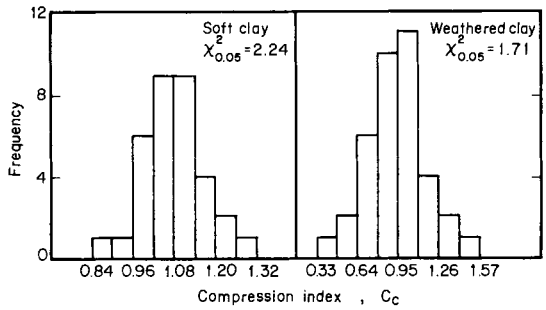


Fig. 7.35. Frequency distribution of measured compression indices of weathered and soft Bangkok clay (Balasubramaniam et al, 1979).

For a probabilistic analysis, the distribution of C_c must be known for each layer into which the soil profile has been divided. If the available data are insufficient for some layers, the problem may be overcome by making use of the coefficient of variation, V, which is defined as the ratio of the standard deviation to the mean (Kay & Krizek, 1971).

There is experimental evidence (Hooper & Butler, 1966) that, if the soil within a given soil profile is of a similar type and has a reasonably consistent geological history, the coefficient of variation will be restricted to a narrow range, even though the mean value of the parameter under consideration may vary from location to location. In Fig. 7.36, the coefficient of variation is plotted versus the mean values of the compression indices of weathered, soft and stiff Bangkok clay from three locations in the Bangkok area. Each point refers to data obtained from within a narrow depth interval. From this figure, a value of 0.18 is obtained for the coefficient of variation using a weighted averaging technique.

Based on the normal probability distribution of C_c and the coefficient of variation, the probability distribution for the mean of the compression index, $\bar{C}_c$, of any layer can then be generated from:

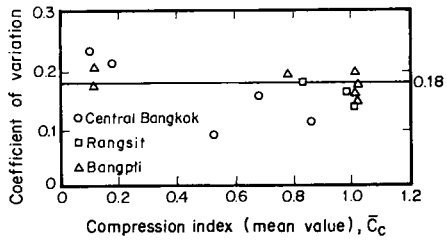


Fig. 7.36. Coefficient of variation versus mean value of compression index of Bangkok clay from three locations (Balasubramaniam et al, 1979).

$$\hat{C}_c = \frac{\bar{C}_c}{1-\frac{ZV}{\sqrt{n}}} \tag{7.80}$$

where Z is the standard normal deviate, and n is the number of samples.

7.6.4 *Coefficient of secondary compression*

7.6.4.1 *Definitions*

The coefficient of secondary compression, C_α, is defined here as the change in void ratio per log cycle of time, $\Delta e/\Delta \log t$, and appears to be the most useful quantity to describe the magnitude of secondary compression, i.e. it is least affected by testing conditions (Mesri, 1973). Another definition often found in the literature is strain per log cycle of time, i.e. $\Delta\varepsilon_v/\Delta \log t$ or $C_\alpha/(1+e)$; in this expression, the initial value, e_o, or the value at the beginning of a load increment, or the value at the beginning of the linear portion of the compression versus log time curve has been used for the void ratio, e.

The factors which influence the coefficient of secondary compression, such as time, effective stress, overconsolidation, load increment ratio, load increment duration, sample thickness, temperature, etc, have been reviewed by Mesri (1973), and some of these will be briefly discussed subsequently. Mesri also presented a classification of soils based on secondary compressibility, which is shown in Table 7.7. The values of $C_\alpha/(1+e)$ fall within a range of

Table 7.7. Classification of soils based on secondary compressibility (Mesri, 1973)

Coefficient of secondary compression, $C_\alpha/(1+e)$, %	Secondary compressibility
<0.2	very low
0.4	low
0.8	medium
1.6	high
3.2	very high
>6.4	extremely high

about 0.1 to 10%. Among the various clays, mineralogical composition and physico-chemical environment appear also to be of importance. Thus, sodium smectite has the highest coefficient of secondary compression, and kaolinite the lowest, while illite takes an intermediate position. Correlations of C_α with natural water content, initial void ratio, e_o, and compression ratio have also been made (Ladd, 1973; Mesri, 1973).

7.6.4.2 Interrelationship between C_α and C_c

An important contribution to the knowledge on C_α was made by Mesri & Godlewski (1977), who postulated, based on an empirical analysis of observed data, that for any one soil, there exists a unique relationship between C_α and C_c that holds true at all combinations of time, effective stress and void ratio. This is illustrated in Fig. 7.37 by a surface in (e, log σ'_v, log t) space.

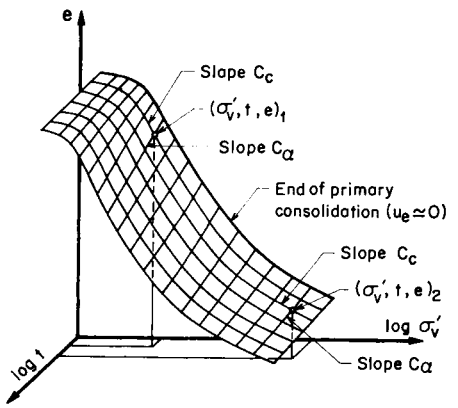


Fig. 7.37. Relationship between C_α and C_c during secondary compression in the e—log σ_v—log t space (Mesri & Godlewski, 1977).

When C_α is plotted versus C_c, an approximately linear relationship is obtained with many soils, as shown in Fig. 7.38 for Mexico City and Leda clays. Because of the stress dependency of C_c and C_α, it is important that C_c and C_α are evaluated at the same effective stress. In Fig. 7.38, C_c was taken from the e—log σ'_v curve corresponding to the end of primary consolidation, and C_α was obtained at any consolidation stress from the first linear log time cycle beyond the transition from primary to secondary compression. It is believed (Mesri & Godlewski, 1979) that the e—log σ'_v curve at the end of primary consolidation is unique, i.e. that a single curve applies to both thin laboratory specimens and clay layers in the field with much greater thickness. The values of C_α/C_c vary within a relatively narrow range, i.e. from 0.025 to 0.10, with the high values corresponding to organic soils.

It is now possible, by means of Mesri & Godlewski's postulate, to predict the variation of C_α with time. The procedure is illustrated in Fig. 7.39. An

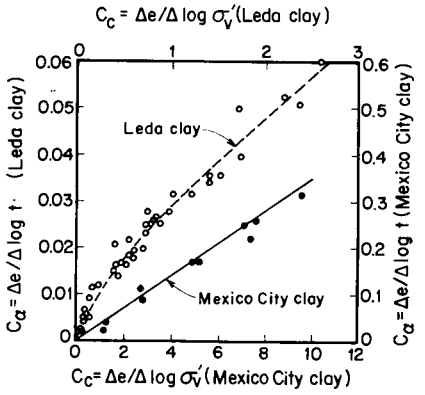


Fig. 7.38. Relationship between coefficient of secondary compression and compression index for Mexico City clay and Leda clay (Ladd et al, 1977, with data from Mesri & Godlewski, 1977).

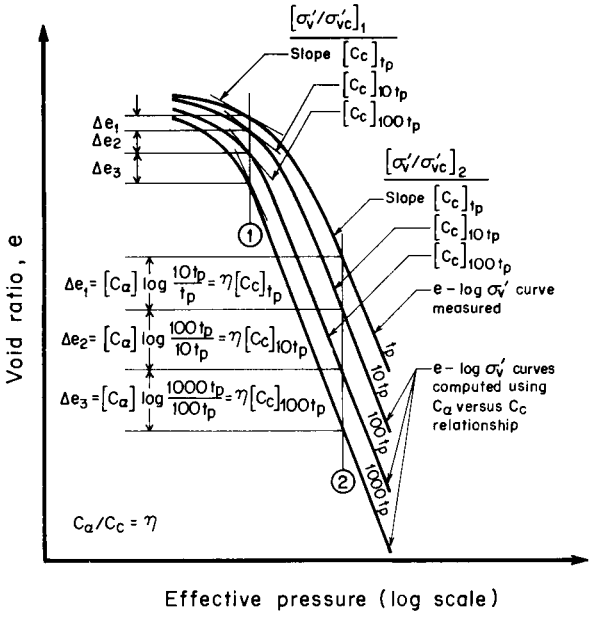


Fig. 7.39. Procedure to obtain relationship between C_α and time at various effective stresses (Mesri & Godlewski, 1977).

e—log σ_v' curve corresponding to the end of primary consolidation ($t=t_p$) is obtained first. If the relationship between $C\alpha$ and C_c is known, either from a curve of the kind shown in Fig. 7.38, or by assuming a constant $C\alpha/C_c$ value, the curve corresponding to $t=10t_p$ can be constructed at any stress, σ_v', by employing the relationship for Δe_1 given in Fig. 7.39 and assuming $C\alpha$ remains constant from t_p to $10t_p$. In a similar way, the e—log σ_v' curve for 100 t_p can be constructed from the $10t_p$ curve, etc. A family of curves obtained in this way can also be seen in Fig. 7.37.

Assuming a constant value of $C\alpha/C_c$ each for Mexico City and Leda clays, and evaluating $C\alpha$ versus time at various ratios of σ_v'/σ_{vc}', Mesri & Godlewski,

demonstrated further that C_α increases with time for $\sigma'_v > \sigma'_{vc}$, whereas it slightly decreases with time for $\sigma'_v < \sigma'_{vc}$. Note that in Bjerrum's time line plot (Fig. 7.9), C_α also varies with time, otherwise the time lines would be equidistant.

By knowing the e—log σ'_v curve at the end of primary consolidation and the C_α/C_c ratio, it is also possible to predict the shape of the compression-time curve. According to the Terzaghi theory, the compression versus log time curve exhibits an inflection point (see, for example, Fig. 7.13). During consolidation, the average void ratio and average degree of consolidation are related by (Mesri & Godlewski, 1979) $\partial e/\partial \log t = \Delta e_{100} \partial U/\partial \log t$, where Δe_{100} is the void ratio change up to the end of primary consolidation ($t = t_p$) At the inflection point, the slope $\partial U/\partial \log t$ is about 0.7, hence:

$$\frac{\partial e}{\partial \log t} = 0.7\ \Delta e_{100} \tag{7.81}$$

For pressure increments in the normally consolidated (virgin) range, i.e. $\sigma'_{vi} > \sigma'_{vc}$, $\Delta e = C_c \log [(\sigma'_{vi} + \Delta\sigma'_v)/\sigma'_{vi}]$, and Eq. 7.81 becomes:

$$\frac{\partial e}{\partial \log t} = 0.7\ C_c \log \frac{\sigma'_{vi} + \Delta\sigma'_v}{\sigma'_{vi}} = 0.7\ C_c \log (1 + LIR) \qquad (7.82).$$

where σ'_{vi} is the pressure before application of the pressure increment, $\Delta\sigma'_v$. The maximum of C_α/C_c is about 0.10 and, therefore, all LIR's which produce a $\partial e/\partial \log t/C_c > 0.1$ should yield a compression-time curve with an inflection point (S-shaped curve). For $LIR = 1$, $(\partial e/\partial \log t)/C_c = 0.21$, and the inflection point will thus be observed for all soils with $LIR = 1$. If, however, $LIR = 0.2$, for example, then $(\partial e/\partial \log t)/C_c = 0.055$, and only soils with $C_\alpha/C_c < 0.055$ will produce the S-shaped curve. Similarly, it can be shown that, for an increment that spans σ'_{vc}, the slope of the compression-time curve depends on $\sigma'_{vc}/\sigma'_{vi}$ in addition to C_α and LIR. These results should also be compared with Section 7.4.3.8.

It follows from the preceding analysis of the compression-time curves, that a classification according to shape as done by Leonards & Girault (1961) is not very practical, because many shapes of these curves are possible depending on the combination of LIR, C_α/C_c and $\sigma'_{vc}/\sigma'_{vi}$.

As a practical application, long-term field records with secondary compression settlement were analyzed by Mesri & Godlewski (1977) to show that the observations were consistent with predictions made by the theory outlined above. For example, embankment loading of a clay within the recompression range, $\sigma'_{vf}/\sigma'_{vc} < 1$, caused large secondary settlements. This observation could be explained by the fact that, for this condition, C_α increased with time.

When evaluating C_α from laboratory test data, the strong dependency of C_α on the effective stress should be remembered. Figure 7.34 illustrated the

variation of C_c with pressure. Because of the close relationship between C_α and C_c, a similar variation of C_α can be expected, with a peak in the range of σ'_{vc} to $2\sigma'_{vc}$. While it was previously thought that the load increment ratio could affect the value of C_α (because C_α depends on σ'_{vf}, and different *LIR*'s cause different σ'_{vf}), there is now enough experimental evidence (Mesri & Goodlewski, 1979) to conclude that the $C_\alpha - \sigma'_v$ relationship is unaffected by *LIR*. This conclusion is also supported by data on weathered and soft Bangkok clay published by Brand & Kanjanophas (1971) and by Moh et al (1972).

For handling secondary compression in engineering practice, Ladd (1973) suggested the following assumptions for cases in which the *LIR* is sufficiently large to cause some primary consolidation (see also Raymond & Wahls, 1976):

(1) C_α is independent of time, at least during the time span of interest.
(2) C_α is independent of the thickness of the soil layer (of course, the thicker the layer, the longer the time required for primary consolidation, but the strain or Δe per log cycle of time remains constant).
(3) C_α is independent of load increment ratio as long as some primary consolidation occurs.
(4) For normally consolidated clays with C_c=constant over the normal range of engineering stress, C_α/C_c is more or less constant.

These working hypotheses are obviously an oversimplification of actual behaviour, but they are consistent with Mesri & Goodlewski's findings, although the variation of C_α with time becomes important with long-term observations.

The C_α values defined above are strictly valid only for K_o conditions. Ladd et al (1977) presented data on the increase of the ratio of axial creep rate in drained triaxial tests to C_α from oedometer tests with the stress ratio q/q_f. At stress levels of about 0.4 to 0.6 of the strength, the ratio is around 1.0.

7.6.5 Coefficient of consolidation

Numerous methods have been proposed to evaluate the coefficient of consolidation from laboratory tests. Space does not allow a detailed treatment of all these methods here, and the reader should consult the relevant references for further information. Well-known among the existing techniques are the so-called *fitting methods* which are based on the similarity between observed compression-time curves and theoretical U_s versus T_v curves (Eq. 7.40) obtained from the Terzaghi theory. An improved comparison is possible when the time is plotted to a square root or a logarithmic scale. Widely used are the *Taylor square-root of-time* and the *Casagrande logarithm-of-time* fitting methods described in most textbooks on soil mechanics (see, for example, Taylor, 1948).

Experience has shown (Ladd, 1973) that c_v values obtained from the square-root-of-time fitting method for normally consolidated clays are typically 2 ± 0.5 times larger than those obtained by the log-time fitting

method. Ladd et al (1977) concluded that, for incremental oedometer tests, the log-time fitting method or Scott's method (see below) are better suited than the square-root-of-time fitting method.

Scott (1961, 1962) proposed a method which utilizes the ratio of compressions taking place up to different times, so that continuous compression-time readings are not required. This is accomplished by evaluating the ratio:

$$\frac{U_s(T_v)}{U_s(NT_v)} = \frac{d_s - d_t}{d_s - d_{Nt}} \tag{7.83}$$

versus T_v, where d_s is the corrected zero dial reading (Lambe, 1951), d_t and d_{Nt} are the compression dial readings at times t and Nt (N is a number >1), and T_v is the time factor (Eq. 7.39). Note that, for $N=\infty$, $U_s(NT_v)=1$. Scott provided a convenient graph of Eq. 7.83 for the rapid determination of T_v, from which c_v can then be computed (Fig. 7.40).

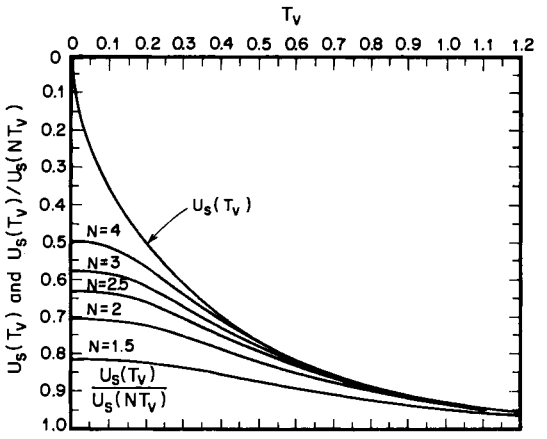


Fig. 7.40. Degree of consolidation and consolidation ratio versus time factor (Scott, 1961).

Another fitting method, developed by Su (1958), makes use of the steepest tangent to the compression versus log time plot, and is suitable for compression-time curves which do not show an inflection point (S-shape), or when secondary effects are not of interest and the straight-line tail of the curve is not needed. Su derived the following relation for the compression dial reading, d, at degree of consolidation U_s (%):

$$d_{U_s(\%)} = d_s - \frac{h_T}{0.688} U_s \tag{7.84}$$

where d_s is the corrected initial compression dial reading, and h_T is the difference in intercept of the steepest tangent over one log cycle of time.

Equation 7.84 can be evaluated for any U_s. For example, from $U_s=0.9$, d_{90} is obtained, and from the corresponding t_{90}, the c_v value can be computed.

Recently, a very efficient fitting method was proposed by Parkin (1978). He plotted the rate of compression (velocity) versus time to a double-logarithmic scale. In this way, the analysis becomes independent of initial and final conditions.

Various fitting methods are also available for determining the coefficient of consolidation for radial flow (Shields & Rowe, 1965; Berry & Wilkinson, 1969; and others).

If oedometer equipment is available which allows the measurement of pore pressure dissipation during consolidation, c_v can also be obtained directly from pore pressure measurements. According to the Terzaghi theory, the percentage of pore pressure dissipation corresponding to t_{50} and t_{90}, for example, are 27% and 84.5% respectively, and the actual times can be determined from a plot of pore pressure dissipation versus time after the end point (100%) has been established (see e.g. Crawford, 1964).

Finally, the coefficient of consolidation can also be obtained from the results of constant gradient tests (Lowe et al, 1969) and constant-rate-of-strain tests (Smith & Wahls, 1969; Wissa et al, 1971).

7.6.6 Coefficient of earth pressure at rest

Many of the methods used in settlement analysis make use of the initial stresses in the ground, which in turn requires a knowledge of the coefficient of earth pressure at rest, K_o. This quantity is commonly defined as the ratio of horizontal to vertical effective stress at zero lateral strain, i.e.:

$$K_o = \frac{\sigma_h'}{\sigma_v'} \tag{7.85}$$

The value of K_o depends on the stress history of the soil deposit, and it increases with the overconsolidation ratio.

Tests to determine K_o have been devised for both in the laboratory and in-situ. An overview of laboratory methods which have been proposed to measure K_o was presented by Abdelhamid & Krizek (1976). Emphasis today is given to the evaluation of K_o in-situ; however, considerable difficulties still have to be overcome (Tavenas at al, 1975; Tavenas, 1975a).

Often, the experimental determination of K_o, which requires special skills and equipment, is substituted by semi-empirical formulae which predict K_o from readily available soil data. Lee & Jin (1979) have collected from the literature data on K_o and other relevant soil parameters (effective friction angle, φ', overconsolidation ratio, OCR, and plasticity index, I_p) for a large number of cohesive soils. They also summarized available semi-empirical expressions for K_o, and compared them with the data collected. A selection of such formulae is given in Table 7.8.

Table 7.8. Formulae for the prediction of K_o

Formula	Remarks	Reference
$K_o=(1+\frac{2}{3}\sin\varphi')\frac{1-\sin\varphi'}{1+\sin\varphi'}$ $K_o\approx 0.9\ (1\text{-}\sin\varphi')$	Theoretical derivation	Jáky (1944)
$K_o=1-\sin\varphi'$		Jáky (1948)
$K_o=0.95-\sin\varphi'$	For cohesive soils, based on data obtained from remoulded specimens	Brooker & Ireland (1965)
$K_o=\tan^2\left(45-\frac{\varphi_{em}}{2}\right)$	$\varphi_{em}\approx\varphi_e$ (Hvorslev) and $\varphi_e=1.15\ (\varphi'-9°)$	Rowe (1957) Abdelhamid & Krizek (1976)
$K_o=0.19+0.233\log I_p\,(\%)$	Based on data published by Kenney (1959)	Alpan (1967)
$K_o=0.24+0.31\log I_p\ (\%)$	K_o determined from in-situ tests	Lee & Jin (1979)
$K_o(\text{OC})=K_o(\text{NC})\ OCR^{m_1}$ with $m_1=0.54\exp(-I_p/281)$		Alpan (1967)
$K_o(\text{OC})=K_o\ (\text{NC})\ OCR^{m_2}$ $m_2=\text{f}\ (I_p)$		Schmidt (1966) Ladd et al (1977)
$K_{o\,(\text{min})}=(0.95-\sin\varphi')$ $K_{o\,(\text{max})}=(0.95-\sin\varphi')\ OCR$		Tavenas et al (1975)

Note: NC = normally consolidated
OC = overconsolidated

K_o correlates well with the effective friction angle, φ', and Jáky (1944) derived a theoretical relationship between K_o and φ' (see Table 7.8), which later was simplified to $K_o=0.9\ (1-\sin\varphi')$, and finally to (c.f. Alpan, 1967):

$$K_o = 1-\sin\varphi' \tag{7.86}$$

Equation 7.86 has been verified to fit reasonably well data obtained in laboratory oedometer tests in which σ'_v and σ'_h were directly measured (Brooker & Ireland, 1965; Abdelhamid & Krizek, 1976; and others). Ladd et al (1977) presented a plot of K_o versus φ' with data of normally consolidated clays collected from many sources, and most of the points fall within a band of $K_o=(1-\sin\varphi')\pm 0.05$.

Since the friction angle, φ', correlates with the plasticity index, I_p, (see, for example, Kenney, 1959), K_o also correlates well with I_p (Ladd et al, 1977). Alpan (1967) established a relationship between K_o and I_p, given in Table 7.8, based on data published by Kenney (1959). A similar expression was proposed by Lee & Jin (1979).

For normally consolidated soft clays, the K_o values typically fall into the range 0.45 to 0.7. This can also be verified from the expressions given in

Table 7.8. Overconsolidation significantly affects K_o, and during one-dimensional unloading, K_o increases. For this situation, Ladd et al (1977) recommended the use of Schmidt's empirical equation (Table 7.8) with $m_2=0.41$. Upon recompression, however, the empirical formulae given in Table 7.8 overestimate K_o considerably, and K_o is no longer simply related to *OCR* (Campanella & Vaid 1972; Ladd et al, 1977).

7.6.7 Elastic parameters

Elastic parameters are required in the elastic method of settlement analysis (Section 7.3.6), but their main use is in finite element analyses, where they constitute the basic soil parameters. While the magnitude of deformation is determined by the parameters E_u, υ_u, E', and υ', an additional quantity, namely the permeability, k, (or one of the coefficients of consolidation, depending on the strain conditions, Table 7.3) is needed to compute the deformation rate. The main difficulty in the use of elastic parameters arises from the selection of numerical values, particularly for the undrained modulus, E_u, because these values are influenced by a number of factors, some of which vary even during the deformation process. This is also the reason why today, in spite of the sophisticated analysis procedures now available, predictions of settlement behaviour have in many instances remained rather poor.

In the laboratory, elastic parameters are usually determined from triaxial tests. Davis & Poulos (1963, 1968) described a testing procedure involving a two-stage dead loading test which is suitable to evaluate the parameters used in Section 7.3.6. In this procedure, the soil specimen is first consolidated under in-situ (geostatic) stress conditions. Then the stress increments corresponding to those caused by the foundation load are added, and the specimen is allowed to deform under undrained conditions. From the resulting axial strain, E_u can be calculated. Finally, by permitting drainage under unchanged total stresses, additional strains (axial and volumetric) are generated, and E' and υ' can be calculated from the total axial and volumetric strains. Furthermore, the drained stage also yields c_3 (Table 7.3), from which the coefficient of permeability can be computed.

More frequently, however, E_u is evaluated from strain-controlled unconsolidated-undrained (UU) or consolidated-undrained (CU) triaxial tests. A comprehensive study of the factors influencing E_u determined by such tests was made by Ladd (1964).

Poulos & Ahlston (1974) determined elastic parameters of kaolin by four different testing procedures, namely: (1) a one-stage dead loading test with drainage from the instant of loading, (2) a two-stage dead loading test as proposed by Davis & Poulos (1963), (3) an undrained strain-controlled test, and (4) a drained strain-controlled test. All tests started from K_o consolidation. They found that the values of E' and υ' obtained from the three tests involving drainage were in good agreement, while the values of E_u were highly dependent on the testing procedure. They recommended that the one-stage dead loading

test not be used for the determination of E_u, and that specimens be consolidated initially under in-situ stress conditions. D'Appolonia et al (1971) also stated that E_u values obtained from unconfined compression or UU triaxial tests would be much too low.

The values of E', υ' and particularly E_u are very sensitive to the initial shear stress level. Both E_u and E' decrease rapidly with increasing shear stress level. Further factors which affect mainly E_u are (Ladd et al, 1977): (1) the degree of sample disturbance (this can be overcome partly by reconsolidation to in-situ stresses), (2) consolidation stress, (3) amount of aging allowed prior to shear, (4) rate of strain (the secant modulus in CU compression tests on sensitive and plastic clays often decreases $20\pm10\%$ per log cycle of strain rate), (5) applied shear stress system, (6) stress path (Yudhbir & Varadarajan, 1975), and (7) stress history (*OCR*).

It is common to normalize the undrained modulus with respect to the undrained strength, s_u, or to the consolidation stress. The ratio E_u/s_u decreases with increasing plasticity and organic content; it also decreases with the overconsolidation ratio for $OCR>2$ (Ladd et al, 1977).

Data compiled from various projects by Simons (1974) show the value of E_u/s_u to vary between 40 and 3000. D'Appolonia et al (1971) recommended the use of E_u/s_u values between 1000 and 1500 for lean inorganic clays of moderate to high sensitivity, and considerably lower values (e.g. around 100) for highly plastic clays and organic clays. Bjerrum (1964) proposed the use of $E_u/s_u=200$ to 500, but later (Bjerrum, 1972) this range was changed to 500 to 1500, with the undrained strength, s_u, being based on vane tests.

It is probably possible to calibrate the E_u/s_u ratio of a clay for a given area and loading procedure from back analyses of observed performance of structures. Thus, for Bangkok soil conditions, a range of E_u/s_u from 70 to 250 in the soft clay has been found suitable for finite element analyses.

The drained modulus, E', can often be calculated from the theoretical relationship between E' and m_v (Poulos, 1975):

$$E' = \frac{(1-2\upsilon')\,(1+\upsilon')}{(1-\upsilon')\,m_v} \tag{7.87}$$

m_v can be determined from the compression index, C_c:

$$m_v = \frac{C_c}{1+e_o}\,\frac{\log\,(\sigma'_{vf}/\sigma'_{vi})}{\sigma'_{vf}-\sigma'_{vi}} \tag{7.88}$$

where σ'_{vi} and σ'_{vf} are the initial and final effective stresses respectively over the stress range considered. For soft clays, E' commonly ranges from 1.4 to 4.2 MPa. E' and E_u are related by:

$$E_u = \frac{3E'}{2\,(1+\upsilon')} \tag{7.89}$$

Finally, while the undrained Poisson's ratio, υ_u, is always taken as 0.5, the drained ratio assumes values from 0.35 to 0.45 for soft clays and 0.30 to 0.35 for medium clays (Poulos, 1975).

7.7 CONCLUSIONS

Various methods for the prediction of the magnitude of settlement of a structure on soft ground have been reviewed, starting from the conventional one-dimensional odeometer method which was introduced by Terzaghi almost 50 years ago. These methods involve different soil parameters and/or testing procedures to evaluate the vertical strains caused by load application. While the original one-dimensional approach only considers strains generated by an increase in the effective stresses (when the pore water is squeezed from the soil), the methods developed later improve the prediction capability by incorporating also the contributions of undrained deformations (immediate settlement) and creep (secondary compression).

Considerable progress has also been made in the prediction of the *rate* of settlement for one-dimensional problems. The most recent consolidation models do not require any of the restrictive and often unrealistic assumptions associated with the original Terzaghi theory, and they therefore enable a fairly precise prediction of laboratory test results. When applied to field data, these more advanced models allow at least a qualitative prediction and interpretation of the phenomena observed in time-settlement or excess pore pressure records. Of practical usefulness also are methods which allow the prediction of future settlements based on an analysis of the past record.

Models of two- and three-dimensional consolidation which correctly provide for a coupling between the magnitude and progress of settlement (Biot theory) are still limited in their application to research problems, and effects of lateral drainage are more commonly taken into account by an extension of Terzaghi's diffusion-type theory. Comparisons between the two approaches have revealed that the differences are never very great, and that for most practical applications, the diffusion theory is adequate.

A large number of procedures and devices are now available to evaluate soil parameters relevant to settlement analysis. While laboratory procedures still predominate, more emphasis is now given to the development of in-situ methods, because laboratory specimens are affected by sample disturbance. The forecasting of settlements still contains a number of uncertainties, and it requires considerable experience on the part of the engineer. With careful analysis of well-documented case histories, however, it is possible to calibrate several methods of prediction in a given area of soft clay.

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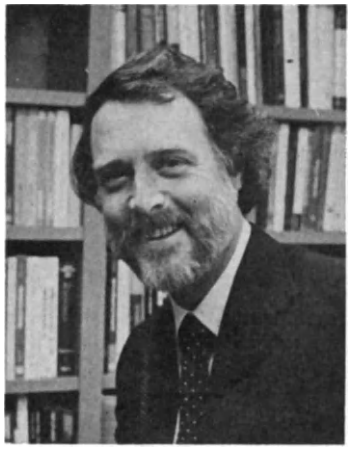
Chapter 8

DESIGN AND PERFORMANCE OF EXCAVATIONS AND TUNNELS IN SOFT CLAY

by

G. WAYNE CLOUGH and BIRGER SCHMIDT



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Dr Schmidt has worked on driven and immersed tunnels throughout the USA, and occasionally abroad. His projects include tunnels for subways in Washington, Baltimore and San Francisco, underground experimental facilities for the Stanford Linear Accelerator Center in California, and deep nuclear waste disposal in the State of Washington. He has contributed to the literature through more than 20 publications on tunnel design, construction monitoring, exploration, and the prediction of ground movements due to tunnelling.

Chapter 8

DESIGN AND PERFORMANCE OF EXCAVATIONS AND TUNNELS IN SOFT CLAY

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8.1 INTRODUCTION

Soft clay deposits are found in the environs of many of the great urban centres of the world. These soils have created interesting excavation and tunnelling problems for geotechnical engineers in Mexico City, Oslo, Tokyo, Singapore, Bangkok, San Francisco, New York, Chicago, Boston and New Orleans, to name only a few. Some clay deposits in urban areas have been written about so extensively as to become landmarks; for example, San Francisco Bay Mud or Boston Blue Clay. The first fundamental geotechnical studies of the performance of excavations and tunnels were made by K. Terzaghi and R.B. Peck based on their observations of structures in Chicago Clay.

Based on present trends, the development of the underground environment will continue and expand. New and more challenging projects are on the horizon in many cities of the world. New construction techniques are being utilized, such as the slurry wall and the slurry and earth pressure balance moles. The prospect of more extensive construction using new techniques places pressure on the geotechnical engineer to improve his technology for dealing with construction in soft clays.

8.2 DEFINITIONS OF INDICES TO SOFT CLAY BEHAVIOUR FOR EXCAVATIONS AND TUNNELS

8.2.1 Performance classification of soft clays

The important indices for the behaviour of soft clay in excavations and tunnels must be functions of the material characteristics as well as the nature of the problem. This has been long recognized in procedures such as the *Tunnelman's Classification of Soils*. In this format, a clay may be termed *firm, squeezing* or *swelling,* depending upon the state of the clay, the ground water conditions, and the size and depth of the tunnel.

The performance of a soft clay in an excavation or a tunnel depends on the ability of the clay to withstand the stresses imposed by the opening, the stress-strain response, the swelling potential, the rate at which the clay deposit will dissipate excess pore pressures, and the original state of stress in the ground. These parameters are functions of the material as well as the environment. Within this context, the methods of construction are also important. Poor construction procedures can alter the response of the clay to loads, by reducing its strength and stiffness as a result of remoulding, by increasing the swelling via adverse ground water flow patterns, and by abetting pore pressure dissipation and creep through lengthening the construction period. All of these factors can change the performance classification of a clay, perhaps from good to poor. In the following paragraphs, the factors important to the behaviour of clays in excavation and tunnelling problems, but excluding the influence of construction, are described. The effects of construction on clay response are discussed later.

8.2.2 Strength of clay relative to induced stresses

The degree to which the strength of a clay is mobilized is an important index to performance for any problem. In the case of excavations and tunnels, a very useful way to classify this effect for short-term conditions is demonstrated in Fig. 8.1. Two charts are shown, one for excavations and one for tunnels. In these charts, the cohesion of the clay (undrained shear strength), s_u, is plotted versus depth for various established reference lines which define degrees of stability of an excavation or a tunnel. The reference lines for excavations are derived from limiting values of stability numbers, as follows:

$$\text{Stability number, } N = \frac{\sigma_{vo}}{s_u} \tag{8.1}$$

in which σ_{vo} is the total overburden pressure. The relationship between s_u and excavation depth for stability is then defined in terms of limiting stability numbers as:

$$s_u = \frac{\sigma_{vo}}{N} \tag{8.2}$$

Surcharge effects may also be included by adding a term for the magnitude of the surcharge, q, to the numerator to give:

$$s_u = \frac{\sigma_{vo}+q}{N} \tag{8.3}$$

Selected reference stability numbers for excavations are given in Table 8.1. Each of these numbers defines a slope of one of the reference lines for the classification chart. Between the lines, certain types of behaviour may be expected, based on the nature of the stability criteria and past experience.

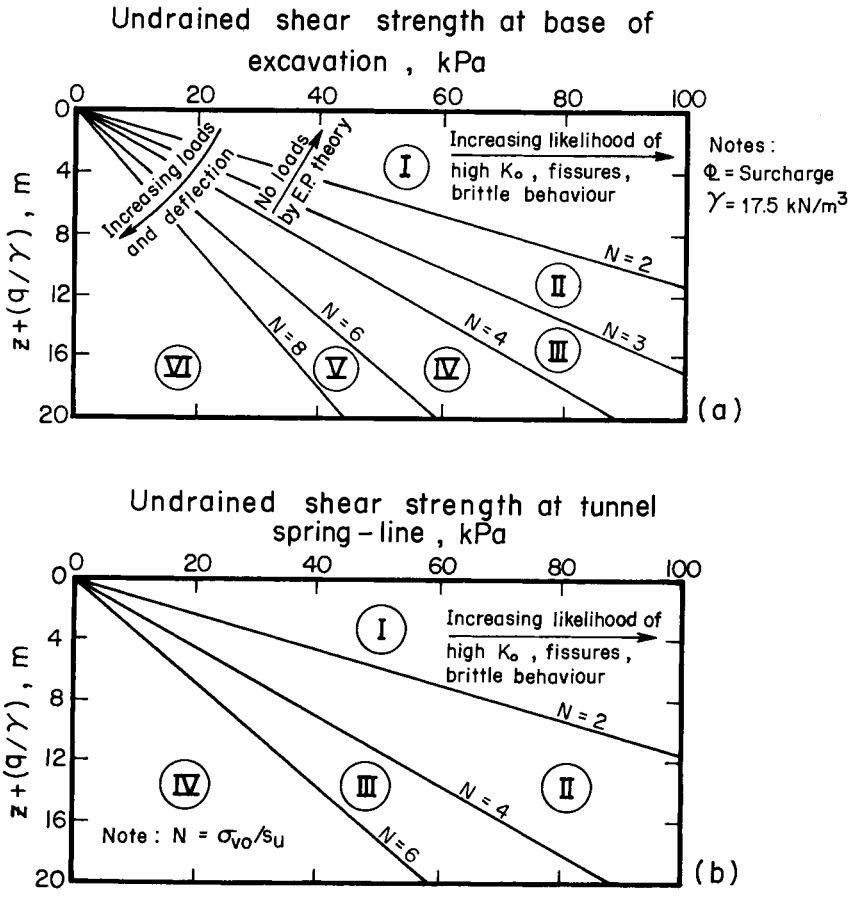


Fig. 8.1. Classification charts for: (a) excavations, (b) tunnels in soft clays. (Note that $N=\sigma_{vo}/s_u$).

These zones of behaviour are defined in Fig. 8.1. For example, zone I is everywhere above the line defined by $N=2$. In this zone, it is uncommon for any type of excavation in clay to cause problems, since the factor of safety against shear failure of even an unsupported vertical slope is greater than 1.5. Increasing zone numbers indicate increasing need for excavation support and probably more severe stability and deformation problems.

An excavation in soft clay may be classified by the zone into which it falls. The excavation is located on Fig. 8.1 by plotting the undrained shear strength at the base of the excavation versus its depth; each stage of the excavation may be considered by using the appropriate parameters at each stage. The examples shown in Fig. 8.2 illustrate important principles based on the classification chart.

Table 8.1. Limiting stability numbers for excavations

Criterion	Limiting stability no.
Vertical slope stability in brittle clay	2
Vertical slope stability in ductile clay*	4
Basal heave stability, no nearby underlying stiff layer	6
Basal heave stability, nearby underlying stiff layer	8

* This stability number also defines the strength for zero wall loadings via Coulomb or Rankine theory for no wall friction.

Three typical shear strength profiles are depicted in Fig. 8.2, one for normally consolidated clay over the entire depth, s_{u1}, one for a normally consolidated clay with a stiff overlying crust, s_{u2}, and one for a heavily overconsolidated clay, s_{u3}. The shear strength profile, s_{u1}, shows an increase typical of a low plasticity clay and is defined by the ratio $s_u/\sigma'_{vo}=0.2$, where s_u is the cohesion or undrained shear strength of the clay and σ'_{vo} is the effective overburden pressure. An excavation into this clay would fall into zone VI on the chart, regardless of depth, since its strength does not increase rapidly enough to compensate for the increase in shear stresses as depth increases. Fortunately, most clays have an overlying fill layer or stiff crust and some overconsolidation effects. The influence of a stiff crust is demonstrated by profile s_{u2}. An excavation into this soil would not be in zone VI until the depth is about 12 m or so. The shear strength profile for the highly overconsolidated clay is such that zone VI is reached only at an excavation depth of 36 m.

The classification chart for tunnels comparable to that for excavations is also given in Fig. 8.1. This chart is developed in terms of *overload factors* (identical to the stability number, N), σ_{vo}/s_u, in which σ_{vo} is the total overburden stress at the tunnel spring-line. The overload factor has been used by many investigators to correlate with observed behaviour of tunnels in clay. For soft to medium clays, experience and theory show that important demarkations in behaviour are defined by the σ_{vo}/s_u values of 6, 4 and 2. At or above $\sigma_{vo}/s_u=6$, face instability can be shown to occur; for ratios of σ_{vo}/s_u between 4 and 6, plastic yielding occurs in the clay, and early support is usually required. Values of σ_{vo}/s_u between 2 and 4 represent a zone of limited yielding, while below 2, little or no yielding occurs in the clay and the behaviour is largely elastic. Thus, critical overload factors serve to define the variation of required undrained shear strengths with depth for different degrees of stability, as do stability numbers for excavations. Lines for the limiting undrained shear strengths for overload factors of 2, 4 and 6 are shown in the tunnelling chart in Fig. 8.1. The most striking feature of this chart is its similarity to that for excavations. The basic boundaries between elastic, limited plastic and plastic

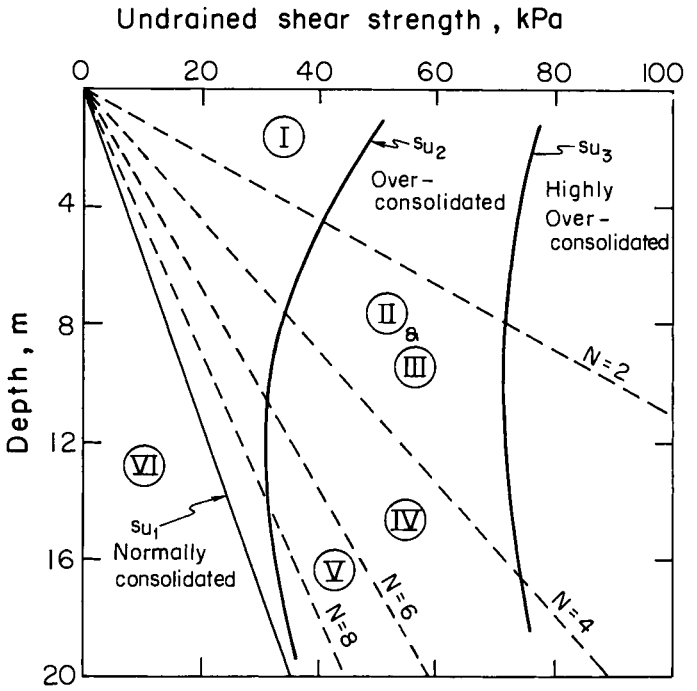


Fig. 8.2. Relationship between shear strength profiles and zones of excavation behaviour.

behaviour of the clays around the tunnel are the same as those for the excavation case. For example, an overload factor (or stability number) of 2 defines a boundary below which largely elastic behaviour occurs and deformations are small. This zone corresponds exactly with that defined for similar behaviour for excavations in clays. Similarly, in both cases a stability number of 6 defines a boundary above which significant plastic yielding occurs in the soil and large deformations are a strong possibility.

8.2.3 Stress-strain response

The stress-strain response of clay has a significant effect on the performance of an excavation or a tunnel, especially on the amount of deformation that will occur. In this chapter, clay stiffness and sensitivity are used to characterize stress-strain response.

8.2.3.1 Stiffness

The stiffness of a clay may be characterized in terms of its undrained Young's modulus. Available data show that values of this parameter are a function of the overconsolidation ratio and the plasticity index of the soil. Figure 8.3 summarizes the present state-of-the-art; modulus values are defined

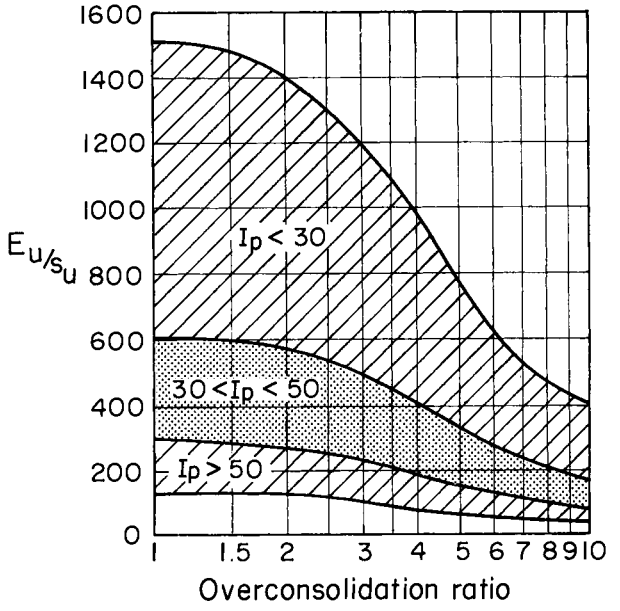


Fig. 8.3. Undrained Young's modulus as a function of undrained shear strength, plasticity index and overconsolidation ratio (Duncan & Buchigani, 1976).

as multiples of the undrained shear strength. The multiplier decreases as overconsolidation ratio and plasticity index increase.

Studies of a number of excavation case histories have shown that the values given in Fig. 8.3 can be used to make reasonable predictions of excavation deformations. Case history studies have also shown that the use of stress-strain parameters from conventional undrained test data often leads to estimations of movements that are too large because disturbance in normal testing procedures strongly reduces soil stiffness.

8.2.3.2 Post-failure behaviour

For clays, the sensitivity suitably defines post-failure response. Sensitivity is defined as the ratio of the peak strength in an undisturbed condition to the strength determined in the remoulded state. Clays with sensitivities greater than 8, termed *sensitive clays,* are subject to a substantial and rapid strength loss after failure. These materials require special consideration, since local yielding can lead to large deformations, and construction effects can cause sizeable areas of remoulding.

8.2.4 Swelling potential

Swelling of a clay by moisture uptake can lead to strength loss and extra loading of supports. Long-term slope failures and creep effects are common

in clays which exhibit swelling characteristics. All clays swell to some extent when triggered, with the higher plasticity clays ($I_p > 30$) being more likely to cause problems. Note, however, that for swelling to occur, the process must be activated by an environmental change.

8.2.5 Rate of pore pressure dissipation

The rate of pore pressure dissipation determines whether the clay behaviour will be undrained or drained and influences, therefore, the nature of the deformation or stability problem. General solutions for the rate of consolidation around excavations and tunnels are not available, although finite element procedures are being developed for such purposes (Osaimi, 1977). At the present time, only broad judgements, based on simple theories, may be made about the degree of pore pressure dissipation that will occur during excavation or tunnel construction. These judgements are influenced by the permeability of the clay and the boundary conditions. In defining the permeability and boundary conditions, the structure of the clay needs to be considered. Rowe (1969) has shown that thin layers of silt and sand in many clays lead to rapid drainage of excess pore pressures and transmission of pore pressure effects to large distances from the location of construction. This is consistent with the reports by D'Appolonia & Lambe (1971), Chang & Duncan (1971) and Parsons (1976), who have observed that excavation heave tends to occur 10 to 100 times faster than predicted from the results of tests on small oedometer samples. These results suggest that careful study needs to be given to structures in clay deposits before estimating the rate of pore pressure dissipation.

8.2.6 Initial state of stress in the ground

The initial state of stress can be important for the stability of an opening. The initial state of stress is usually defined in terms of the vertical gravity stresses and the ratio of the lateral to vertical stress. When effective stresses are used, this ratio is equal to the coefficient of lateral earth pressure at rest, K_o. A total stress version of the lateral earth pressure coefficient, K, may be defined for a deposit with the ground water-table at the surface:

$$K = K_o + (1 - K_o)\frac{\gamma_w}{\gamma} \tag{8.5}$$

where γ_w is the unit weight of water and γ is the total unit weight of soil.

The value of K_o varies with the soil strength and overconsolidation ratio. For a normally consolidated soil, K_o is empirically equal to $1 - \sin \varphi'$. The effect of overconsolidation ratio, OCR, was found by Schmidt (1966) to be:

$$K_o = (1 - \sin \varphi')\,(OCR)^b \tag{8.6}$$

where b is usually between 0.4 and 0.45.

The value of K_0 can vary between 0.6 and 1.6 for clays with *OCR* values from 1 to 10. However, in terms of total stresses, the K value will range no more than $\pm 25\%$ of unity for *OCR* values between 1 and 10. Only at very high *OCR* values does the lateral total stress coefficient become significantly greater than one. In such cases, there can be tunnelling problems or large excavation heaves, since the lateral stresses are very high relative to the vertical stresses, and passive failure in the soil can result. Thus, in overconsolidated crustal zones or layers, consideration should be given to the influence of the initial state of stress.

8.3 EXCAVATIONS IN SOFT CLAY

8.3.1 Outline of the problem

In order to design an excavation in soft clay, predictions as to basic stability, support loads and movements are normally needed. Total stress methods for predicting degree of excavation stability and support loads were developed in the 1940s by Terzaghi (1941, 1943), Peck (1943), Terzaghi & Peck (1948), Skempton & Golder (1948) and others. Since then, gradual improvements and modifications in these procedures have been made. Bjerrum & Eide (1956) provided a general solution for basal heave analysis of deep excavations. Shortly thereafter, instrumentation programmes were undertaken in the early 1960s in deep excavations in soft clays in Oslo and Mexico City. The results revealed that larger support loads than previously suspected existed where large movements occurred in the soils underlying an excavation (Peck, 1969; Rodriguez & Flamand, 1969; Bjerrum et al, 1972). New rules were created to handle this case.

The recent emphasis on movements around supported excavations reflects the fact that deformation control is often the most important function of excavation support systems. The growing body of movement data and successful application of finite element techniques to excavations provides the raw material for ongoing research. Considerable effort is now devoted to the documentation of excavation performance and the definition of damage criteria for buildings adjacent to excavations. Finite element procedures are being improved and are becoming more efficient, with the result that realistic modelling of excavations is possible. With the increased emphasis on movements, it has been realized that our conventional laboratory test procedures are inadequate to define soil stiffness, one of the key parameters in predicting movements. Improved testing techniques in the laboratory and in-situ are producing results which are having an impact on excavation system design. Finally, associated with the trend towards deformation control, new construction technologies are finding greater application. These include the slurry wall and the tied-back wall. Additional information is required, therefore, to handle the analysis of such cases.

8.3.2 Relationship between excavation stability, loads and movements of a supported excavation

The subjects of excavation stability, support loads and movements are often viewed as independent of one another. They are, of course, related, and it is useful to establish a general picture of this behaviour before examining each subject separately. This will be done within the context of a case history review and a finite element study.

8.3.2.1 Excavation for Levi Strauss Building, San Francisco

The excavation for the Levi Strauss Building in San Francisco was 14 m deep and extended through an upper 8 m thick rubble fill layer into a 12 m thick layer of soft San Francisco Bay Mud. A section through the subsoils and the excavation is shown in Fig. 8.4. The performance of the excavation

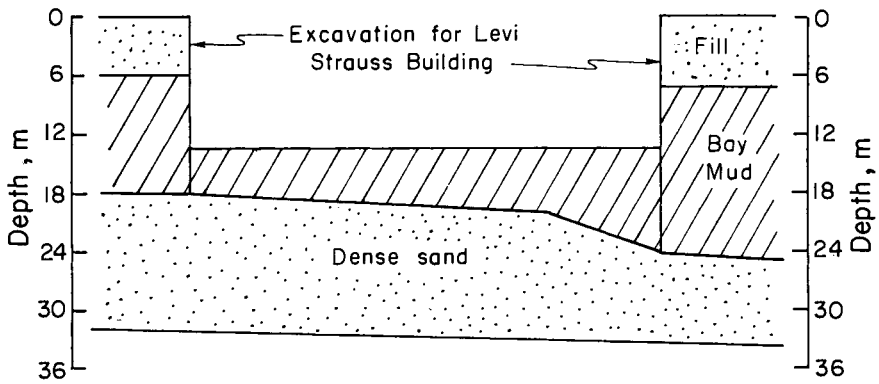


Fig. 8.4. Section through excavation for Levi Strauss Building, San Francisco (Tait & Taylor, 1974).

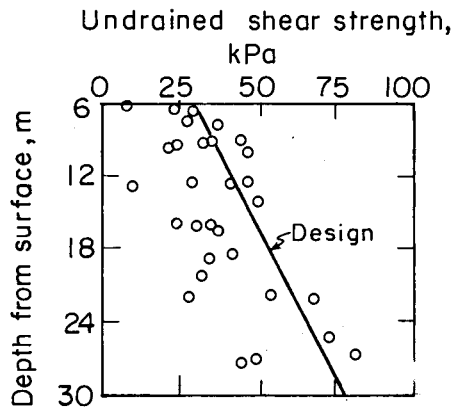


Fig. 8.5. Undrained shear strength test results on San Francisco Bay Mud (Tait & Taylor, 1974).

has been described by Tait & Taylor (1974) and provides an interesting record of behaviour in a marginally critical situation. The Bay Mud is nearly normally consolidated and has a plasticity index of 25%, a liquid limit of 60% and a water content of about 60%. Undrained shear strength test results for the soil are given in Fig. 8.5; the strength increases with depth, with a s_u/σ'_{vo} ratio of about 0.25. The line drawn on the shear strength plot represents a design strength profile provided by the consultant to the excavation contractor. Given the 14 m depth of the excavation, the stability number would be 5.5 based on this shear strength interpretation. If a low average of the data is used, a stability number of 8.3 results. The 8.3 value implies that bottom heave might

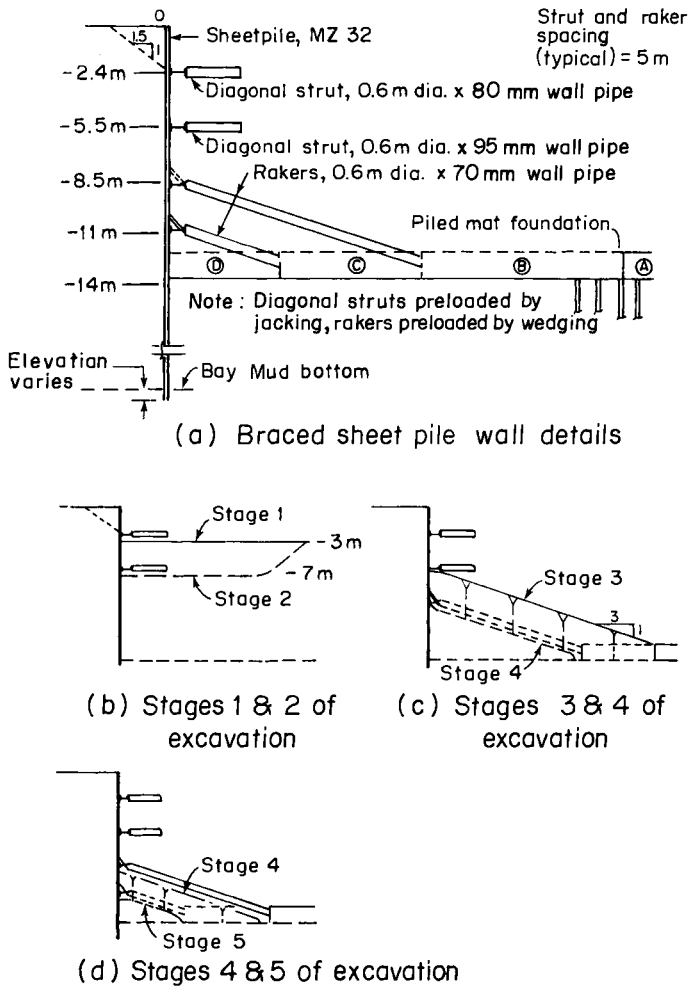


Fig. 8.6. Wall details and installation sequence for Levi Strauss Building excavation (Tait & Taylor, 1974).

occur, while the 5.5 value indicates stability against bottom heave. This situation obviously required engineering judgement about the expected performance. The resulting performance, therefore, is especially instructive.

The excavation wall was an *MZ*-32 sheet pile wall which penetrated through the Bay Mud to an underlying sand layer. Details of the wall and support system are given in Fig. 8.6. The wall was supported in the upper 5 m by two levels of crosslot pipe struts. Below these struts, the soil was removed so as to leave a 3:1 earth berm (composed of Bay Mud) against the wall. A central slab was then cast inside the perimeter established by the toe of the berm, and a level of rakers was installed on top of the berm with reaction points on the wall and the slab. The berm was subsequently removed in two stages and a second level of rakers placed when about two-thirds of the berm was excavated.

Lateral movements of the wall of the excavation were recorded by four inclinometers around the site. The maximum movements are of most interest, and these are depicted in Fig. 8.7 for various stages of excavation. In the upper part of Fig. 8.7, the profiles of movement show that only small movements ($<$ 30 mm) occurred during the 8 m excavation through the rubble fill where the wall was supported by the horizontal crosslot struts. Removal of the central area of the excavation with berm support of the wall led to up to 100 mm of additional movement. After final excavation of the berm and raker installation, the wall had deflected 180 mm. Street settlements reached a maximum of 150 mm in the locality of the large wall movements.

The lower diagram in Fig. 8.7 gives a plot of the maximum movement and factor of safety against bottom heave of the excavation versus time. The factor of safety against bottom heave was calculated using the design shear strength profile of Fig. 8.5. The maximum rate of movement occurred during the excavation for the centre slab while only the earth berm supported the wall. This maximum rate also corresponds to the stage at which the factor of safety against bottom heave ranged from values of 1.4 to 1.3. For this range of factor of safety, D'Appolonia & Lambe (1971) have shown that local yield becomes significant in normally consolidated clays. The observed movement data suggest that an important transition in behaviour occurred at the Levi Strauss Building site when local yield became significant. The excavation was apparently stable against failure, but the margin was not large. A clearer picture of the effects of subsoil yielding can be demonstrated by the results of a finite element analysis, described in the following paragraphs.

8.3.2.2 Effect of yielding

The problem chosen for the finite element study is a 12 m wide excavation, 15 m deep, into a 30 m thick homogeneous clay with a shear strength profile defined as $s_u = 8.83 + 0.2\sigma'_{vo}$ (kPa). The stress-strain response is taken as perfectly elastic-plastic. The excavation is assumed to be supported by a crosslot braced sheet pile system with the sheet pile wall penetrating the full

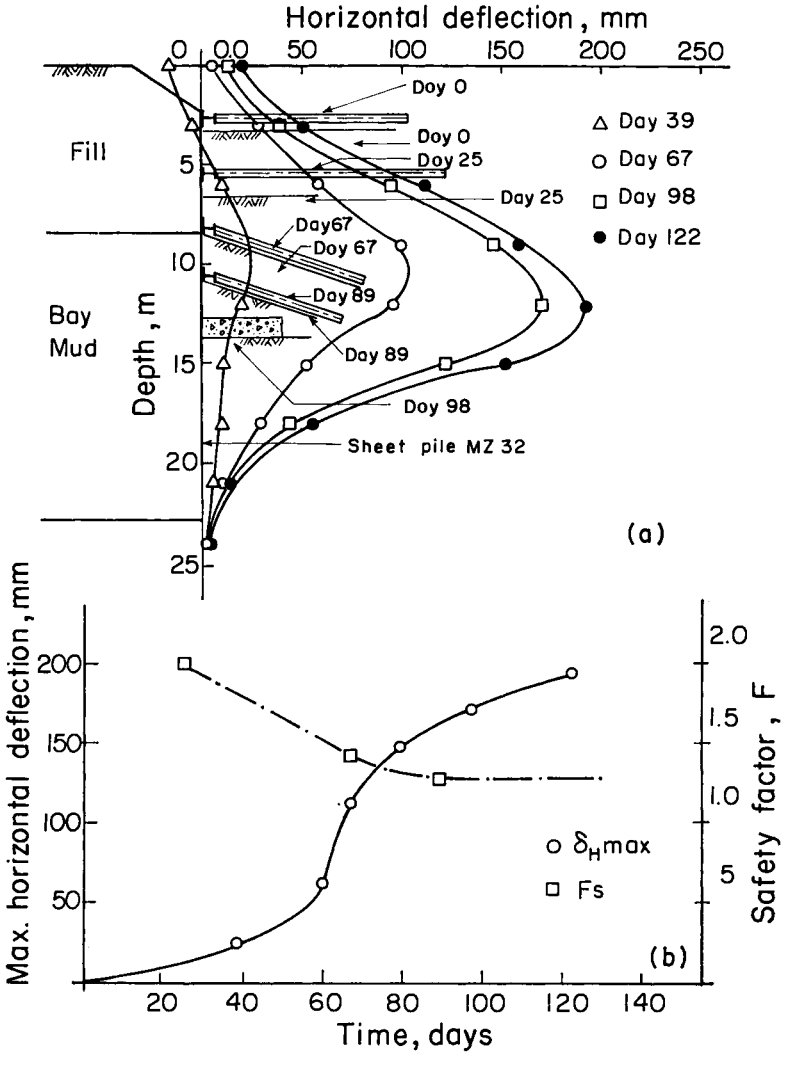
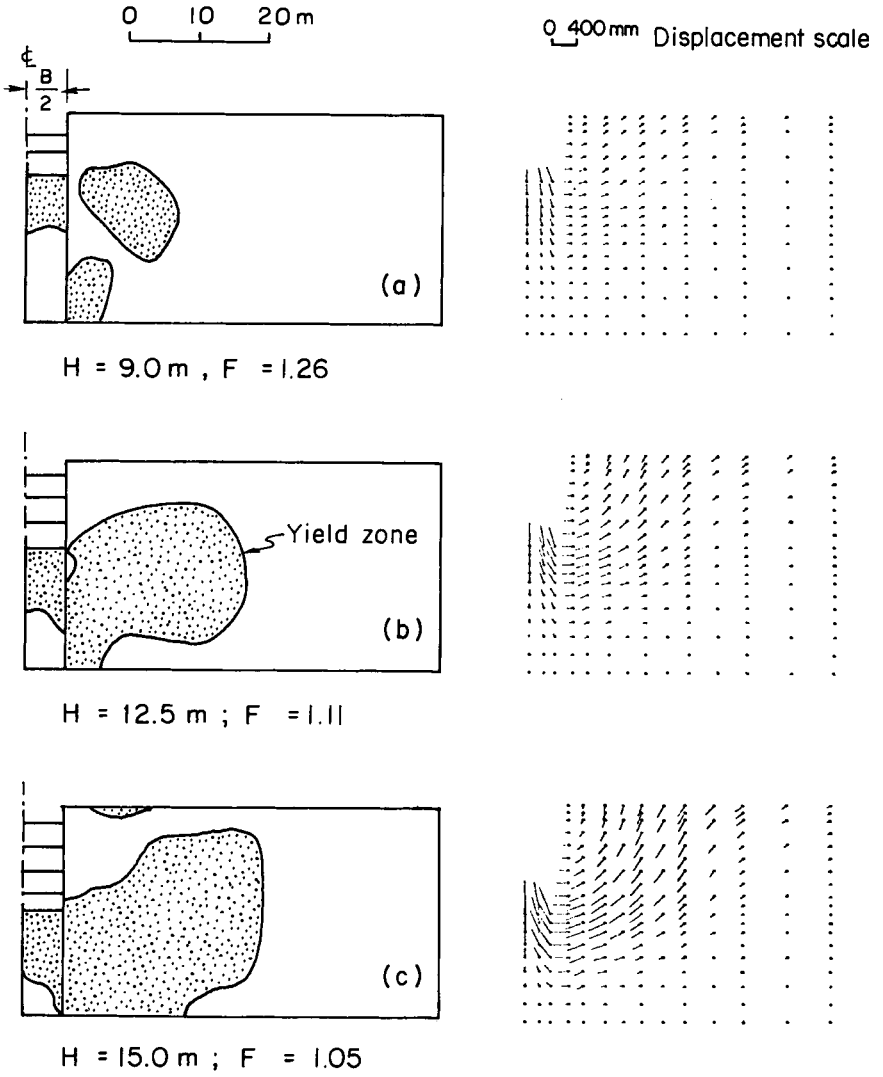


Fig. 8.7. Performance of excavation for Levi Strauss Building: (a) variation of horizontal deflection with depth for various stages of excavation, (b) variation with time of maximum movement and factor of safety against bottom heave (Tait & Taylor, 1974).

depth of the clay stratum and pinned at its base. The factor of safety against basal heave theoretically reaches 1.0 at an excavation depth of about 12 m (neglecting any stabilizing effects of the wall).

The predicted zones of yielding and soil displacement vectors for excavation depths of 9, 12.5 and 15 m are shown in Fig. 8.8. Significant local yield occurs at the depth of 9 m, where the factor of safety is about 1.3. Local yield is first observed at a depth of 5 m when the factor of safety is about 1.5. As the



H = Excavation depth
B = Excavation width = 12 m ; Strut stiffness = 17,800 kPa
F = Factor of safety against bottom heave;
$s_u = 8.83 + 0.2\,\sigma'_{vo}$ (kPa) ; $\gamma = 17.8$ kN/m^3 ; $E = 300\,s_u$
Sheet pile wall PZ 38 driven into firm layer (hinged)

Fig. 8.8. Displacement and yielding patterns for deepening excavation in soft clay (Mana, 1977).

excavation is deepened below 9 m, the yielding zones grow. Displacement vectors show increasing trends of soil movements below the excavation bottom as the yielding zones get larger. These patterns of movements are obviously linked to the tendency for bottom heave. The incremental movement for each excavation step also become larger as the bottom stability decreases.

Strut loads corresponding to each excavation stage are plotted in Fig. 8.9. Several trends are important. First, the strut loads increase strongly as the excavation is made immediately below the previously installed strut. The rate of increase is larger as the excavation gets deeper and the subsoil yielding becomes more prominent. As the subsoils yield, more load must be carried by the struts, and arching occurs in the sense described by Bjerrum et al (1972). The large inward movements of the sheet pile below the excavation bottom cause the lateral stresses in this area to decrease and to be transferred upwards towards the stiff strut supports. The loads in a strut increase very little or decrease slightly after the lower struts have been installed. Thus, the effect of a given excavation stage is very local and most significant at the next adjacent strut level.

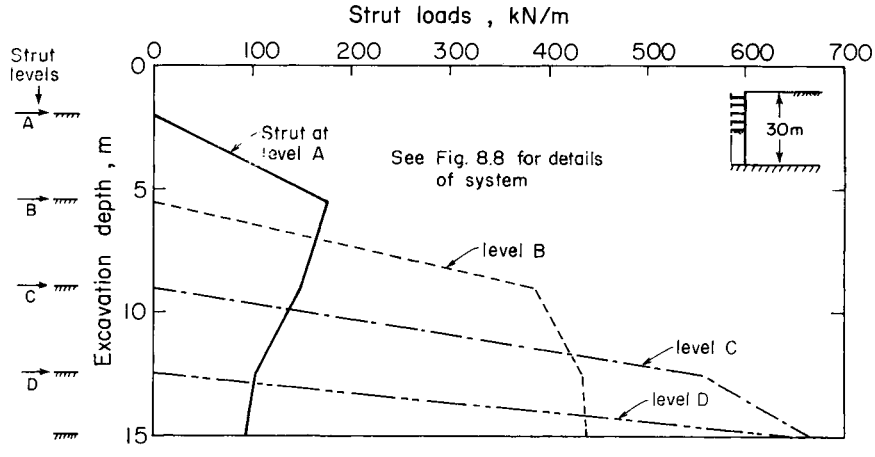


Fig. 8.9. Strut loads for a deepening excavation (Mana, 1977).

The Levi Strauss excavation and finite element data demonstrate the integral relationship among the stability, support loads and movements of an excavation in soft clay. Until the excavation stability reaches a level where local yield begins, movements are small and largely lateral, and strut loads are small. Once a significant zone of local yield has developed, movements become large and more directed beneath the excavation bottom, and each succeeding strut level is subjected to increasingly larger loads.

8.3.3 Excess pore pressures around excavations

The conventional approaches to analysis of short-term behaviour of excavations in clay assume undrained response for the soil and no excess pore pressure dissipation. It is pertinent to ask under what conditions the undrained condition does, in fact, prevail. The early evidence in support of the undrained approach consisted primarily of documentation of failures which were reasonably predicted by $\varphi = 0$ or total stress analyses (Skempton & Golder, 1948; Bjerrum & Eide, 1956). Such back calculations are not definitive proof, however, and do not establish the length of time for which the undrained condition holds.

More recently, pore pressure measurements in the clay soils behind a number of supported excavations in Oslo, Norway and Boston, Massachusetts have been reported by Lambe (1971) and DiBiagio & Roti (1972). Most of these measurements were complicated by the fact that dewatering was used to relieve pressure in underlying sands and pervious rock layers. They suggest that the undrained conditions exist for a short period but that significant dissipation of excess pore pressures around excavations can occur, especially during prolonged periods of construction within the excavation, and where the clay contains sandy and silty lenses.

Additional insight into this problem is provided by the results of finite element analyses of consolidation around excavations reported by Osaimi & Clough (1979). A simple case of one-dimensional excavation in homogeneous clay was studied where consolidation during the removal of soil was taken into account. The permeability of the soil and the rate of excavation were varied from analysis to analysis to study their influence. The average degree of consolidation at the end of excavation is plotted versus excavation rate in Fig. 8.10 for all cases of soil permeability. For the soil with a permeability of 1×10^{-10} m/sec, essentially zero consolidation occurs for even the slowest soil removal rate of 0.03 m/day. Thus, for this example, undrained behaviour is appropriate for analysis of the end-of-construction state. For the higher soil permeabilities, the degree of consolidation is strongly a function of the rate of excavation, with significant drainage indicated for several cases. In the field, the presence of drainage lenses of sand and silt will probably speed up consolidation over that shown in Fig. 8.10. The findings indicated by the results in Fig. 8.10, and those from the limited field observations, suggest that both undrained and drained conditions may need to be considered in design analyses for short-term excavations.

8.3.4 Stress state surrounding an excavation and its effect on soil response

Recent developments in our understanding of soft clay behaviour have shown that the loading history or stress path to which the soil is subjected is important in the definition of soil parameters to be used in analysis. Bjerrum et al (1972) and Ladd & Foott (1974) suggested procedures of soil testing

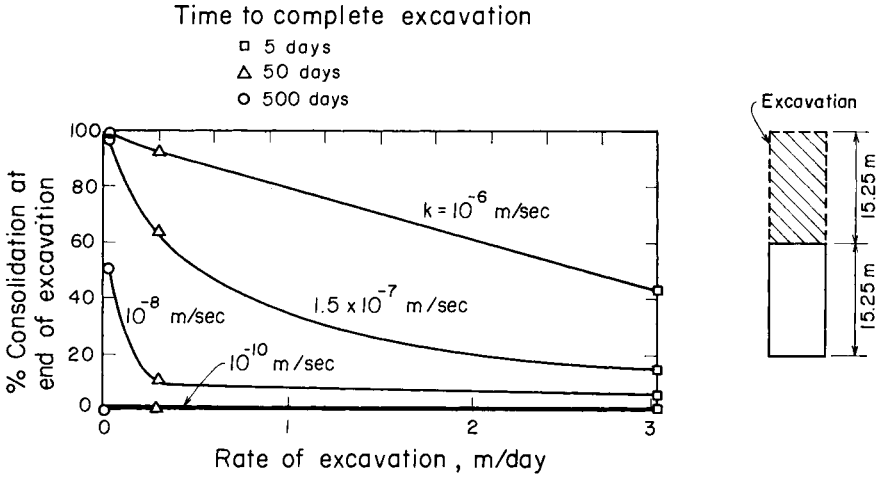


Fig. 8.10. Degree of consolidation at the end of one-dimensional excavation into homogeneous clay as a function of excavation rate and permeability (Osaimi & Clough, 1979).

and property interpretation that account for the loading history of excavations. Lambe (1970) and Henkel (1970) presented stress paths which are followed by soil elements retained by and in front of excavation support walls, and Tsui (1974) described the stress path for soils supported by preloaded wall systems. Some important findings of these studies may be summarized as follows:

(1) The soil behind the excavation wall (the active side) is loaded to failure by a reduction in lateral stress. At failure, the major principal stress is in the vertical direction and the minor principal stress is in the horizontal direction.

(2) The soil remaining on the excavation side of the wall (the passive side) is loaded to failure as the horizontal stress becomes larger than the vertical stress. At failure, the horizontal stress is the major principal stress and the vertical stress is the minor principal stress.

(3) Clays loaded under undrained active conditions demonstrate up to twice the strength of clays loaded under passive conditions.

(4) When preloads are applied to a wall during excavation, the shear stresses in the soil are reduced. The shear stresses are alternately increased and reduced by the application of excavation and preload steps respectively. The result of this loading sequence is that soil stiffness is momentarily increased during and after preloading, since an unload-reload type of phenomenon is occurring.

The essence of these findings is that the conventional procedures for determining soil shear strength and stiffness need modification for use in the

design of excavations. The deterrent to performing more accurate tests lies in the extra cost and the fact that considerable experience has been correlated with the older approaches. It is likely, however, that more appropriate testing procedures will be used in the future in those projects whose scope and sensitivity justify a more sophisticated design approach. The establishment of a more rational basis for the analysis of excavations in soft clay must incorporate such important concepts.

8.3.5 Evaluation of the stability of excavations in soft clay

In this chapter, comments on unsupported excavation stability will be brief, since this subject has been reviewed thoroughly in recent state-of-the-art papers. More detail will be devoted to supported excavations.

8.3.5.1 Unsupported excavations

Methods for evaluating the stability of unsupported excavations in soft clay are well established and are readily available via charts and computer programs. A review of case histories involving analyses of excavations in soft clay was presented by Bjerrum (1973), who concluded that the conventional methodology predicted behaviour reasonably well except for a few well-defined instances. Exceptions to the rule were found for high plasticity clays ($I_p > 50$) and for cases where stiff, desiccated crustal layers overlie soft clay. Difficulties with high plasticity clays occur because their strength is strongly time-dependent, and short-term laboratory tests do not adequately account for this factor. As a result, failures occur in instances where calculated factors of safety are as great as 1.5. A rate correction factor was proposed to reduce the conventional laboratory strength results to a field-type shear strength. Ladd & Foott (1974) suggested a testing procedure which uses slow rates of loading to account for differences between laboratory and field strengths.

In clays with stiff, desiccated crustal layers, experience has shown that the supporting effect of the crust should be neglected. The presence of a water-filled crack in such a crust should be considered in the stability analysis for even short-term conditions (Aas, 1971).

8.3.5.2 Supported excavations

The stability of a supported excavation in clay involves a variety of problems. Modes of instability are indicated in Fig. 8.11. These involve possible failures in berms and slopes around the excavation, basal heave into the excavation bottom, soil wedge slippage behind a cantilever wall, and deep seated rotational slump-type failures behind and beneath the wall. Often a stability failure does not result in complete collapse, but rather large movements are observed. Very commonly, the failure is local and is triggered by a surcharge load or a small excavation for a building footing which is made next to the wall or at the toe of a berm supporting the wall (see Clough & Davidson, 1977).

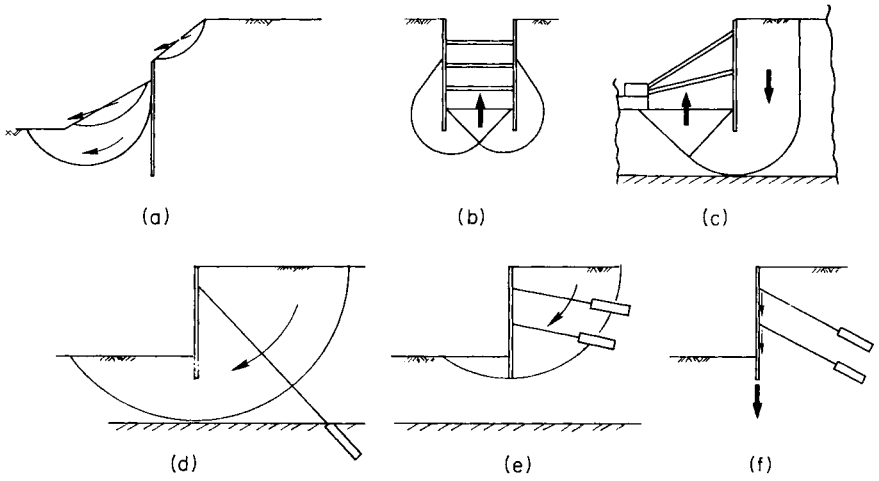


Fig. 8.11. Modes of instability for supported excavations in soft clay.

Methods of analysis. The analysis of the stability of a supported excavation in clay depends upon the mode of failure likely to develop. For a simple slope in or above the excavation, or for possible deep slump-type failures, conventional slope stability analysis procedures are used. For basal heave, the techniques shown in Fig. 8.12 are recommended. Two types of analysis are described in the figure. The first, proposed by Bjerrum & Eide (1956), is applicable for deeper excavations ($H/B > 1$) with no nearby underlying stiff base which inhibits failure. This approach neglects the effect of wall penetration beneath the excavation bottom and can be excessively conservative if the wall toe is embedded in an underlying stiff layer and the wall has enough capacity not to fail under the action of the deep-seated soil movements. The second type of basal heave stability analysis shown in Fig. 8.12 was developed by Terzaghi (1943) and is applicable primarily to shallow ($H/B < 1$) or wide excavations.

Failures of supported excavations. Instructive studies of stability failures of structurally supported excavations in soft clay have been described by Bjerrum & Eide (1956), Broms & Stille (1976) and Clough & Davidson (1977), with each investigation concentrating on different types of problem. Bjerrum & Eide (1956) examined classical basal heave case histories of strutted excavations. Their examples consisted primarily of very deep and narrow excavations (width $\approx$ 1.5 m) where the soil clearly heaved into the bottom of the cut. Broms & Stille (1976) reviewed a series of tied-back (anchored) wall failures, only two of which could be ascribed to deep rotational movements. In both cases of deep failure, the walls were supported by only one set of anchors, and the toe of the walls were embedded in the moving soil mass. Thus, the walls offered no structural restraint against the failure mass. The other failures

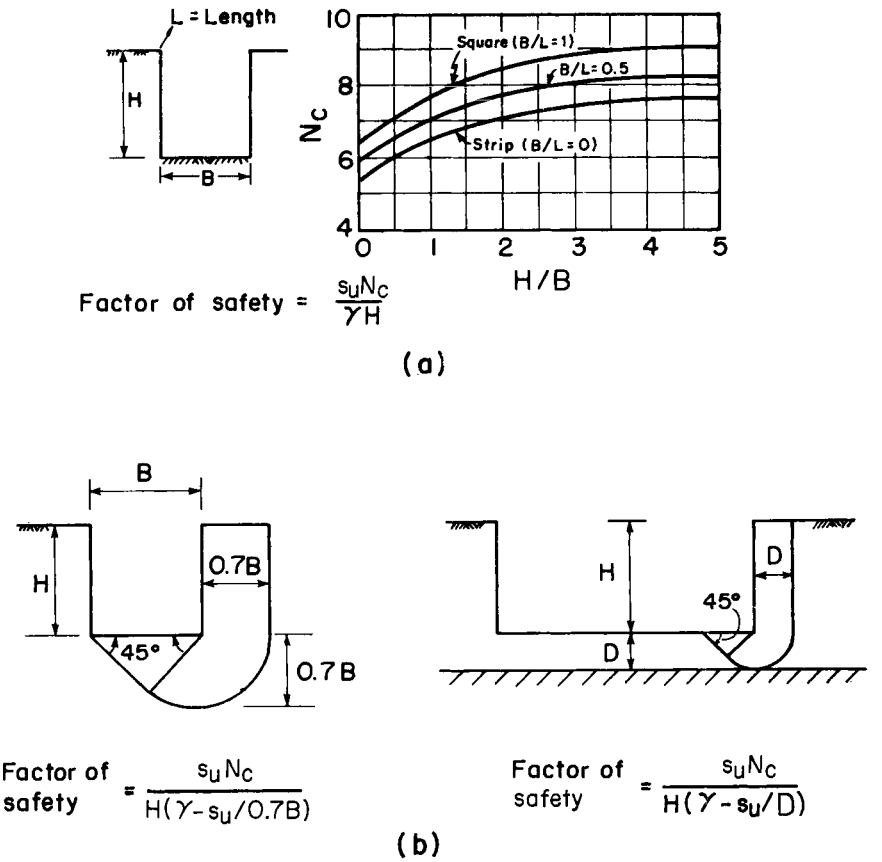


Fig. 8.12. Methods of basal heave analysis: (a) for deep excavations with $H/B > 1$ (Bjerrum & Eide, 1956), (b) for shallow or wide excavations with $H/B < 1$ (Terzaghi, 1943).

described by Broms & Stille (1976) were induced by either frost action in the soil behind the wall or by construction incidents such as pile driving next to the wall. Pile driving next to an exposed excavation wall in soft clay should be avoided wherever possible because of remoulding effects and loss of soil strength, and because of the likelihood of soil heave. The driving of piles with followers through the overburden before excavation is the preferred practice.

Clough & Davidson (1977) described a series of failures of earth berm-supported walls in soft clays. Problems with berm-supported walls are not uncommon, since the berms are highly stressed by outward wall movements, and they are easy targets for removal at the wrong time by an overzealous or uninformed contractor. Also, the effectiveness of a berm can be overestimated where deep-seated movements are likely. In this case, the berm is simply carried upward with the basal heave effect.

From all of the stability failures that were reviewed by each of the published studies, there emerge two categories, namely: (1) those that could be predicted using the established design methodology, and (2) those induced by construction or environmental circumstances. There is about an equal number in each category. It is apparent that good engineering design practice can minimize chances of failures, but careful construction practice and monitoring procedures are also required to secure the best odds against failure.

8.3.6 Loads on support systems in soft clay

According to classical earth pressure theory concepts, the loads on support systems in clay are functions of surcharge effects, unit weight, shear strength of the soil, and height of the excavation. Although these factors indeed play significant roles in determining loads, there are many other important parameters to be considered, such as overall and local system flexibilities, degree of stability against basal heave, amount and stiffness of the soil beneath the excavation bottom, degree of preloading, construction procedures, degree of seepage pattern development, frost heave in the soil behind the wall, and temperature. In evaluating the relative importance of the factors that affect loads, a distinction should be made between those that influence the distribution of the load and those that influence the magnitude of the resultant load.

8.3.6.1 Magnitude of the system load

In this chapter, the term *system load* is used to describe the resultant of the loads on the active side of the wall which act against exposed and embedded portions of the wall support system. This can be illustrated by Fig. 8.13, where the earth pressures predicted by finite element analysis for an excavation support system in clay are shown. The excavation is the same as that described earlier. Yielding zones, displacements and strut loads were given in Figs. 8.8 and 8.9. For the sake of reference, it should be recalled that local yield in the subsoil first occurs when the excavation is at a depth of 6 m, and that at a 15 m depth substantial yielding occurs.

The earth pressures on the active side of the wall for stage 2, following excavation to a depth of 5.5 m below the first strut level, are essentially linear and about equal to the classical Rankine values. This is not surprising, since the wall deflection profile in Fig. 8.13 shows lateral movements all the way to the bottom of the wall, movements which are in the order of those needed to produce Rankine conditions ($\approx 1\%\ H$). At stage 3, the earth pressures around the strut levels are higher than Rankine, but lower than Rankine below the struts to a depth of 26 m, whereupon the pressures again exceed the Rankine values. At this stage, deflection of the sheeting below the last strut level produces an arching effect. After final excavation to stage 5, the trends at stage 3 are even more prominent, showing increased pressures on the exposed wall portion and decreased pressures on the embedded wall portion. Inter-

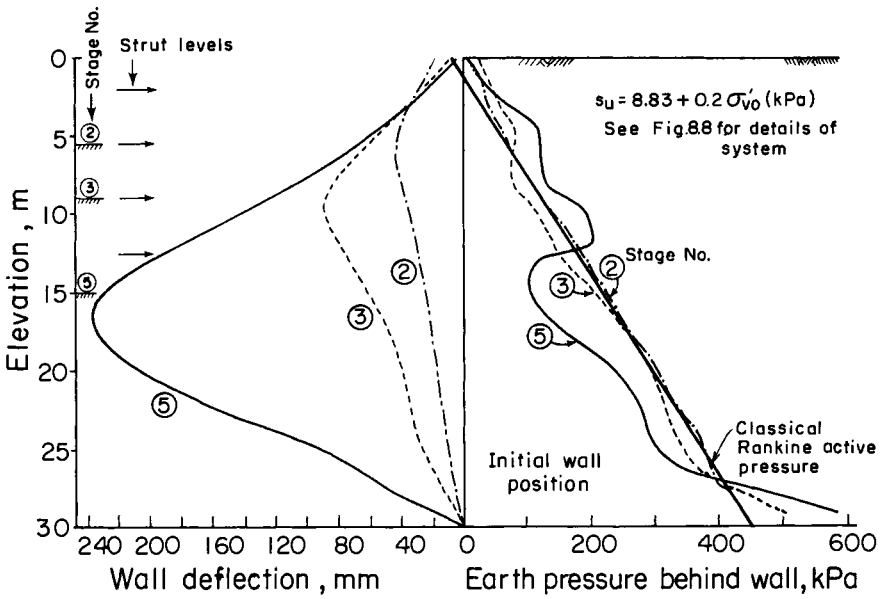


Fig. 8.13. Earth pressures and wall deflections predicted by finite element analysis for deepening excavation in clay (Mana, 1977).

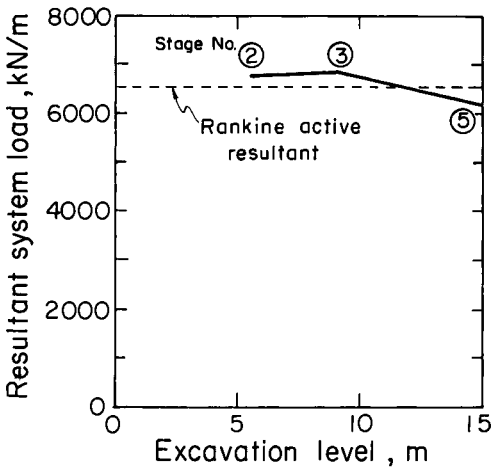


Fig. 8.14. Resultant of earth pressure acting on exposed and embedded wall portions of example shown in Fig. 8.13 (Mana, 1977).

estingly, in spite of the differences in pressure profiles at stages 2, 3 and 5, resultants of the pressure diagrams along the full length of the wall are about the same. This is illustrated in Fig. 8.14, where the resultant is plotted versus excavation stage. It can be seen that the resultants are all about equal to the classical Rankine resultant for the entire depth of the wall.

These results show that the system load is essentially a fixed quantity when the wall movements are large enough to induce the Rankine state over the entire wall length. The results also show clearly that the distribution of the load is strongly influenced by the flexure of the wall below the last strut level and the degree of stability of the excavation. The larger the movements in the basement soils, the stronger is the arching effect whereby the lateral stresses below the excavation are reduced and the stresses on the strutted wall portion are increased. To determine if a system will in fact move enough to mobilize the Rankine condition over the full wall length, an estimate of system movement is needed. Procedures for this task are described in Section 8.3.7.

8.3.6.2 Strut load diagrams

Strut load diagrams are design devices intended to provide an estimate of the maximum load which might be expected to occur on the struts of a broad excavation. They reflect the loads on only the exposed portion of the wall and are not actual pressure distributions but envelopes to the maximum values expected. The most widely used versions are those proposed by Terzaghi & Peck (1948, 1967). Since the principle of these diagrams is well described by Peck (1969), no detailed description will be given herein.

It is of interest to consider the evolution of strut load diagrams for soft clays. The different versions of the Terzaghi-Peck diagram published in 1948, 1969 and 1974 are shown in Fig. 8.15. In 1948, only one diagram was provided for soft clays, and the maximum pressure ordinate was $\gamma H - 4s_u$; the resultant of this diagram exceeds the Rankine resultant by about 50%. By 1969, experiences in Mexico City and Oslo had shown that especially large strut loads could occur where basal stability was a major problem, and two situations were considered for soft clays. Where the stability number of the cut, $\gamma H/s_u$, was 6 or less, the recommended diagram was essentially the same as the 1948 version except that the shape was slightly changed. Where the stability number was greater than 6 and a large depth of clay remained beneath the excavation bottom, a larger diagram with a maximum ordinate of $\gamma H - 2s_u$ was recommended. These larger loads reflected the exaggerated arching effects which occur when basal movements become significant. Further consideration of observed data led Peck in 1974 to modify the load diagrams again. In this version of the diagram, two explicit possibilities are given, as long as the basal stability number, N_b, is less than 7 (see Fig. 8.15). In a design with $N_b < 7$, the load diagram which gives the largest resultant is recommended. Should N_b exceed 7, no particular diagram was suggested in the 1974 version, instead a

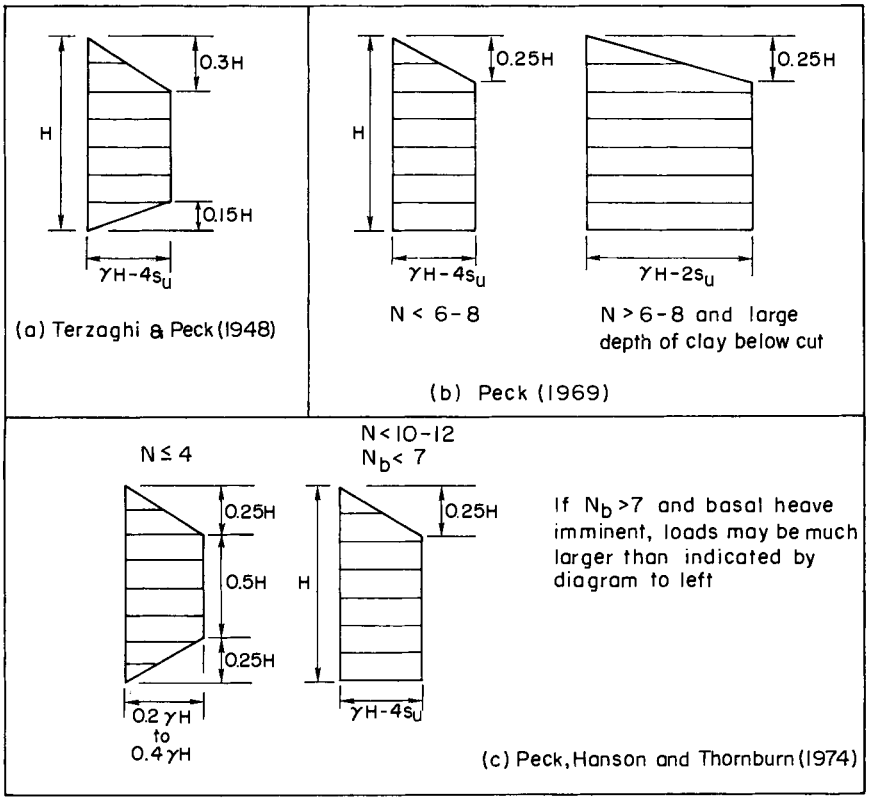


Fig. 8.15. Evolution of strut load diagrams for soft clays. (Note that $N = \gamma H/s_u$ and $N_b = \gamma H/s_{ub}$, where s_u and s_{ub} are the undrained shear strength of the clay in the cut walls and below the base respectively).

warning was given that the loads could be considerably greater than those indicated by either of the two explicit diagrams.

The fundamental reasons for the increases in strut loads when movements occur in the clay below the excavation were depicted in Fig. 8.13. In design, the major question confronting the engineer is when to apply the larger strut loads. If soil strengths are determined by tests in which disturbance is significant, the strengths might be lowered enough to indicate that the large strut loads are applicable. Because the larger strut loads obviously apply only in exceptional cases, it behoves a designer to be aware of local experience and not rely exclusively on calculations of stability numbers.

8.3.6.3 Deviations from the Terzaghi-Peck loadings

It was recognized by Terzaghi & Peck (1969) that the strut load diagrams set forth for soft clays do not apply to all conditions. There are, in fact, several notable exceptions.

Excavation carried to rigid base. If an excavation is made in soft clay and it bottoms out in a stiff material, then at the final depth there will be no load redistribution effects due to arching, and the system and strut loads will be one and the same. In this case, the loads will be those calculated by Rankine or Coulomb theory, and they will be distributed in a triangular fashion. Loads of this type have been measured by Skempton & Ward (1953) and Humphreys (1962).

Tied-back walls. Tied-back walls are not often employed in soft clays except where the anchorage for the tie-backs can be achieved in an underlying stiff layer. Case histories of this type of system are described by Hanna & Seeton (1967), McRostie et al (1972), Murphy et al (1975) and Broms & Stille (1976). The distinction between a tied-back wall and a strutted wall is two-fold: (1) tie-backs, such as high strength cables or rods, are 10 to 20 times more flexible than typical struts, and (2) tie-backs are usually preloaded to higher levels than struts. The flexibility of rod or cable tie-backs is such that the wall will move outward in response to any attempt by the soil to increase pressures on the exposed wall by arching. Thus, the high load levels observed for struts in clays will not exist with flexible tie-backs.

Most tie-backs are strongly preloaded, and this can change the behaviour pattern. If the tie-backs are preloaded with those loads recommended by Terzaghi & Peck (1948), or slightly higher, the tie-back loads will remain essentially unchanged unless major stability problems occur. This has been confirmed by analytical procedures as well as by field measurements (see Hanna & Seeton, 1967 and Clough & Tsui, 1974). It appears that any tendency towards development of Rankine type loads is counter-balanced by the preloads, which are larger than the Rankine loads if the Terzaghi-Peck magnitudes are used. The justification for the use of the Terzaghi-Peck preload values has to do with the reduction of movements, as described in the next section of this chapter.

8.3.6.4 Loads on elements other than struts or tie-backs

Struts and tie-backs are the supports for an excavation wall system and, as such, they receive the highest loads, since earth pressures tend to concentrate around support points. This has long been recognized by designers of wall systems, and it is common practice to design walls, walers and lagging for loads about 70% of those indicated by the strut loading diagrams (see Peck et al, 1974). Several aspects can affect this reduction. One is the construction procedure. White (1958) reported significant differences in loads on lagging in soft clay depending upon placement of the lagging behind the back or front flange of a soldier pile. Placement of the lagging behind the back flange leads to higher loadings, since arching effects in the soil are destroyed by excavation behind the flange for the lagging. A second aspect relates to tied-back walls, which are usually spaced more closely than struts, so much so that arching between the tie-backs can be quite small. Clough & Tsui (1974) showed that

support spacings closer than 1.5 m for typical sheet pile walls reduce arching effects.

8.3.7 Movements of excavation support systems in soft clay

The major factors which affect the movements of the ground and the support system adjacent to an excavation are as follows:

(1) type of support system,
(2) stiffness of support system,
(3) degree of wall embedment,
(4) degree of preloading,
(5) method of support system construction,
(6) time period for construction,
(7) methods of construction of structures within excavation,
(8) size of surcharge loads,
(9) weather,
(10) subsoil condition and properties,
(11) surrounding structures, and
(12) excavation shape and depth.

Perhaps the most interesting fact derived from this list is that many of the variables which influence performance are only partially or marginally under the control of the designer or the builder. Because there are important parameters which can cause movements to vary but which cannot be foretold, there are limits to the accuracy of any movement prediction for excavation support systems.

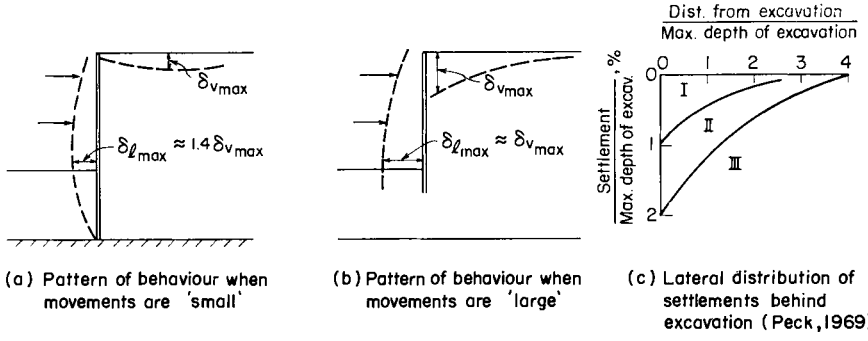
In the following paragraphs, available knowledge concerning the movements of excavation support systems in soft clay is summarized in such a fashion as to provide a useful means of predicting performance. The degree of accuracy of the predictions is enhanced by careful construction control.

8.3.7.1 Basic movement patterns

The basic deformation response of the soil and wall system in soft clay to the effects of excavation stress relief was shown earlier in this chapter in Fig. 8.8. It was demonstrated that as long as the yield in the subsoil was minimal, deformations of the soil and wall system are predominantly lateral. As yielding becomes more significant, the vertical component of soil motion behind the wall increases, and the bottom begins to heave.

Additional insights into basic response patterns are depicted in Fig. 8.16; the soil surface and wall movements are shown for the final excavation depth of a braced wall where the movements are 'large' in one case and 'small' in the other. The effects of wall end conditions are also demonstrated. The

primary distinction between the cases of large and small system movements is in the form of the soil surface deformations. For small movements, the maximum soil surface settlement occurs at some distance back from the wall. In this case, if the wall does not settle, the friction between the wall and the soil will not be fully mobilized, and this force holds up the soil next to the wall. When large movements take place, however, the friction force is fully mobilized and cannot resist further soil movements; as a result, the maximum surface settlements occur next to the wall. Field measurements consistently support these trends (see, for example, Lambe, 1970; Clough et al, 1972; O'Rourke et al, 1976).



(a) Pattern of behaviour when movements are 'small'

(b) Pattern of behaviour when movements are 'large'

(c) Lateral distribution of settlements behind excavation (Peck, 1969)

Fig. 8.16. Basic response patterns of wall system in clay.

The lateral extent of the settlements that occur behind a wall is indicated in Fig. 8.16 by the normalized plot presented by Peck (1969), where the ratio of settlement to excavation depth is plotted versus distance from the excavation divided by the excavation depth. If settlements are small and fall into zone I on this chart, then movements will be felt to a distance behind the wall of twice the excavation depth. On the other hand, if settlements are large enough to fall into zone II, they may extend some four excavation depths behind the wall.

8.3.7.2 Relationship between tendency for basal heave, soil stiffness and movements for sheet pile walls

In recent publications, Peck (1969), D'Appolonia (1971) and Goldberg et al (1976) have presented data on movements of excavations in clay, primarily based on performances in Chicago and Oslo. These data show that settlements associated with excavations where basal stability is a problem are larger than those where no such problem exists. It is possible to refine the trends developed in the earlier papers by being selective about the data used.

In order to relate movements to basal heave tendencies exclusively, the effects of other variables must be excluded. Fortunately, enough field data are now available to do this with guidance provided from finite element analyses.

In assembling and correlating the data, the Authors observed two ground rules. First, cases where obvious construction problems occurred were eliminated; the effects of construction problems are significant and are discussed subsequently. Second, the factor of safety against basal heave was calculated in every case using the procedures of Terzaghi (1943) described earlier in this chapter, with no allowance made for effects of wall embedment; the factor of safety so obtained is probably not the actual factor of safety against basal heave, but it serves as a useful index to the degree of stability of the excavation.

Maximum lateral movements of sheet pile walls that are crosslot braced are normalized by dividing them by the height of the excavation, and these are plotted versus the basal heave factor of safety in Fig. 8.17. Data for the plot comes from excavations in San Francisco, Chicago, Boston and Oslo. It should be noted that movements recorded during the early stage of a wall life, when it is cantilevered and not braced, are excluded from Fig. 8.17. As will be shown later, the cantilevered wall behaves in a substantially different manner from a braced wall.

Also shown in Fig. 8.17 are results predicted by finite element analyses of braced wall systems in homogeneous clay deposits of essentially normally consolidated clay. Details of these analyses were given by Clough & Denby (1977). Nonlinear soil behaviour, construction sequence, and shear strength and soil stiffness increasing with depth were all considered. The trend line shown for the finite element results was defined for the case where the initial tangent modulus of the stress-strain curve, E_i, was calculated as 600 s_u. This relationship falls within that defined in Fig. 8.3 for low to medium plasticity clays which are normally or slightly overconsolidated. The theoretical trend can be seen to correlate well with that of the average of the field data.

It is clear from the field and from finite element results that movements increase as the factor of safety against basal heave decreases. There is a rapid increase as the factor of safety of 1.0 is approached. The field data also suggest that movements of walls which have the toe embedded into an underlying stiff layer (open symbols in Fig. 8.17) are less than those which have the toe still in the moving soil mass (solid symbols in Fig. 8.17). This trend cannot be considered to be certain, however, since most of the data for free-end walls comes from one case history in Oslo, Norway (Vaterland III), where the excavation was flooded for a period of several weeks. The data for this example, therefore, may not be representative of average behaviour. Additional information is needed before firm conclusions can be drawn.

The correlation of wall movements and factor of safety against basal heave establishes several important design guides. First, the expected movements for an excavation with similar soil and wall conditions can quickly be evaluated directly from the chart. Second, the finite element analyses can be seen to give reasonable predictions of soil behaviour. This provides confidence in the use of the finite element procedures in the study of other systems.

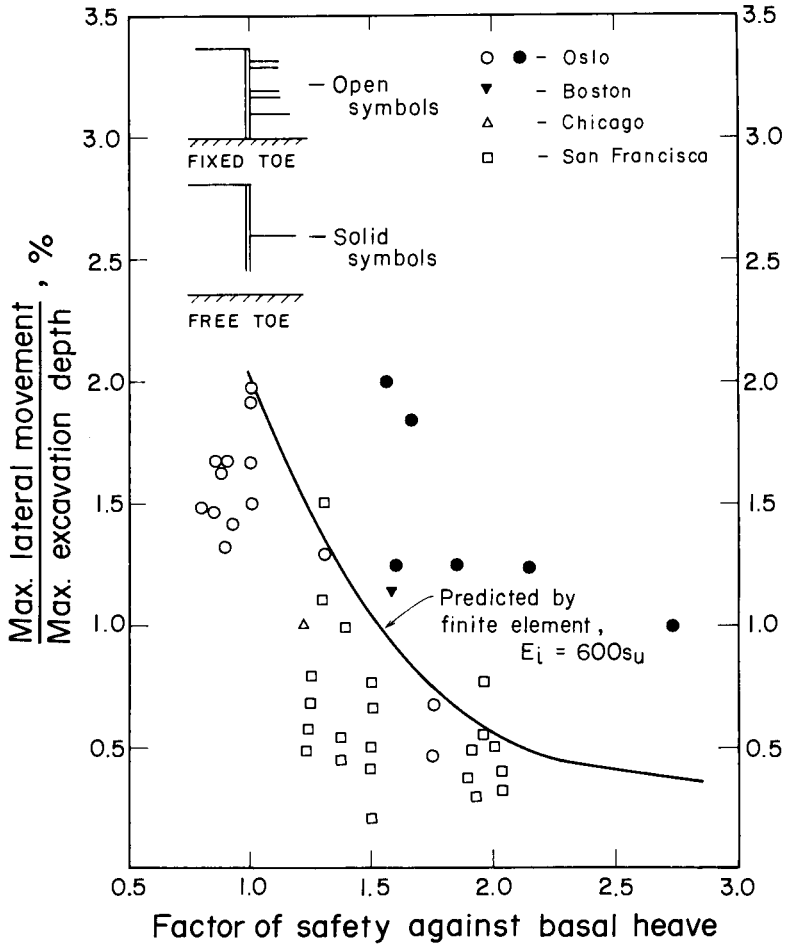


Fig. 8.17. Relationship between maximum wall movement and factor of safety against basal heave.

Settlements behind the wall systems for which the lateral movements were plotted show trends in factor of safety variation similar to those observed in Fig. 8.17. At low factors of safety ($F < 1.5$), the maximum settlement may be assumed about equal to the maximum lateral movement, while at higher factors of safety, the settlement is usually 50% less than the lateral movement.

8.3.7.3 Movements of earth berm supported walls

The preceding discussion was exclusively concerned with crosslot braced walls. In wide excavations of deep clay deposits where tie-back anchorage cannot be obtained, excavations are constructed by use of an earth berm-raker

support system. An example of the earth berm system was described earlier for the Levi Strauss Building in San Francisco (see Fig. 8.5).

The earth berm-raker system in soft clays may generally be expected to move more than the crosslot braced type wall, because a substantial amount of excavation is made before the structural supports (rakers) are supplied. Unfortunately, these systems are not as well documented by field data as are the crosslot braced systems, and they do not allow a detailed study of parameter effects. Finite element studies by Clough & Denby (1977), however, have defined theoretical relationships between settlements behind bermed sheet pile walls and the stability number, $N_b = \gamma H / s_{ub}$, for the condition of the excavation after the berm has been removed and the rakers installed. These relationships are shown in Fig. 8.18. Various curves are drawn for different size berms and wall end conditions. For reference, the response of a cantilever wall and a crosslot braced wall are also shown. All of the data assume normally consolidated clays with shear strength and stiffness increasing with depth. The soil stiffness is defined by an initial tangent modulus of 600 s_u.

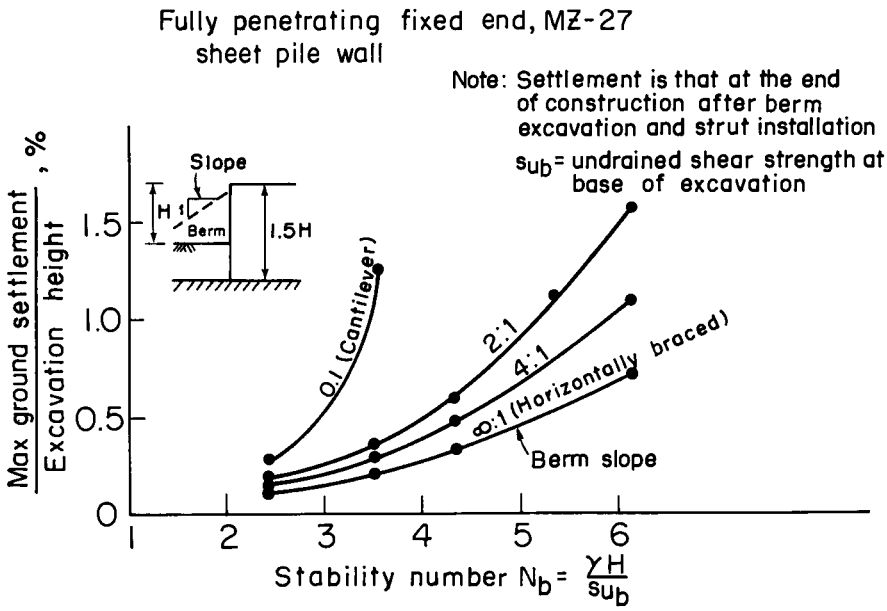


Fig. 8.18. Effect of earth berm on wall movements (Clough & Denby, 1977).

The trends of the data in Fig. 8.18 show that an increase in the size of the berm serves to reduce movements and that, as expected, the bermed walls move more than crosslot braced walls. The cantilever wall clearly moves more than the bermed or braced walls. Perhaps most importantly, the data demon-

strate the areas of effectiveness of berm support. The following zones of behaviour may be observed:

(1) At low stability numbers ($N_b < 3$), a flattening of the berm slope has little effect on movements.
(2) At stability numbers between 3 and 5, a flattening of the berm slope up to that defined by a 2:1 slope produces large movement reductions; further increases have little effect.
(3) At high stability numbers (>5), a flattening of the berm slope from 2:1 to 4:1 produces a large decrease in settlements, but large movements occur anyway, since the soil moves beneath the wall and carries the berm with it.

8.3.7.4 Effect of support system stiffness

The support system stiffness involves the wall, the supports themselves, the spacing between supports, and the length of wall embedded below the excavation bottom. Variations in any of these parameters changes the system stiffness. This is an important concept, since many engineers view the wall stiffness as the only important parameter. Clough & Tsui (1974) showed that variations in brace or tie-back stiffnesses can have as large an effect on movements as variations in wall stiffness.

A valuable study into the effect of wall stiffness and support spacing was presented by Goldberg et al (1976). They defined a stiffness parameter as

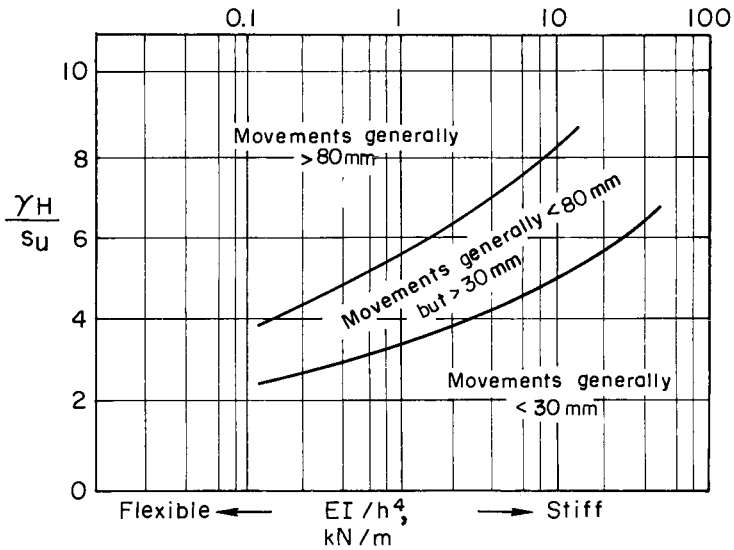


Fig. 8.19. Effects of wall stiffness and support spacing, *h*, on lateral wall movements (Goldberg et al, 1976).

EI/h^4, where EI is the stiffness of the wall and h is the distance between supports. The results of their study are depicted in Fig. 8.19, where the stability number, $\gamma H/s_u$, is plotted versus EI/h^4, and various boundary lines are drawn to establish zones of expected lateral wall movements. These data suggest that an increase in EI/h^4 has a significant effect in reducing movements. For example, at a stability number of 6, an increase in EI/h^4 from a stiffness of 1 to a value of 10 should lead to a reduction in lateral wall movement from about 80 mm to about 30 mm. Increases in stiffness above 10 have smaller effects.

The data in Table 8.2 summarize the field measurements of movements of six slurry wall systems in soft to medium clay. In three cases, in spite of relatively high stability numbers, small movements were observed. In the other three cases, however, large movements ($>0.01H$) occurred. In two of these latter instances, construction effects were significant, while in the third, the EI/h^4 value was low because of large spacings between the support levels. These results show that Goldberg et al (1976) were correct in considering the effect of span length between supports, and that poor construction can eliminate the advantages gained by using a stiff support system.

Table 8.2. Movements of slurry walls in clay

Reference	Location	EI/h^4* (kPa/m)	$\gamma H/s_{ub}$	Max. movement / Excav. depth (%)	Comment
Lambe (1970)	Boston	1470	4.4	0.2	
DiBiagio & Roti (1972)	Oslo	4300	7.0	0.1	
O'Rourke et al (1976)	Chicago	1420	6.6	0.5	
O'Rourke et al (1976)	Chicago	3020	7.6	1.8	Problems caused by nearby foundation construction
O'Rourke et al (1976)	Chicago	6490	6.6	2	Problems caused by nearby foundation construction
Clough & Denby (1977)	San Francisco	109	5.5	1	

* Note that EI is based only on concrete section.

8.3.7.5 Effect of preloading

Preloading of a wall system produces several positive effects toward reducing movements. As a practical matter, preloading removes slack from the system, and thus eliminates a potential source of movement. Each application of preload reduces also shear stresses set up in the soil by previous excavation

effects. This means that the soil is unloaded, and its stress-strain response is stiffened until the next excavation step causes the shear stresses to reload the soil beyond its maximum previous level of shear stresses. This temporary stiffening of the soil leads to reduced movements.

Clough & Tsui (1974) showed that the use of preloads of the order indicated by the design strut load diagrams given by Peck et al (1974) (i.e. 0.2 to 0.4 γH) serves to reduce system movements to 50% of those for a wall with no preloads. The use of higher preloads can reduce system movements further, but at a diminishing rate. There are also dangers in using very high preloads in that the wall can actually move into the soil and damage adjacent utilities (see Saxena, 1974).

8.3.7.6 Influence of construction factors

Throughout the previous discussion, reference has been made to the influence of construction factors on excavation movements. The degree of this influence in soft clays can be substantial, both during excavation and after the excavation is completed. It is beyond the scope of this chapter to discuss all the possible influences of construction procedures. A recent review has been made by Clough & Davidson (1977), and useful specific case history accounts were provided by White (1976), Broms & Stille (1976), O'Rourke et al (1976), Hansbo et al (1973), Lambe et al (1970) and Zeevart (1972).

Additional movements of excavations and even local failures have been produced by late installation of supports, pile driving, caisson construction, loss of water through tie-back holes, interlocks or joints of slurry and sheet pile walls, remoulding and undercutting of clay berms, and surcharge loads from soil and equipment. Clough & Davidson (1977) even cited one case where a strike during a critical period of construction led to large movements. Because these factors cannot always be anticipated, it is difficult to make accurate movement predictions for excavations in clay. However, many of the effects of construction have been defined, and they can be anticipated and controlled therefore by good specifications and construction procedures. The designer must consider how the excavation and subsequent construction will be carried out, identify critical construction effects, and incorporate these aspects into his performance estimates and specifications.

It is also possible for the designer and contractor to use construction techniques to advantage in reducing movements. Reduction of bottom heave in the excavation for the Latino-Americano Tower in Mexico City provides a classical example (Zeevart, 1972). In this case, the final excavation was carried out by staged removal of narrow strips of soil in which foundation elements were built before the subsoils felt the effect of soil excavation. Additional construction procedures which can be used to control movements are:

(1) Dewatering of underlying sand layers and seams during excavation, which reduces excavation heave (but possibly increases settlements behind the excavation).

(2) Driving of piles through overburden using followers, which reduces heave and minimizes possible damage to exposed support walls.

(3) Immediate filling of holes created by the removal of old piles.

(4) Excavation of berms in small and alternate stages while supports are installed.

Other techniques can be defined for individual projects. The use of positive measures to prevent movements can mean the difference between success or failure of a project.

8.3.7.7 Summary of excavation movements

In order to estimate the movements of supported excavations in soft clays, the designer needs to know: (1) soil shear strength, (2) soil stiffness, (3) type of support system, (4) degree of wall toe restraint, (5) degree of preloading, (6) depth of excavation, (7) amount of surcharge load, (8) method and duration of excavation construction, and (9) methods and duration of construction within the excavation. With the data and results of the kind given in Figs. 8.18 to 8.19, approximate evaluations of expected movements can be made. The present state-of-the-art, while still crude, does represent an advance over conditions a decade ago.

The finite element method provides a useful tool capable of predicting movements of excavations. The influence of changes in individual parameters can be evaluated, and in more and more investigations, general behaviour trends are being established. The effects of construction can be modelled by simulating different procedures of construction. The limitations of the finite element procedures lie in their complexity and inaccessibility to the average engineer, and in the fact that not all soil conditions and events during construction can be anticipated. The use of the finite element method in cases of design of supported excavations has been described by Palmer & Kenny (1972), Clough et al (1972), Cole & Burland (1972), Murphy et al (1975) and Stroh & Breth (1976). Murphy et al (1975) and Clough & Mana (1976) demonstrated how the predictions of a finite element analysis can be integrated with an observational programme. Observed movement patterns can be judged within the context of predicted movements and soil stress levels from finite element analyses.

It is clear that methods for predicting movements of excavations are improving. However, significant problems remain to be solved. In particular, an integrated scheme is needed which accounts for the effects of all the major variables in a logical and consistent fashion.

8.3.8 Conclusions on excavations

The stability, loads and movements of excavations and excavation support systems are functions of a large number of variables, some of which are not fully under the control of the engineer. However, our knowledge of the effects of the important parameters is expanding rapidly. Heretofore, many aspects of behaviour were not understood well enough to allow rational decisions to be made. At present, a good technology exists which produces reasonably accurate results, provided that a reasonable degree of engineering judgement is used.

The keys to our developing knowledge of excavation behaviour are the growing store of well-documented performance records and the finite element method. The finite element method has been most usefully applied to the excavation problem by establishing trends of expected performance as a result of changes in isolated variables, and in case history studies. It is valuable in the study of effects which cannot be ascertained by examination of field data.

The definition of the effects of a wide range of parameters allows a designer to make rapid trade-off evaluations. In regard to the effects of construction variables, the designer can use the available data to determine on which parameters he wishes to establish limits in his specifications and where to concentrate construction monitoring efforts. He might also find it possible to mitigate adverse construction effects by the wise selection of construction techniques.

The review of excavation behaviour also shows that we still have significant problems which are not solved. Some specific needs are:

(1) A suitable method to evaluate bottom heave potential for walls embedded below the excavation bottom.
(2) An inclusive procedure to evaluate support system stiffness and its effect on behaviour.
(3) A complete and integrated method to predict movement patterns which account for the effects of all major variables.
(4) Procedures to evaluate soil stiffness values quickly and reliably.
(5) Additional well-documented case histories, including not only load and movement measurements but also in-situ monitoring of the changes in soil stress state and pore pressure.
(6) A means to predict whether drained or undrained behaviour will occur.

8.4 TUNNELS IN SOFT CLAY

8.4.1 Tunnelling technology

The behaviour of soil around a tunnel in soft clay is strongly influenced by the method of excavation and the lining of the tunnel. Usually, tunnels in soft

clay are constructed using the shield method. The shield is basically a steel cylinder which is jacked ahead by thrusting on the in-place liner. The shield provides temporary support of the soil and allows excavation to proceed at the tunnel face. In the tail of the shield, liner segments are erected in preparation for the shield advance.

Details of shield design can be important to performance. It is common to provide an overhanging hood at the front for protection of the personnel and to allow an inclined face, which is more stable than a vertical face. The cutting edge at the front of the shield is often equipped with a relieving bar to reduce friction and to make steering easier. In soft clays, this device may be of little use since the soil will probably close in on the shield just behind the relieving bar. A few tunnels have recently been built using shields with articulated joints at one or two locations to separate the hood and cutting edge, the central part (containing jacks and excavating equipment) and the tail, which trails behind. This allows greater ease in turning and steering without excessive ground loss.

Many types of excavating tools are available, including hand-held spades and clay knives, power spades or knives, hoe type excavators, and rotating and oscillating spokes equipped with teeth of various kinds. The rotating or oscillating excavators may be partially closed, affording some face support but allowing only limited access. A new type of shield, the slurry mole, maintains a bentonite or natural clay slurry under pressure at the face. Soil cuttings from the rotating excavators drop into the slurry and the soil-slurry mixture is pumped out. It is occasionally feasible to push the shield 'blind' with the face closed or nearly closed, allowing the soil to squeeze into the tunnel through small openings. This technique is not much used today because of the distress and large displacements forced onto the surrounding soil.

Support for the tunnel face might be required in soft clays, and this can be provided by compressed air, pressurized slurry of the slurry mole, or by use of mechanical restraints such as breasting boards or closed sections at the shield face. Compressed air is a proven method, but it involves substantial labour and equipment costs and has inherent health hazards. The slurry mole (Bartlett et al, 1973) affords continuous support of the tunnel face and efficient removal of the soil cuttings. The machine itself is expensive, however, and problems can arise with separating the clay cuttings from the slurry, and with handling obstructions.

A recent Japanese development is the earth pressure balance method, in which pressure is maintained against the face through a mass of loosened soil under pressure in front of a bulkhead in the shield. A rotating digger loosens the soil, which is removed from the bulkhead space by an enclosed screw conveyor. This method is better suited to tunnelling in soft clay than the slurry mole, since it eliminates the problems in removal of the excavated soil from the slurry.

The lining is often compeletely assembled within the tail of the shield. This leaves a void after shield advance, which is filled by gravel and/or grout. Alternatively, the liner segments may be erected, but not tied together, and expanded against the soil after shield advance. The lining may or may not be bolted, and varying requirements as to water tightness may impose widely different design and construction criteria.

It is apparent that there are many alternative methods of excavation, support of the soil, and linear construction. Different methods lead to differences in the stresses and displacements in the surrounding clay. In this situation, the problems of the tunnel contractor are also those of the designer. The designer must assess the sensitivity of the project as regards safety, stability, effects of ground movements, and performance requirements of the finished structure; these factors must be the basis for restrictions to be placed on the contractor's operation, and for the monitoring requirements to safeguard the structure and the environment. It is most economical to maximize the contractor's choices and minimize his restrictions. The preparation of tender documents that are safe and adequate, yet allow acceptable freedom in the contractor's operation, is therefore a painstaking task which requires accurate forecasting of soil behaviour based on geotechnical data. Table 8.3 summarizes briefly the important decisions or choices the contractor might face on a tunnel project in clay, and Table 8.4 gives a summary of the designer's tasks.

8.4.2 Problems and methods of analysis of tunnels in soft clay

Certain problems of tunnelling in soft clay stand out in importance. These include: (1) face stability, (2) wall and crown stability, (3) displacements and settlements, (4) effects of settlements on environment, and (5) lining loads.

Table 8.3. Tunnel contractor's alternatives

Excavator:	Hand or power tools	*Propulsion:*	Number of jacks
	Hoe or boom cutter		Capacity requirement
	Rotating or oscillating excavator arms		Jack arrangement
	Open, partly or fully supported face		Provisions for absorbing thrust
	Blind or partly blind shoving	*Lining construction:*	Erect within tail or behind
Immediate support:	Required?—yes/no		Expand against soil
	Shield, or roof support only		Grout or gravel tail void filling
	Slurry support or other pressure balance		Secondary grouting
	Hooded, inclinded or vertical face		Lining type
	Relieving bars on cutting edge	*Spoil removal:*	Pumps
	Articulation of shield		Conveyors
			Cars—rail, rubber wheels
			Combinations, other
		Other:	Shaft arrangement, ventilation, lighting, water control, etc.

Table 8.4. Tunnel designer's tasks

Site investigations:	For design For contract documents and construction
Appraisal of existing structures:	Type Quantity Sensitivity Value
Design of structures:	Adequate and economical Compatible with construction Lining alternatives (one or several): bolted, unbolted, ribs and lagging steel, iron, concrete initial, permanent prefabricated, cast-in-place water tightness Line and grade Tunnel shape Spacing between adjacent tunnels
Tender documents:	Complete with geotechnical data Adequate for project Minimize contingencies Allowable alternatives, restrictions Monotoring provisions Fair and equitable

Each problem has its subparts, such as the sources and components of displacement, or the magnitudes and rates of the displacements which occur through the face when it is advancing. In order to establish solutions for these problems, it is first necessary to define a meaningful framework of concepts and relationships. Since theoretical relationships are only approximations of the actual problem, results must then be verified by tests and field experience, and appropriate adjustments must be made by empirical factors.

Theories of tunnel lining behaviour have been the subject of many papers, such as those by Muir Wood (1975) or Peck et al (1972). Most theories are based on certain assumed types and magnitudes of earth loading, and they disregard important soil behaviour aspects during and after construction, yet they are useful. Theories of elastic and elasto-plastic stresses and displacements around tunnels under simplified assumptions are well established. The most important ones were summarized by Deere et al (1969). Even though important aspects such as gravity stresses, strength variations with depth and deviations from hydrostatic initial stress conditions are usually ignored, the theories have been proven useful and reliable within their limitations. Finite element analyses can take into account additional parameters (as shown, for example, by Hoyaux & Ladanyi, 1970).

Model tests provide helpful verification of theories. Recent important model tests include those performed by Lögters (1974) and by researchers in Cambridge, England (Atkinson et al, 1974; Atkinson & Orr, 1976; Atkinson & Potts, 1977).

Attempts to combine theory and empirical data into useful guidelines have best been exemplified by the paper by Peck (1969), which was followed by others, including the state-of-the-art paper by Cording & Hansmire (1975). Mention should also be made of notable papers on the closely related subject of the susceptibility of structures to settlements by D'Appolonia (1971), Burland & Wroth (1975) and O'Rourke et al (1976).

It is not intended in this chapter to discuss fully all of these aspects of tunnelling in soft clay. This presentation will concentrate on aspects of soil behaviour relevant to tunnelling, essentially as seen from the points of view of the geotechnical practitioner and the tunnel designer. Based on theoretical assessments of stresses and displacements around tunnels and on empirical supporting data, tunnel face and wall support requirements are shown to be functions of specific soil parameters and of minimum settlement criteria. The predictability of soft clay behaviour as a function of soil characteristics, geometrical data and construction conditions has moved soft clay tunnelling from the realm of art towards the realism of engineering.

8.4.3 Stresses around tunnels in clay—a stress path analysis

8.4.3.1 Total and effective stresses

Henkel (1970) presented a stress path analysis for the conditions surrounding a deep excavation in normally consolidated and overconsolidated clays, using a two-dimensional stress space defined by the principal compressive stresses. This type of analysis is also well suited to examine conditions around a tunnel in clay. Although the analyses are in fact not readily amenable to closed quantitative and generalized solutions, certain results and conclusions are useful, and the analyses may be performed with benefit for a variety of conditions other than those assumed herein.

Consider a uniform clay material with an undrained shear strength s_u and the drained shear parameters $c'=0$, and φ'. A tunnel is driven at depth z, where the total overburden pressure is $\sigma_{vo}=\gamma z$. With a pore water pressure u_o at this depth, the effective vertical stress is $\sigma'_{vo}=\sigma_{vo}-u_o$. Assume also, for simplicity, an 'at rest' coefficient, K_o, of unity, so that horizontal and vertical stresses are equal, both in terms of total and effective stress, i.e. $\sigma_h=\sigma_v=\sigma_{vo}$ and $\sigma'_h=\sigma'_{vo}=\sigma_{vo}-u_o$. On a stress path diagram in the (σ_h,σ_v)-plane, the starting point is then located on a 45° line through the origin, both in total and effective stress terms (refer to points A or B in Fig. 8.20 and to point $\bar{B}$ in Fig. 8.21).

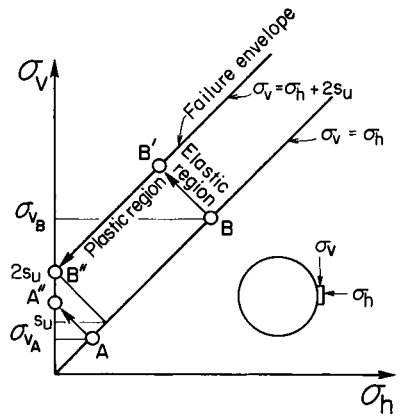


Fig. 8.20. Total stress path for soil element at tunnel spring-line.

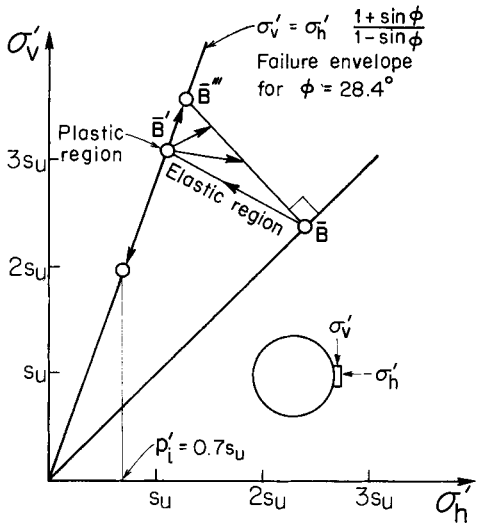


Fig. 8.21. Effective stress path for soil element at tunnel spring-line.

If a circular tunnel is now opened in this material at depth z, with no shield and no lining, the radial stress at the tunnel wall is zero. If the soil remains in the elastic state, i.e. it does not shear and enter the plastic state, the total tangential stress at the tunnel wall will be twice the total overburden pressure, $2\sigma_{vo}$. On the total stress path, the stress point for a soil particle at the spring-line then moves upward to the left along a 45° line, as shown in Fig. 8.20. For a condition where $\sigma_{vo} < s_u$ (point A in the figure), conditions remain elastic. On the other hand, when $\sigma_{vo} > s_u$ (point B in Fig. 8.20), the soil must shear in a region, since the stress path cannot cross the failure envelope. When

the path reaches the failure envelope (point B' in Fig. 8.20), it travels down the envelope until it meets equilibrium. The tunnel might be perfectly stable even if the soil has sheared in a sizeable region around the opening; a plastic zone develops, which sheds load toward more distant regions where increased confinement enables the soil to carry greater stresses.

With a given σ_{vo} and s_u, and an internal pressure, p_i, the radius of the plastic region, R, in the undrained ($\varphi=0$) state is (Deere et al, 1969):

$$R = a \exp \frac{\sigma_{vo} - p_i - s_u}{2s_u} \tag{8.7}$$

where a is the tunnel radius. The radial stress, σ_r, in this zone increases with distance, since:

$$\sigma_r = p_i + 2\, s_u \ln \frac{r}{a} \tag{8.8}$$

and the tangential stress is:

$$\sigma_\theta = \sigma_r + 2\, s_u \tag{8.9}$$

In the plastic zone, the stress point is located on the line between B'' and B' in Fig. 8.20. At the tunnel wall, this point is represented by B'' (with $p_i=0$); at the distance of the plastic radius, $r=R$, the stress point is at B'. The stresses along a horizontal line through the tunnel centre are shown in Fig. 8.22 for $\sigma_{vo}=4\, s_u$. The third principal stress, the longitudinal stress, is:

$$\sigma_y = \tfrac{1}{2}\,(\sigma_r + \sigma_\theta) = p_i + s_u + 2\, s_u \ln \frac{r}{a}$$

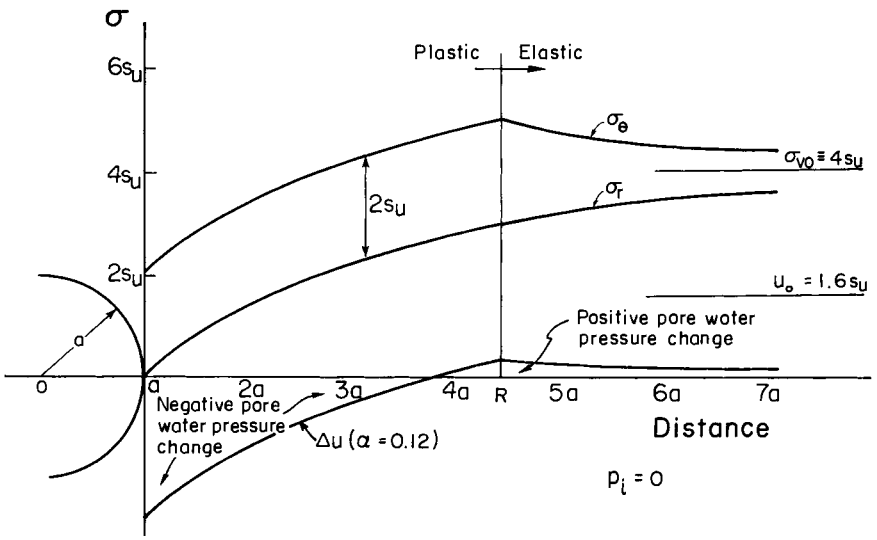


Fig. 8.22. Total stresses and pore water pressures along horizontal axis through centre of a tunnel in clay.

Beyond the distance R, i.e. between points B and B' in Fig. 8.20, the stresses are elastic and are given by:

$$\sigma_r = \sigma_{vo} - s_u \left(\frac{a}{r}\right)^2 \exp\left(\frac{\sigma_{vo} - p_i}{s_u} - 1\right)$$
$$\sigma_\theta = \sigma_{vo} + s_u \left(\frac{a}{r}\right)^2 \exp\left(\frac{\sigma_{vo} - p_i}{s_u} - 1\right) \tag{8.10}$$

For the undrained condition (no volume change), the stress in the longitudinal tunnel direction remains at the total vertical overburden pressure, $\sigma_y = \sigma_{vo}$.

8.4.3.2 Pore pressure development

Stress changes set up pore pressures which must dissipate with time. With the approach suggested by Henkel (1960), the pore pressure change in the plastic region can be determined by the equation:

$$\begin{aligned}\Delta u = & \tfrac{1}{3}(\Delta\sigma_r + \Delta\sigma_\theta + \Delta\sigma_y) \\ & + a[(\Delta\sigma_r - \Delta\sigma_\theta)^2 + (\Delta\sigma_\theta - \Delta\sigma_y)^2 + (\Delta\sigma_y - \Delta\sigma_r)^2]^{\frac{1}{2}} \\ = & \; p_i - \sigma_{vo} + s_u \left(1 + 2\ln\frac{r}{a} + a\sqrt{6}\right)\end{aligned} \tag{8.11}$$

where a is Henkel's pore pressure parameter. The effective stresses are then:

$$\begin{aligned}\sigma'_r &= \sigma_r - u_o - \Delta u = \sigma_{vo} - u_o - s_u - a\, s_u\sqrt{6} \\ \sigma'_\theta &= \sigma'_r + 2\, s_u = \sigma_{vo} - u_o + s_u - a\, s_u\sqrt{6} \\ \sigma'_y &= \sigma'_r + s_u = \sigma_{vo} - u_o - a\, s_u\sqrt{6}\end{aligned} \tag{8.12}$$

The stresses are independent of p_i and of the distance r. Typical values of a are:

Heavily overconsolidated clay:	$a = -0.60$ to -0.24
Lightly overconsolidated clay:	$a = -0.24$ to $+0.12$
Normally consolidated clay:	$a = 0.12$ to 0.48
Sensitive clay:	$a = 0.30$ to 0.80

Beyond the plastic radius, R, the pore pressure change is:

$$\Delta u = a\sqrt{6}\, s_u \left(\frac{a}{r}\right)^2 \exp\left(\frac{\sigma_{vo} - p_i}{s_u} - 1\right) \tag{8.13}$$

and the effective stresses become:

$$\sigma'_r = \sigma_{vo} - (1 + a\sqrt{6})\, s_u \left(\frac{a}{r}\right)^2 \exp\left(\frac{\sigma_{vo} - p_i}{s_u} - 1\right)$$

$$\sigma'_\theta = \sigma_{vo} + (1 - \alpha\sqrt{6})\, s_u \left(\frac{a}{r}\right)^2 \exp\left(\frac{\sigma_{vo} - p_i}{s_u} - 1\right)$$
$$\sigma'_y = \sigma_{vo} - \alpha\sqrt{6}\, s_u \left(\frac{a}{r}\right)^2 \exp\left(\frac{\sigma_{vo} - p_i}{s_u} - 1\right) \tag{8.14}$$

For the conditions shown in Fig. 8.22, the induced pore water pressures are based on an assumed α value of 0.12. The effective stresses are shown in Fig. 8.23. If it is assumed that the original pore pressure (neutral stress) is equal to $u_o = 0.4\ \sigma_{vo} = 1.5\ s_u$, the original vertical effective stress is $\sigma'_{vo} = 2.4\ s_u$. The effective stresses in the plastic zone are then given by Eq. 8.12 with $\alpha = 0.12$, as:

$$\sigma'_r = 1.1\ s_u$$
$$\sigma'_\theta = 3.1\ s_u$$

with the difference being equal to 2 s_u. This stress point is plotted as $\bar{B}'$ on the effective stress path diagram in Fig. 8.21. Since this point represents a plastic condition, it must be on the effective stress failure envelope. The failure envelope shown corresponds to $c' = 0$, $\varphi' = 28.4°$. Point $\bar{B}'$ in Fig. 8.21 represents the stress state throughout the entire plastic zone, and it corresponds to the line $B'-B''$ in the total stress path diagram of Fig. 8.20. The line $\bar{B}-\bar{B}'$ represents elastic stress states outside the plastic zone, and it corresponds to the line $B-B'$ in Fig. 8.20.

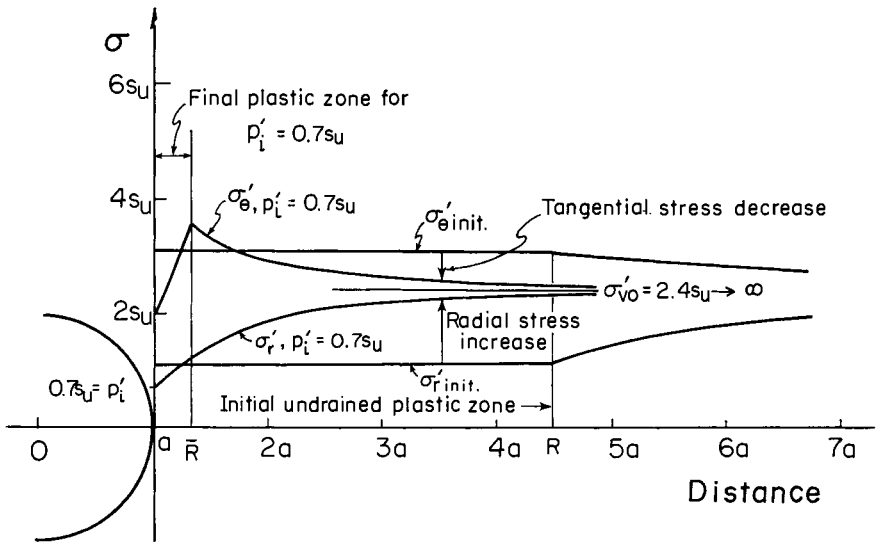


Fig. 8.23. Efiective stress along horizontal axis through centre of a tunnel in clay.

8.4.3.3 Dissipation of pore pressure

So far, consideration has been given only to the situation immediately after the tunnel excavation, i.e. before pore pressure dissipation. The effects of pore pressure dissipation and the resulting time-dependent changes in effective stresses will now be examined.

If the tunnel is tight and does not allow leakage, it may be assumed that the pore pressures eventually reach the original equilibrium condition ($u=u_o=1.6\ s_u$ in the example shown). In this process, however, both the total and the effective stresses must change. As the pore pressures dissipate, a drained condition is gradually obtained, and the boundary of the plastic zone moves closer to the tunnel. At the same time, however, loads are transmitted to the lining which would now be in place. An opening in a frictional material with $c'=0$ is not stable without a lining.

The process of pore pressure dissipation is difficult to follow in detail; it involves the suction of water into the zone at negative Δu from the zone at positive Δu. If the water demand of the negative zone is greater than the surplus of the positive zone, water must travel from greater distances, and a potential volume increase near the tunnel results. If this volume increase cannot occur because of the incompressibility of the lining, loading of the liner occurs, and a new stress equilibrium results.

8.4.3.4 Final state, drained condition

In the final state, the pore pressure is equal to u_o everywhere, and equilibrium has been obtained with a lining load $p_i=p_i'+u_o$. The effective stress distribution is now found by a different set of formulae (Deere et al, 1969). With $c'=0$, the radius of the plastic zone, $\bar{R}$, is now:

$$\bar{R} = a\ [(1-\sin\varphi')\frac{\sigma_{vo}'}{p_i'}]^{\frac{1-\sin\varphi'}{2\sin\varphi'}} \tag{8.15}$$

and the stresses in the plastic zone are:

$$\sigma_r' = p_i' \left(\frac{r}{a}\right)^{\frac{2\sin\varphi'}{1-\sin\varphi'}}$$

$$\sigma_\theta' = p_i' \frac{1+\sin\varphi'}{1-\sin\varphi'} \left(\frac{r}{a}\right)^{\frac{2\sin\varphi'}{1-\sin\varphi'}} \tag{8.16}$$

At $r=\bar{R}$, the stresses are:

$$\sigma_R' = \sigma_{vo}'\ (1-\sin\varphi')$$

$$\sigma_{\theta R}' = \sigma_{vo}'\ (1+\sin\varphi') \tag{8.17}$$

For $r \geqslant \bar{R}$:

$$\sigma'_r = \sigma'_{vo}\,[1 - (\frac{R}{r})^2] + \sigma'_R\,(\frac{R}{r})^2$$

$$\sigma'_\theta = \sigma'_{vo}\,[1 + (\frac{R}{r})^2] - \sigma'_R\,(\frac{R}{r})^2 \tag{8.18}$$

It is seen that the radius of the plastic zone coincides with the tunnel radius when $p'_i = (1 - \sin\varphi')\,\sigma'_{vo}$. It would be tempting to assume that this value, representing an equilibrium of sorts, would also represent the eventual and maximum lining load under $K=1$ conditions. In other words, the maximum total load, including water pressure, would be:

$$p_i = u_o + (1 - \sin\varphi')\,\sigma'_{vo} = \sigma_{vo} - \sigma'_{vo}\sin\varphi' \tag{8.19}$$

However, while this might be a fair approximation in many instances, unfortunately, it will be a poor one in others. In fact, there is no physical law that prevents relaxation beyond the 'elastic' state alluded to above, or prevents it from stopping sooner.

To examine this problem a little further, consider the condition where the lining pressure has reached a certain value p_i. For simplicity, assume that the pore water pressure is now in equilibrium at $u = u_o$ (i.e. hydrodynamic effects have ceased), and consider relaxation or creep without excess pore pressures. For the previous example (Fig. 8.22), with the assumption that p'_i is equal to, say, 0.7 s_u (or 0.28 σ'_{vo}), the resulting stress conditions are shown in Fig. 8.23. For the circumstances shown, which are fairly typical, the radial effective stress increases substantially with time, except for a short distance close to the tunnel. With a slightly greater p'_i, the radial effective stress could increase throughout. On the other hand, the tangential effective stress decreases throughout, except for a local region centred around the effective stress plastic radius, $\bar{R}$.

On the effective stress path diagram in Fig. 8.21, it will be recalled that the initial shear stresses set up by the tunnel excavation cause the stress point to move from $\bar{B}$ to $\bar{B}'$ in the plastic zone, while stress points in the elastic zone remain between $\bar{B}$ and $\bar{B}'$. The dissipation of excess pore pressures and relaxation or creep cause the stress point to move up or down the failure envelope. In the region outside $\bar{R}$, the stress point ends on a line $\bar{B}$—$\bar{B}''$ which is inclined at 45° to the axes.

8.4.3.5 Some conclusions based on stress path analysis

The example analyzed above allows the following conclusions to be drawn with regard to the effects of tunnel construction in soft clay:

(1) As the excavation takes place, a relatively large volume of soil reaches a plastic condition.

(2) Significant pore pressure reductions occur; the pore pressures may even become negative ($u_o - \Delta u < 0$) in a large region.

(3) With time, pore pressure differences dissipate, and effective stress equilibrium must result, which requires a build-up of lining load.

(4) In terms of effective stresses, the final equilibrium stresses are very much different from the initial stresses; the radius of the plastic zone shrinks drastically and might even vanish with time.

(5) Since the total stresses in the final condition are quite different from the initial total stresses, the soil is not subjected to simple consolidation or swell; a complex readjustment of both total and effective stresses must occur.

(6) A typical soil volume close to the tunnel spring-line (at $r \approx 1.2a$ in the example) is first subjected to undrained plane shear involving total stress reduction in all directions, then to a swelling process during which both total and effective stresses are subject to continuous change.

(7) A typical soil volume far from the tunnel wall is subjected to a similar initial undrained shear, and might swell for a while as pore pressures equilibrate; eventually, however, it might tend to consolidate, since the radial stress increase is greater than the tangential stress decrease.

For simplicity, the conditions near a tunnel have been illustrated here by a specific example. The same analysis can be carried out with different values of the ratio σ_{vo}/s_u, the initial pore pressure, u_o, the friction angle, φ', and the pore pressure parameter, α, but within wide ranges similar conditions will prevail.

8.4.4 Consolidation and creep around tunnels

Many authors have reported a continued displacement or volume change or swelling around tunnel openings in both soft and stiff clays, or conversely, a continued build-up of lining pressure, leading sometimes to lining pressures which are even in excess of the overburden pressure. This process must be the result of a tendency of sheared or distressed clay to absorb water and lose strength.

Skempton (1970), in connection with slope stability in London Clay, implied that sheared, overconsolidated clay, when allowed to change volume, eventually reaches a reduced, fully softened shear strength, equivalent to the residual strength or the strength of the normally consolidated clay. This mechanism develops along numerous discrete shear surfaces and is aided by softening along existing fissures, if present.

Laboratory tests representing the stress paths indicated in the previous section, have not, to the Authors' knowledge, been carried out. Tests illustrating similar effects were, however, carried out by Singh et al (1973), who

conducted swell tests and direct shear tests on swelled samples of reconstituted Bearpaw Shale. Some of their principal conclusions were:

(1) The ratio of horizontal to vertical stress, K, reaches a limiting value during unloading, beyond which the sample shears in passive failure.
(2) When the sample during unloading reaches an OCR of about 3.5, K reaches unity; the passive failure condition is reached for an OCR of the order of 10 to 20 (this is in agreement with Eq. 8.6).
(3) For very low normal stresses, the failure envelope is curved and there is no effective cohesion intercept.
(4) Samples swelling in unloading, not in the passive failure condition, follow laws of hydrodynamic retardation (Terzaghi theory) and virtually stop swelling after a while. On the other hand, samples in passive shear failure keep swelling beyond the time at which pore pressures are essentially dissipated.
(5) In accordance with Skempton's implication, swelling might well continue until the water content reaches that of a normally consolidated soil at the prevailing pressure.

Since the excavation of a tunnel in clay involves the total removal of radial stresses along the tunnel wall, and usually results in undrained shear (and eventually always in drained shear), it is not surprising that continued swelling has been observed in unlined tunnels. With a tunnel lining in place, the swelling is restrained, and the result is instead an increase in tunnel lining load, usually in conjunction with lining distortion. Equilibrium is eventually reached, where the water content in a zone near the tunnel is in keeping with the stresses. If the lining is essentially incompressible, the overall volume change of the surrounding soil is near zero. Hence, any increase in water content near the tunnel must be accompanied by a decrease in water content at a distance. This is in agreement with Fig. 8.23, which shows an increase of the mean stress at a distance, but a decrease near the tunnel. The eventual lining pressure is a function of the extent to which swelling and water migration can occur. It is evident that the quantity of excess water in the outer zone is dependent on the radius to which consolidation occurs. This radius is a function of the plastic radius, R, in the undrained shear condition. If the plastic radius is small, or if there is no initial plastic zone, the volume of soil in undrained shear is small or zero, and little or no consolidation or swell occurs. Hence, the build-up of lining pressure most probably stops at or below the value of:

$$p_i = u_o + (1 - \sin \varphi') \sigma'_{vo} \tag{8.20}$$

At this value, the soil is no longer anywhere in a drained shear condition. Since this case occurs with no significant change in the mean effective stress around the opening, it involves little water migration.

Equation 8.20 disregards the effect of an effective stress cohesion intercept, c'. It was seen that for very low stresses this cohesion intercept does not exist. However, for the condition where the undrained plastic radius is small, or where no undrained plastic zone exists, the soil never really reaches shear failure in the drained condition, because lining stresses build up to prevent it; hence, the effective cohesion intercept might remain intact. Under those circumstances, it can be shown that the equilibrium lining pressure takes the form:

$$p_i = u_o + \sigma'_{vo}\,(1 - \sin\varphi') - c'\cos\varphi' \tag{8.21}$$

This equation may be used for $\sigma_{vo}/s_u \leqslant 1$, whereas Eq. 8.20 may be used for $\sigma_{vo}/s_u > 1$.

Several factors must be considered in addition to those described above. The analyses have assumed that the 'at rest' coefficient, K_o is unity and that the effects of ground surface proximity and gravity forces may be neglected. A relatively shallow tunnel in soft clay will develop an undrained shear zone which very nearly reaches the ground surface; in fact, since the shear strength decreases towards the ground surface, it is virtually certain that the entire soil mass above the tunnel is affected and influences the behaviour of the clay at the tunnel depth. If the clay is normally consolidated, or nearly so, disturbance and reconsolidation leads to a stress state which closely approximates to the original normally consolidated state. In other words, the vertical load on the tunnel approaches the full overburden pressure, and the horizontal pressure on the side of the tunnel reaches at least the normally consolidated 'at rest' pressure. If the lining is flexible, it must deflect to build up uniform pressures all around, and if the soils are very compressible and the lining very flexible, the horizontal stress eventually reaches the overburden pressure. Depending on the circumstances, the average lining pressure then varies between:

$$p_i = \sigma'_{vo} + u_o = \sigma_{vo} \tag{8.22}$$

and

$$p_i = \sigma'_{vo}\,(1 - 0.5\sin\varphi') + u_o \tag{4.23}$$

It is seen that the standard design assumption that a shallow tunnel in soft clay carries the full overburden pressure is fair. The liner distortion required for this stress adjustment can be estimated from consolidation theory, on the assumption that local horizontal consolidation occurs under a load of $\sigma'_{vo}\sin\varphi'$, but is perhaps better estimated on the basis of experience (Peck, 1969).

Another example of practical interest is represented by the condition of a relatively shallow tunnel in overconsolidated clay, where K_o is near the practical limit of plastic shear, i.e. around 2.5. In that instance, K in terms of total stresses might be about 1.8, depending on the location of the water-table. The ratio σ_{vo}/s_u might be of the order of 0.5, and conditions should remain elastic if K is unity. According to elastic theory, the tangential stress at the spring-line is:

$$\sigma_{\theta s} = (3 - K)\,\sigma_{vo} = 1.2\,\sigma_{vo} \tag{8.24.}$$

Since $q_u = 2\,s_u \approx 4\,\sigma_{vo}$, conditions initially remain elastic at the spring-line However, at the crown and invert:

$$\sigma_{\theta c} = (3K - 1)\,\sigma_{vo} = 4.4\,\sigma_{vo} \tag{8.25}$$

Here, the soil is significantly overstressed. In fact, the entire body of soil above the crown and some distance below the invert might be placed in shear failure, which is not too surprising since the soil was nearly in passive failure in terms of effective stresses before the tunnel was constructed. The overstress triggers the mechanism of swelling, and again we find a tendency towards load increase in the crown, with an accompanying 'squat'. The squat is, however, well resisted by the stiff soil at the spring-line. Before the lining is placed, the crown overstress might well cause tendencies to 'slabbing' or other types of instability, especially in a fissured clay.

If the swelling continues to build up towards an equilibrium condition bounded by normal consolidation at the prevailing pressure, it is easy to see that the lining pressure might exceed the overburden pressure, since the prevailing average pressure with $K > 1$ is greater than the overburden pressure. The maximum pressure on the lining might, after a long time, reach a magnitude of the order of:

$$p_i = u_o + \tfrac{1}{2}\,(1 + K_o)\,\sigma'_{vo} \tag{8.26}$$

or, for the circumstances quoted above, 30 to 50% higher than the total overburden pressure.

The rate of lining pressure build-up, and the rate of lining distortion, has been shown (Peck, 1969) to be linear with the logarithm of time, both for soft and stiff clays. These mechanisms continue for years. They follow the same type of relationship observed for creep or secondary consolidation in laboratory samples of clays. At the present time, however, a method of predicting rates of load build-up or distortion has not been devised.

In the past, it has been common to distinguish between two types of clay behaviour in tunnels, namely *squeezing* and *swelling*. Squeezing is the displacement or encroachment of clay into the tunnel space and towards the lining under constant volume, with no change in moisture content. Swelling, as the word suggests, involves the absorption of water with consequent volume change. The discussions presented in this and the previous sections refine these concepts and show both to be functions of shear strength, pore pressure parameters and initial and final stress states. Methods are now available to assess the probability and probable severity of squeeze and swell problems.

The undrained shear strength, s_u, the total overburden pressure, σ_{vo}, and the ratio between them (the *overload factor*), $OF = \sigma_{vo}/s_u$, define the general types of problem to be expected and the *squeeze potential*. More complex analysis, involving both pore pressure parameters, effective strength para-

meters and initial K condition, are required for a full assessment of the *swelling potential.*

Many authors have attempted to assess the swelling potential on the basis of laboratory swell tests or classifications based on Atterberg limits and clay mineral characteristics. While such methods might serve to identify soils that could produce large strains or loads under swelling conditions, they cannot identify the potential for swelling in tunnels, which is essentially a function of stress changes and strength parameters.

The conditions and considerations indicated apply generally to clays which are saturated and not indurated. Many hard clays or clay shales are not saturated and have been subjected to diagenesis or metamorphosis; they might be cemented and very well capable of sustaining high locked-in stresses which upon release will cause behaviour that is not readily predictable.

8.4.5 Soil displacement and ground loss caused by tunnelling in clay

8.4.5.1 Theoretical considerations

Stresses and stress changes around tunnels are interesting, and it is necessary to study them for a full understanding of clay behaviour. Except for lining load problems, however, they are not of direct concern to the tunnel builder. The tunnel contractor does not observe stresses, but he observes the effects of stresses, namely strains, distortions, displacements and ground surface settlements.

The construction of a tunnel produces displacements of the surrounding soil, axially towards the advancing tunnel face and radially towards the tunnel walls, which are eventually arrested by the temporary and permanent tunnel supports. In impermeable clays, most of the displacement occurs as squeeze under undrained conditions. Subsequent swelling, while of significance for lining loads and distortions, usually has little effect on surface settlements. In instances where the finished tunnel will act as a drain for the clay, or where for other reasons consolidation or reconsolidaton is imposed on the surrounding soils, delayed settlements might occur. In the general case, however, most significant displacements occur quickly during construction.

The progression of displacements at the tunnel face is a complex three-dimensional problem which depends on a variety of factors. For an order-of-magnitude assessment of probable displacements, this complexity may be ignored, and it can be assumed that the tunnel bore opens instantaneously such that displacements are essentially radial.

It is convenient to consider the volume of lost ground, i.e. the volume by which the theoretical tunnel cross-section is larger than the actual volume of soil removed. This volume can be calculated by summing up the theoretical displacements around the periphery, which result from removing the original supporting stresses around this periphery.

If the theoretical tunnel volume is V_o, and the volume reduction is ΔV, the relative lost ground volume for purely elastic conditions is:

$$V_l = \frac{\Delta V}{V_o} = (1+K)\,\sigma_{vo}\,\frac{1+\upsilon}{E_u} \tag{8.27}$$

Elastic conditions prevail for $\sigma_{vo} \leqslant 2\,s_u/(3K-1)$ when $K \geqslant 1$, or $\sigma_{vo} \leqslant 2\,s_u/(3-K)$ when $K \leqslant 1$. With Poisson's ratio $\upsilon \approx 0.5$ (little or no volume change) and $K=1$, this reduces to:

$$V_l = 3\frac{\sigma_{vo}}{E_u} \tag{8.28}$$

where E_u is the elastic modulus of the clay.

In the inelastic range, with the assumption of $K=1$, the relative lost ground volume can be written (approximately, but nearly correct for small displacements) as:

$$V_l = \frac{\Delta V}{V_o} = 2\,s_u\,\frac{1+\upsilon}{E_u}\,\exp\left(\frac{\sigma_{vo}}{s_u} - 1\right) \tag{8.29}$$

For $\sigma_{vo}/s_u = 1$, this equation reduces to the elastic equation, Eq. 8.28. The E-modulus is that applicable to the clay in the elastic state, and the equation assumes zero volume change in the plastic zone.

When an interior pressure (air pressure or lining pressure) of magnitude p_i is applied, the identical equations (Eqs. 8.28 & 8.29) apply with $(\sigma_{vo} - p_i)$ substituted for σ_{vo}. The average relative reduction of tunnel diameter, $\Delta d/d$, is equal to one-half of V_l.

This information allows us to draw a ground reaction curve. This task is made easier and more generally applicable by the fact that the ratio E_u/s_u varies only within a rather narrow range for a multitude of clays. For low plasticity clays, the data in Fig. 8.3 shows that E_u generally varies between $500\,s_u$ and $1500\,s_u$; thus, $s_u/E_u = 0.002$ to 0.00067. With this ratio, the maximum lost ground without internal pressure is:

$$V_l = m\,\exp\left(\frac{\sigma_{vo}}{s_u} - 1\right) \tag{8.30}$$

for $\sigma_{vo}/s_u \geqslant 1$, with m between 0.006 and 0.002. For $\sigma_{vo}/s_u < 1$, the equation becomes:

$$V_l = m\frac{\sigma_{vo}}{s_u} \tag{8.31}$$

For elastic conditions then, the ground loss is less than 0.2 to 0.6% of the excavated volume, or essentially negligible. However, the ground loss increases very rapidly for increasing σ_{vo}/s_u. For $\sigma_{vo}/s_u = 6$, which is the approximate value beyond which the face might become unstable, the ground loss is

between 30 and 90% (the equation is not truly valid for very large strains, however).

8.4.5.2 Field and model test data

Equations 8.30 & 8.31 are shown graphically in Fig. 8.24; Eq. 8.30 forms a straight line on this semi-log plot. The figure also shows a number of field data points from the sources indicated. For overload factors ($OF = \sigma_{vo}/s_u$) greater than about 2.5, the data points are consistently below the theoretical curves, while for values of OF smaller than about 2.5, they are often above the curves. There are several reasons for this:

(1) Most tunnels in clays with $OF > 2$ or 3 are driven with shields, which substantially reduce ground losses by providing immediate wall support.
(2) In stiff fissured clays (e.g. London Clay or Boom Clay from Belgium, from which several of the case histories have been selected), and where the value of OF is usually small for a near-surface tunnel, ground loss is often augmented by loosening along fissures.
(3) Several case histories show data which include an undetermined quantity of consolidation settlement.

Recent model tests using overconsolidated kaolin (Atkinson & Potts, 1977) indicated a relationship rather similar to that shown in Fig. 8.24 for crown settlements as a function of safety factor or overload factor. Data from Atkinson & Potts are shown in Fig. 8.25, replotted in the same format as Fig. 8.24; the curvature for large values of OF might be expected from the fact that modulus values are reduced at high strains.

8.4.5.3 Ground loss and overload factor

Let us examine in some detail the potential ground losses and the factors which affect these for various overload factors. When the overload factor is smaller than about 2, the theoretical potential ground loss is less than about 1%. The ground loss might be greater than predicted because of loosening along fissures. It is not greatly affected by construction procedures, since the rather small displacements tend to occur before they can be restrained. A shield is required only to prevent loosening and to cut a clean shape, and the lining can usually be erected and expanded directly against the soil behind the shield or some distance behind the working face. The small ground loss usually produces inconsequential settlements.

For overload factors between about 2 and 4, the potential ground loss might reach magnitudes of 10%. Ground movements through the face are generally small, and a shield can effectively restrain ground losses to a value smaller than 2 or 3%. Where adjacent structures might be affected by settlements, the use of a shield is advisable, even if the tunnel opening would be stable without initial support.

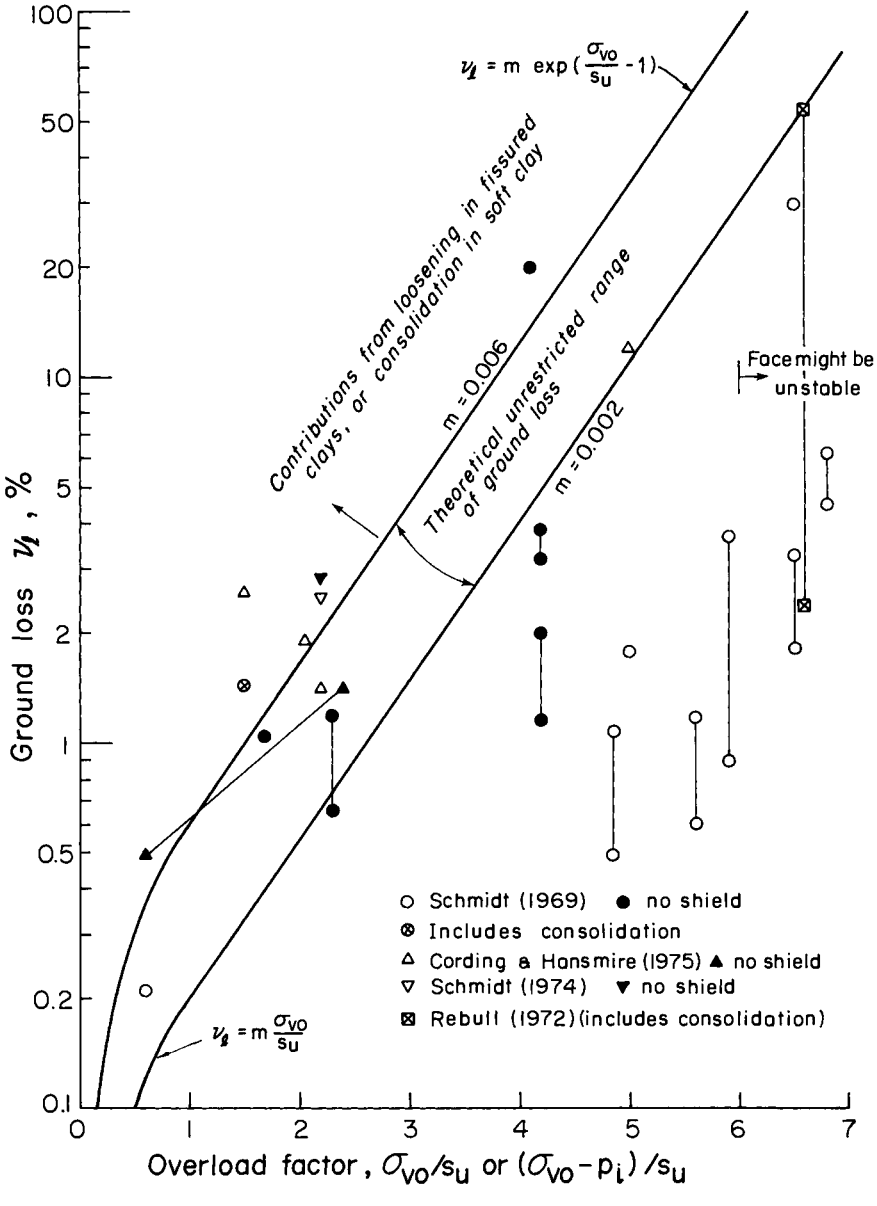


Fig. 8.24. Relationship between overload factor and ground loss.

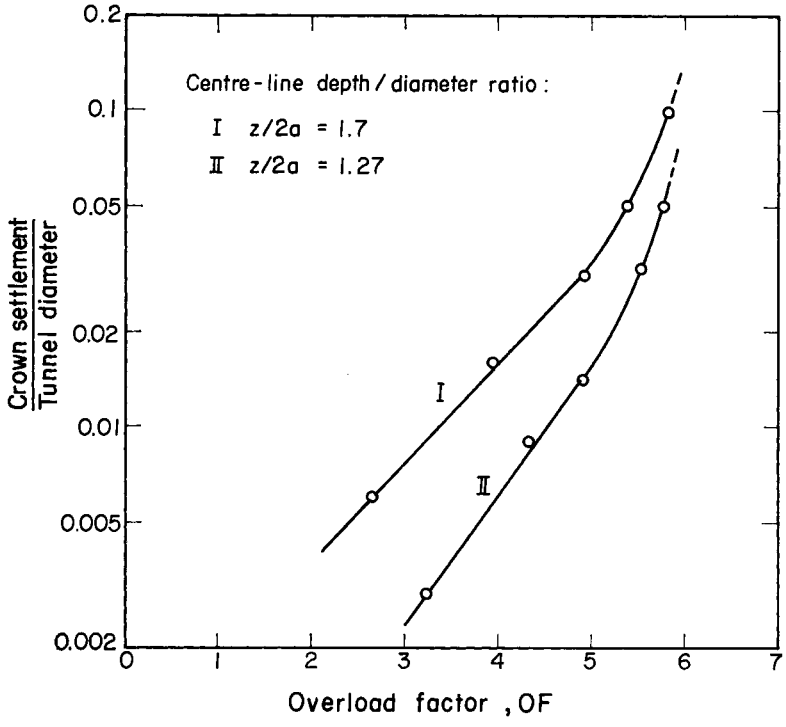


Fig. 8.25. Results of model tests on tunnels in kaolin (Atkinson & Potts, 1977).

For overload factors between 4 and 6, a shield is usually required, even if substantial settlements are acceptable. The shield is effective in reducing radial displacements. In the lower range, between $OF=4$ and $OF=5$, the shield also tends to reduce face displacements. For a value of OF near 6, however, face displacements tend to increase, and face support is often advisable, especially where surface settlements are critical. For overload factors greater than 6, the face might be unstable, and full face support is usually required. This may be accomplished mechanically, by slurry support or by compressed air.

When compressed air is used, the air pressure required to maintain losses below a given maximum can be determined readily from the curves given in Fig. 8.24 for the direct effect of the air pressure, p_i, on the overload factor. The slurry pressure required for a slurry supported tunnel face can be determined in the same way.

8.4.5.4 Shield tunnelling and face movements

As indicated, a shield provides immediate wall and crown support. In addition, it tends to reduce soil displacements through the face. Unless high air pressure is maintained in the tunnel, however, a shield will not eliminate

ground losses. Ground loss will occur to an extent through the face, into the void left behind a relieving bar, and into the tail void behind the lining. Steering of the shield around curves, and involuntary deviations from straight shoving *(ploughing* or *yawing)* also contribute to ground loss. Calculations based on simple geometry can provide estimates of maximum potential ground losses (see for example, Schmidt, 1974). Usually, these ground losses can be held within about 3% (see Fig. 8.24).

With a high overload factor, the tail void is quickly filled by squeezing clay. To avoid or reduce this contribution to the ground loss, air pressure is required, or the tail void must be filled almost at once with grout or gravel. For more moderate overload factors (3 to 5), tail void filling is usually effective when employed ring by ring, or up to a distance of one-quarter to one-half of the diameter behind the shield.

Soil displacements through the face are difficult to assess. Field measurements (Cording & Hansmire, 1976) from shield tunnels in soft clay ($OF=2$ to 5) suggest that between one-quarter and one-third of the ground loss under these circumstances occurs through the face. For overload factors higher than 5, a greater face contribution may be expected.

Elasto-plastic computer studies by Ghaboussi & Ranken (1975) suggest that if the average face displacement measured axially toward the tunnel is δ, then the ground loss per unit length of tunnel contributed from the face is approximately:

$$V_l = \frac{\delta}{ka} = \frac{2\delta}{a} \tag{8.32}$$

where a is the tunnel radius. Figure 8.26 shows how this equation may be viewed in simplistic terms, assuming a linear relationship between the face advancement and the displacement of a particle towards the face. The equation suggests an average value of $k=1/2$.

Both model and field test data are available to elucidate on this problem. Lögters (1974) measured centre-line displacements while advancing a model tunnel in gelatine, and found the relationship shown in Fig. 8.27. Rebull (1972)

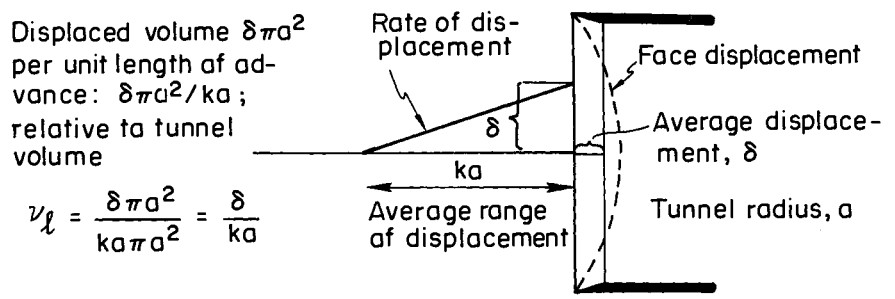


Fig. 8.26. Theoretical relationship between lost ground volume and face displacement.

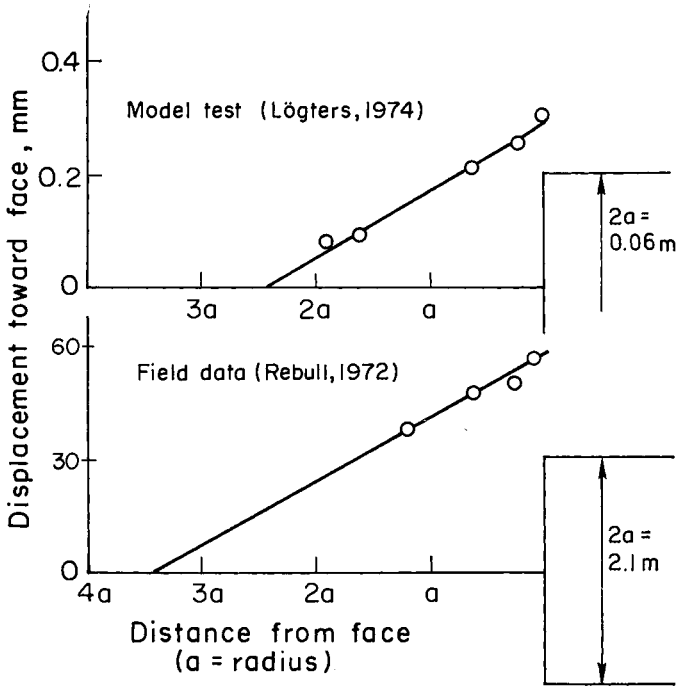


Fig. 8.27. Model test and field data relating displacement towards the face and distance from the face.

measured displacements at a number of points along the centre-line in front of a tunnel face in soft clay (OF between 6 and 7) while tunnel advance was temporarily halted. In these instances, the observed range of displacements was of the order of 2.5 to 3.5 times the tunnel radius. Earlier reports (Terzaghi, 1943; Ward, 1969) indicate that for a variety of OF values, earth movements begin at least at a distance equal to twice the radius. Though displacements at the centre-line do not represent the average over the height of the face, the coefficient in Eq. 8.32 appears to be doubtful. A value of $k \approx 1$ or greater (Fig. 8.26) would appear more appropriate, or:

$$V_l = \frac{\delta}{ka} \approx \frac{\delta}{a} \tag{8.33}$$

The case history quoted above (Rebull, 1972) also affords a rare look at time-dependent face displacements in soft clay (OF=6 to 7) while the face is stagnant. While many early measurements have shown that face movements virtually stop when the tunnel advance is halted, even for values of OF up to 5 and possibly greater, Rebull reported significant movements. Some of his data are shown on a log time plot in Fig. 8.28.

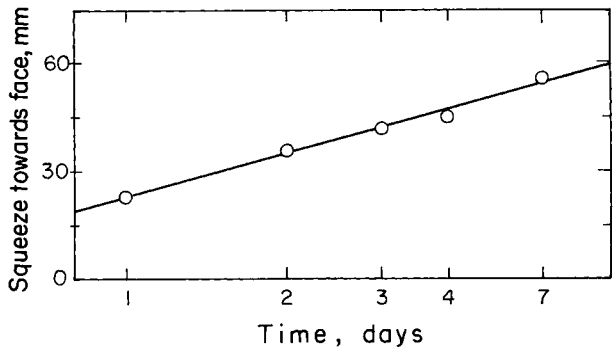


Fig. 8.28. Time variation of face displacements during a halt in tunnelling, overload factor=6 to 7 (Rebull, 1972).

Based on the data presented in the preceding paragraphs, we may conclude that:

(1) Face losses for OF-values up to about 5 are of the order of one-third the total ground losses when a shield is used.

(2) Face support becomes important for OF greater than about 5 when settlements are critical.

(3) Face losses are significantly reduced by a firm support of just a part of the face; the concepts behind Eqs. 8.32 & 8.33 suggest that with one-half of the face firmly supported, the relative ground loss could be reduced to one-quarter.

(4) During work stoppage, the face should be supported for the duration, at least when $OF>6$, and preferably when $OF>5$.

(5) When compressed air is used for ground movement control, the overload factor modified by the internal pressure should be held in the range between 4 and 5.

(6) Slurry face support is usually not necessary or cost-effective for $OF<5.5$.

Some of these findings are summarized in Fig. 8.29.

8.4.6 Settlements caused by tunnelling—acceptable ground loss

The previous sections examined ways of predicting tunnel stability and ground loss, and ways to combat excessive ground loss. Methods of estimating ground surface settlements will now be considered.

Tunnels are often driven under city streets or at other places where settlements caused by the tunnelling could have adverse effects. The questions of which level of effort and which methods should be employed to minimize ground losses (and thereby settlements) are then important. Fortunately,

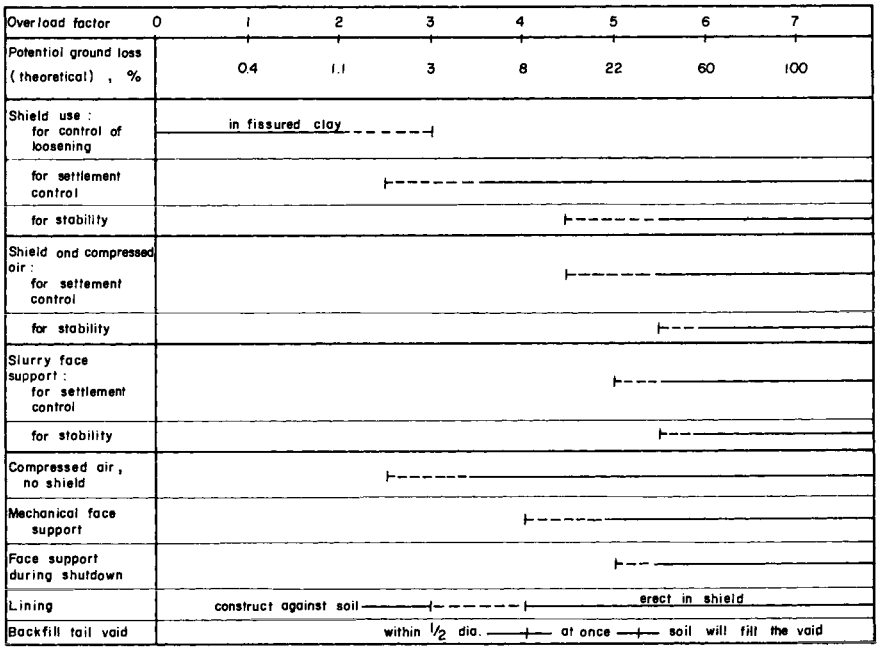


Fig. 8.29. Recommendations for tunnelling operations in relation to overload factor.

methods of estimating settlements caused by tunnelling in clay are now well established. They can be briefly summarized thus:

(1) Since the initial ground loss occurs under undrained conditions, i.e. without volume change, the volume of the settlement trough is (very nearly) equal to the ground loss in the tunnel.

(2) The shape of the settlement trough approximates closely to that of the error function. If s_{max} is the maximum settlement over the tunnel centre-line, the equation for the settlement trough can be written as:

$$s = s_{max} \exp\left(-\frac{x^2}{2i^2}\right) \tag{8.34}$$

This shape is shown with other characteristics in Fig. 8.30; the quantity i is the distance from the centre-line to the point of inflection.

(3) The volume of the settlement trough per unit length along the centre-line is equal to:

$$V = 2.5\, i\, s_{max} \tag{8.35}$$

With a given ground loss, the settlement trough is defined if either i or s_{max} is determined.

(4) Empirical evidence has shown that the quantity i for tunnels in clay can be determined with very close approximation from the purely geometric relationship:

$$\frac{i}{a} = \left(\frac{z}{2a}\right)^{0.8} \tag{8.36}$$

where z is the tunnel centre-line depth. This relationship is shown in Fig. 8.31.

(5) With these data and empirical relationships, the shape and size of the settlement trough is completely determined when the ground loss is known.

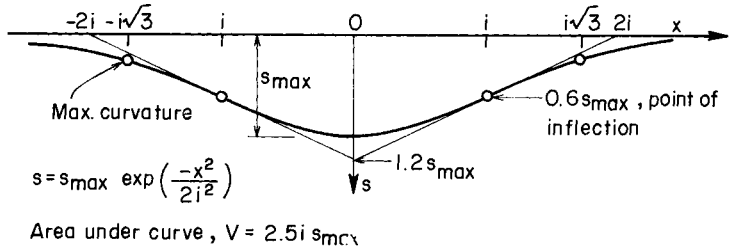


Fig. 8.30. Use of error function to approximate settlement trough.

Natural deposits and contractors' procedures are rarely uniform and perfect, and deviations from the above norms are common. As examples, dense granular soils above the cohesive tunnelling medium tend to bulk or expand, resulting in a reduction in width and volume of the trough, while a loose sand might densify and produce the opposite effects. Both effects are commonly observed.

Very large displacements, involving soil volumes at considerable lateral distances, tend to widen the trough (see the three black triangles in Fig. 8.31). Varved clays, or any other circumstances which tend to produce a greater than ordinary lateral extent of the source of ground loss, will produce settlement troughs wider than indicated by Eq. 8.36. On the other hand, ground losses associated with loosening above the tunnel crown are often centrally concentrated. Such loosening, especially if extensive, tends to work its way straight up and produce a narrow trough. Frequently, a settlement trough is observed which follows the ordinary shape and which originates from ordinary elastic and plastic displacements, together with a central enlargement of highly variable character caused by loosening or other ground looses concentrated over the crown. These types of variation can often be anticipated, but predictions of their effects on settlements are difficult to make. The first

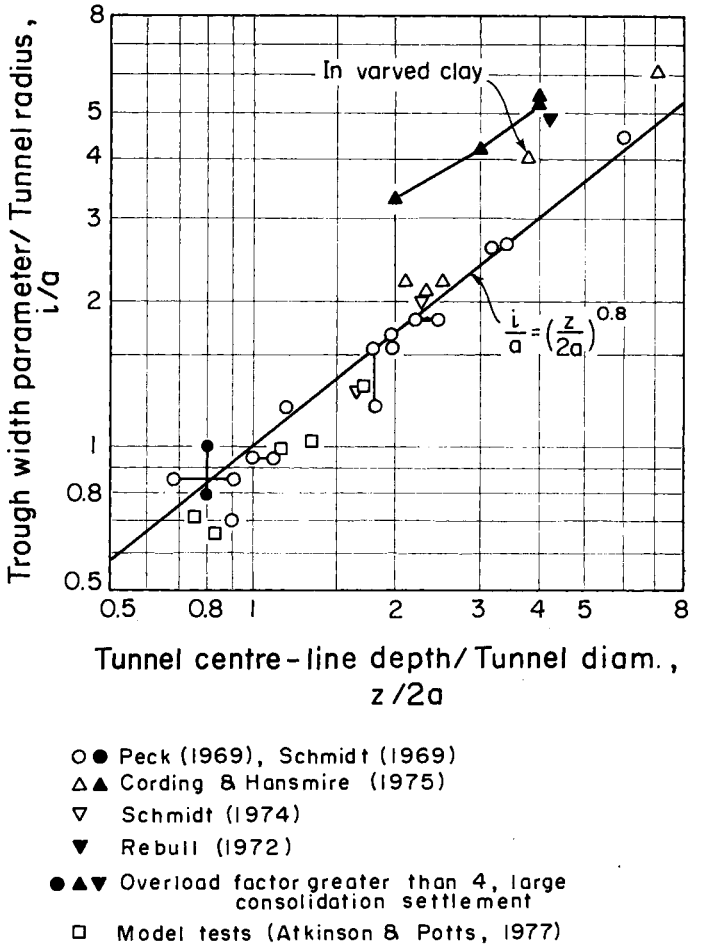


Fig. 8.31. Normalized logarithmic plot of settlement width parameter, i, versus tunnel depth, z.

estimations must be based on ordinary expectations, and judgement based on the possible and plausible variations must temper final analyses and recommendations.

Let us consider the effects of ordinary settlements on buildings or other structures. Most writers consider that damage will occur when either a given slope of the trough (relative differential settlement) or a given curvature is exceeded. Often, a maximum permissible slope of 1/300 or 0.0033 is quoted.

The maximum slope of the settlement trough occurs at distance i, and is:

$$\frac{ds}{dx} = 0.61\,\frac{s_{max}}{i} \tag{8.37}$$

For a given ground loss V_l (%), s_{max} is calculated from:

$$\frac{\pi a^2 V_l}{100} = 2.5\, i\, s_{max}$$

$$\text{or} \qquad s_{max} = 0.0126\, V_l \frac{a^2}{i}$$

With $i = a\,(z/2a)^{0.8}$, we find the slope:

$$\frac{ds}{dx} = 0.023\, V_l\,(\%)\,\left(\frac{a}{z}\right)^{1.6} \tag{8.38}$$

For a given tunnel depth, z, and radius, a, this equation will tell if an estimated ground loss will produce unacceptable slopes or differential settlements. For example, assume a tunnel depth of 3 diameters, $z=6a$, and a ground loss of $V_l=2.5\%$, and we find $ds/dx=0.0033$. For twice the depth, we find $ds/dx=0.0011$, or only one-third of the slope for the same ground loss. While 2.5% ground loss might be the maximum permissible in the first instance, 7.5% might be acceptable at twice the tunnel depth.

This problem may also be analyzed by evaluating the curvature of the subsidence trough. Maximum curvatures occur over the centre-line and at a distance of $\sqrt{3}\, i$. Over the centre-line, the radius of curvature is:

$$\rho = \frac{i^2}{s_{max}} \tag{8.39}$$

This radius is even more dependent on depth than the maximum slope of the settlement trough. It can be shown from Eq. 8.36, that as the tunnel depth increases from $z=6a$ to $z=12a$, the minimum radius of curvature increases by 5.3 times.

It is evident that a much larger ground loss can be tolerated for a tunnel at great depth than for one near the ground surface. The equations presented here allow us to estimate not only the surface effects based on the estimated ground loss, but also the tolerable ground loss for prevailing conditions. With these methods, it is possible to evaluate the tunnel support requirements in a rational fashion, and to select the type of tunnel excavation and support method which will perform adequately at least cost.

8.4.7 Monitoring of tunnel construction

The monitoring of ground movements during tunnel construction is now performed as a matter of course on many projects. Monitoring serves several purposes: (1) it allows the owner or engineer to determine if the tunnel contractor is in compliance with the requirements, (2) it permits diagnoses to be made of sources of ground loss, and thus points a way to remedy faulty

construction practices, (3) it permits predictions of further settlements down the line of construction, and (4) it serves as legal documentation, if required.

The simplest monitoring consists of the measurement of surface settlements and surface horizontal displacements. The monitoring of deep settlement points and of deep horizontal displacements by inclinometers and plugs is also often performed. In addition, tunnel distortions are measured, and the encroachment of soil into the tail void is gauged through ports in the lining. Volumes or weights of soil removed are measured and compared with theoretical volumes, and grout quantities are also compared with theoretical volumes. Methodologies of monitoring have been the subject of several recent papers, for example by Dunnicliff & Schmidt (1974) and Schmidt (1976).

The single most useful monitoring device for diagnostic purposes is often a settlement point just above the tunnel crown. Settlements of this point in the soil mass indicate the proportion of ground movement occurring in front of the cutting edge, along the shield, in the tail void and over the lining. Such data indicate whether improvements could or should be made in face support, shield steering, tail void filling, or in other tunnelling details.

Comparisons between settlements over the crown, s_c, and at the ground surface, s_s, are useful. An elaboration of Eq. 8.36 indicates that the ratio of s_s to s_c in the final state should be approximately:

$$s_s/s_c = (2a/z)^{0.8} \tag{8.40}$$

with the assumption of no volume change in the subsiding soil mass. The same qualifications apply to this equation as to Eq. 8.36. The curve of Eq. 8.40 is plotted in Fig. 8.32 for comparison with recent data from various sources, from both field and model test observations. While the scatter is somewhat greater than for comparable data for Eq. 8.36 (Fig. 8.31), the relationship still appears useful.

Data regarding other subsurface displacements, for example horizontal ground surface displacements, have also been reported in several of the

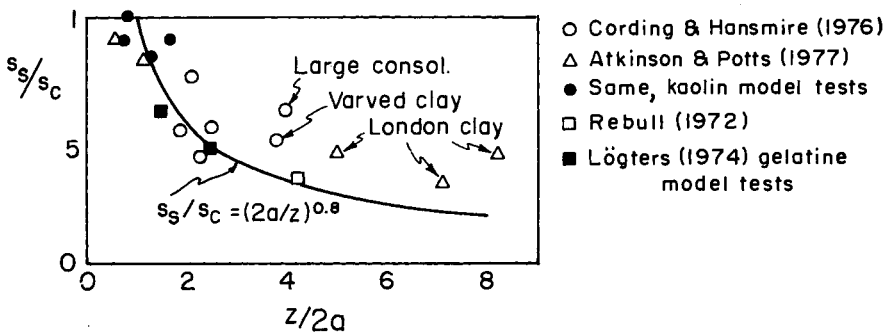


Fig. 8.32. Variation with tunnel depth of ratio of maximum surface settlement to settlement at the crown.

references quoted in this chapter. Empirical equations for horizontal strains and displacements, as well as for subsurface displacements, have been developed by Schmidt (1969), but tunnel monitoring data are yet too meager to verify these equations for tunnels in clay.

8.4.8 Conclusions on tunnels

During the last ten years, our understanding of soil behaviour during tunnelling has greatly improved. It is now possible to identify the types of problem and the behaviour to be expected during construction, and to assess these problems and behaviour modes in a rational manner. Based on geotechnical data, designs and construction methods can be selected that make the most economical use of the available technology, and which take advantage of the inherent strength and deformation properties of the soil. This state of affairs has come about primarily because of the large number of field observations taking place and the improved methodologies of monitoring.

The complexities of soil behaviour during tunnel construction do not allow theories to be taken at face value. They must be proven and calibrated by field experience. With the improved understanding of soil behaviour, and the large amounts of field observations now available, it is possible to expand theories and make extrapolations and predictions based on the theories. While the methods of analysis reported herein still leave many questions unanswered, they are sufficiently documented and proven to provide useful results for practical problems, and to serve as a framework for future developments and field verifications.

Finite element computer programs using elasto-plastic parameters are now being employed to study soil behaviour around tunnels, and with modest calibrations, these will turn into reliable tools for the prediction of ground movements. Stability problems, however, are not easily handled by computer. The problems of interaction between adjacent tunnels, common to many rapid transit and other traffic tunnels which are often built in pairs, is one that is well suited for computer analysis. This topic was the subject of considerable discussion by Peck (1969), but little new information on twin tunnels in clay, theoretical or empirical, has come forth since then. This is a topic worthy of further development.

Other topics are also in need of further development, some of which were barely touched upon in this chapter. They are:

(1) Methods of creep and relaxation analysis around tunnels to enable predictions to be made on the basis of geotechnical data.

(2) Testing of soils by subjecting them to the stress paths typical of clays around tunnels; shearing by the simultaneous increase of stress in one direction and decrease in the other direction, followed by reconsolidation.

(3) Evaluation of the excavation problems of clay hardness and stickiness based on geotechnical data.
(4) Further work on the analysis and prediction of subsurface displacements and of horizontal strains and displacements at the ground surface using computer programs and empirical data.
(5) Further work on the interaction between surface structures and ground surface settlements to determine the effects of structures on settlements, and to determine the sensitivity of structures to settlements typical of tunnelling.
(6) Development of methods of analysis and organization that will allow monitoring data to be interpreted and put to use immediately rather than on future projects.
(7) Further development of construction technology (other than compressed air) that will provide immediate support of soft soil in the shield tail void, for use when settlements must be kept to a minimum.

Most of these problems are close to their resolution, or at the very least considerable improvement can be made on the basis of available knowledge. It is hoped that researchers will take advantage of all the available field data, and that engineers will continue to monitor and report relevant tunnel construction data.

ACKNOWLEDGEMENTS

A number of individuals helped the Authors in the preparation of this chapter. Stanford graduate students who provided valuable assistance include A.I. Mana, A. Osaimi, P. Mayu, R.R. Davidson, G.M. Denby, L.A. Hansen and K.C. Sohn. Partial financial support for the work was provided by National Science Foundation Grant GK-38550.

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Chapter 9

METHODS OF IMPROVING THE ENGINEERING PROPERTIES OF SOFT CLAY

by

G. PILOT

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Dr Pilot, together with his colleagues, is well-known for his publications on aspects of the engineering problems of soft clay. Over a number of years, he has made major contributions to our understanding of the behaviour of embankments on soft clay and to the literature on the improvement of engineering properties of clay. For some years, he has been *Professor Adjoint* of Soil Mechanics at Ecole Nationale des Ponts et Chaussées, and he is presently a member of the Subcommittee on Landslides of the International Society for Soil Mechanics and Foundation Engineering.

Chapter 9

METHODS OF IMPROVING THE ENGINEERING PROPERTIES OF SOFT CLAY

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9.1 INTRODUCTION

In the process of designing and building structures on soft soils, or when foundation distress or failure is encountered, the engineer needs to consider whether the soil properties can be economically improved. He needs to be familiar with the means available to accomplish this improvement and with the limitations of each method. This chapter deals with the several methods available for the improvement of the engineering properties of soft clays. The methods discussed are:

(1) *Preloading (*or *precompression),* which is a well-known method currently in widespread use.

(2) *Vertical drains* in conjunction with precompression, which is a method now widely employed, particularly because of the recent innovations in drain technology.

(3) *Dynamic compaction,* which is quite a new process, initially limited to coarse-grained soils, but very recently adopted for fine grained materials, including soft clays.

(4) *Stone columns,* which is of fairly recent origin.

(5) *Electro-osmosis,* which although seldom used, is very useful in certain circumstances.

(6) *Heat treatment,* which is a makeshift method used when no alternative method can be employed.

(7) *Chemical treatment,* which is a little known method that merits wider use.

Not included in this chapter are methods which, although commonly used in geotechnical engineering, do not improve specifically the properties of soft soils; blasting, relief piles and the application of fabrics are examples of these.

In this chapter, the background and general principles of each soil improvement method are briefly reviewed, the construction method is fully explained, and the resulting improvement is analysed. A number of case histories are

discussed to illustrate the use of each method. The volume of literature and information available on these topics is enormous. Many published *proceedings* of conferences and symposia relate either entirely or partly to soil improvement. Of these, the *Proceedings of the Conference on the Placement and Improvement of Soil to Support Structures* (*ASCE,* 1968) summarized the state-of-the-art at that time. This chapter updates the state-of-the-art on the fast changing methods of improving soft clays and, as such, reference in this chapter is made mainly to literature published since the ASCE Conference. Very recently, *ASCE* (1978) published *Soil Improvement—History, Capabilities, and Outlook,* which is worth particular mention for its chapter on the treatment of soft soils.

9.2 PRELOADING

9.2.1 General principles

The technique of improving soil properties by precompression consists in surcharging the ground with a uniformly distributed surface load prior to the construction of the intended structure. The surcharge results in one or more of the three effects:

(1) primary consolidation settlement,
(2) secondary compression settlement, and
(3) increased undrained strength of the soil.

The preloading technique takes the two forms: (1) *overloading*, and (2) *staged construction* (Fig. 9.1). In the overloading process, a surcharge *(overload)*, p_s, over and above the design (final) loading, p_f, is placed on the site temporarily and is removed when the intended structure can be built with the occurrence of little or no additional settlement; the ratio p_s/p_f is known as the *overload coefficient*. Overloading (Fig. 9.1a) is intended for use at sites

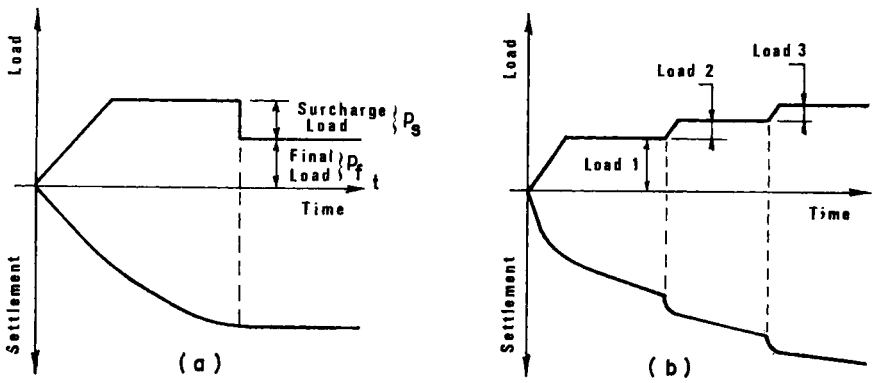


Fig. 9.1. Principle of preloading: (a) overloading, (b) staged construction.

of all forms of structures, including buildings and storage tanks, and where fills are to be built for roads, railways and airfields. Its use often permits shallow foundations to be substituted for deeper expensive ones that would otherwise be necessary.

Staged construction (Fig. 9.1b) is employed mainly as a means of gradually increasing the shear strength of a soft clay which would otherwise be inadequate to carry the intended embankment without failure.

Precompression surcharges are effected by means of:

(1) earthfills, which have been employed for over fifty years, sometimes in conjunction with sand drains (see Section 9.3),
(2) water filled reservoirs, and
(3) vacuum application.

9.2.2 Primary consolidation caused by preloading

The primary consolidation which results from the use of surcharging has been discussed by Aldrich (1965) and Johnson (1970a). The settlement-time relationship for the design load, p_f, applied to a soil without secondary compression is shown by curve (f) in Fig. 9.2; ρ_f denotes the final consolidation settlement that would occur if the design load alone were applied without precompression. When an overload, p_s, is applied with the design load, the settlement-time relationship is of the form shown as curve ($f+s$); it can be seen that the settlement ρ_f is reached at a time t_{sR}, and the overload can then be removed.

Terzaghi's one-dimensional consolidation theory provides the means for estimating the time for which an overload need be applied to bring about the

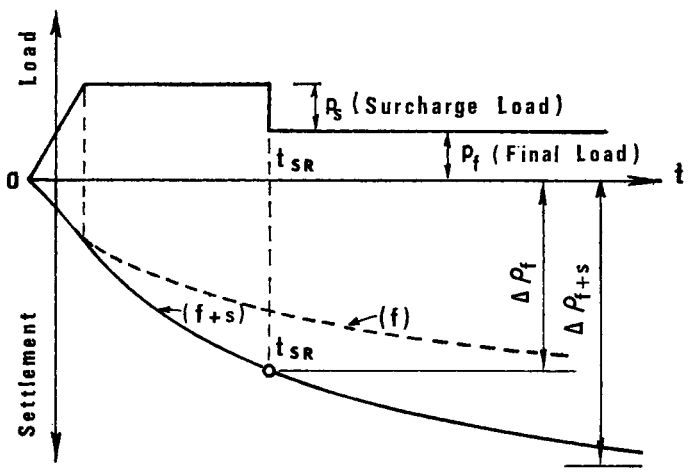


Fig. 9.2. Primary consolidation caused by preloading.

final settlement under the design load. With the conventional notation, the final consolidation settlements of a normally consolidated clay under the design load, p_f, and under the design load plus overload, p_f+p_s, will be:

$$\rho_f = \frac{2H}{1+e_o} C_c \log \left(\frac{\sigma'_{vo}+p_f}{\sigma'_{vo}}\right) \tag{9.1a}$$

$$\rho_{f+s} = \frac{2H}{1+e_o} C_c \log \left(\frac{\sigma'_{vo}+p_f+p_s}{\sigma'_{vo}}\right) \tag{9.1b}$$

where $2H$ is the thickness of the soft clay stratum with initial void ratio e_o and compression index C_c, and at initial vertical effective stress σ'_{vo}. For overconsolidated (or quasi-preconsolidated) soils, significant compressions would only occur at pressures above the maximum past pressure, σ'_{vm} (or critical pressure, σ'_{vc}, in the case of quasi-preconsolidation).

If U_{f+s} denotes the average degree of consolidation of the clay layer under p_f+p_s at time t_{sR}, then for no further settlement to occur under p_f alone:

$$\rho_f = U_{f+s} \cdot \rho_{f+s} \tag{9.2}$$

and Eq. 9.1 then gives:

$$U_{f+s} = \frac{\log \left(1+\frac{p_f}{\sigma'_{vo}}\right)}{\log \left[1+\left(\frac{p_f}{\sigma'_{vo}}\right)\left(1+\frac{p_s}{p_f}\right)\right]} \tag{9.3}$$

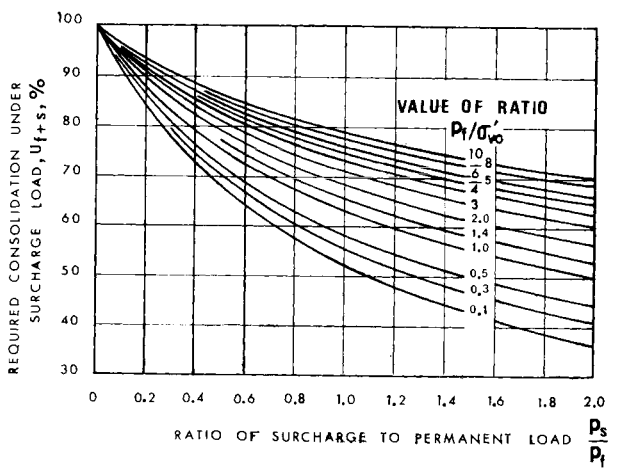


Fig. 9.3. Relationship between design load, p_f, overload, p_s, and degree of consolidation required under preload, U_{f+s}, to eliminate primary consolidation under design load.

The relationships between U_{f+s}, p_f/σ'_{vo} and p_s/p_f are shown graphical in Fig. 9.3; the time t_{sR} can be obtained from U_{f+s} in the usual way. It should be remembered that Eq. 9.3 and Fig. 9.3 are only valid if the maximum past pressure is exceeded, i.e. $(p_f+p_s+\sigma'_{vo})>\sigma'_{vm}$, and precompression is only effective in these circumstances.

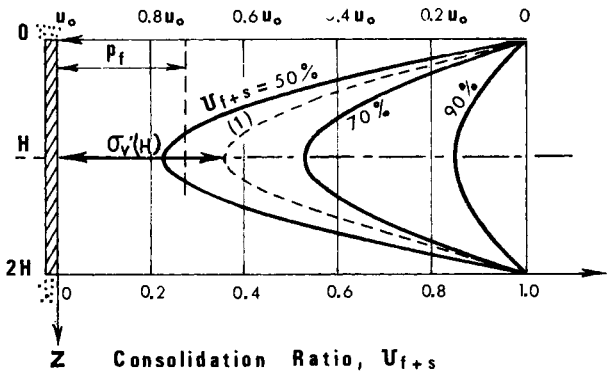


Fig. 9.4. Distribution of excess pore pressure in a clay layer during consolidation.

Figure 9.4 shows the distribution of excess pore pressure in a clay layer undergoing consolidation. The degree of consolidation (or *consolidation ratio*) at any point can be expressed as:

$$U(z) = 1 - \frac{u_e(z)}{u_{eo}} \tag{9.4}$$

where $u_e(z)$ is the excess pore pressure at depth z at any time, and u_{eo} is the initial excess pore pressure under the applied surface load. For no further settlement to occur when the overload, p_s, is removed at time t_{sR}, the effective stress, σ'_v, at the centre of the clay layer must exceed the design loading p_f, and the excess pore pressure distribution in the clay would be as shown by curve (1) in Fig. 9.4. This criterion can be expressed as:

$$\sigma'_v(H) = u_{eo} - u_e(H) \geqslant p_f \tag{9.5}$$

Since $u_{eo}=p_f+p_s$, the criterion becomes $u_e(H) \leqslant p_s$, and this substituted into Eq. 9.4 gives:

$$U_{f+s}(H) \geqslant \frac{p_f}{p_f + p_s} \tag{9.6}$$

This criterion differs from that given by Eq. 9.3 in that it is independent of the existing average overburden pressure, σ'_{vo}, and this can be seen to give the lower bound values of U_{f+s} as given by Eq. 9.3.

The estimation of the time, t_{sR}, at which the surcharge load can be removed is difficult, since the methods available for the calculation of rates of settlement have been found generally to be unreliable. The problem with the use of Terzaghi's one-dimensional consolidation theory for this purpose is that the coefficient of consolidation, c_v, can be only approximately assessed for field conditions. Whereas the calculated value of t_{sR} is useful as a rough guide to the maximum overload time necessary for a particular situation, it is essential to install instrumentation to monitor settlements and pore pressures as the basis of the engineering decision as to when the overload should be removed.

9.2.3 Secondary compression caused by preloading

When an overload is applied to the surface of a soft clay deposit, the resulting settlements are due, in part, to *secondary compression* which takes place at the same time as the much larger primary consolidation. The role of secondary compression can be gauged by reference to Fig. 9.5, presented by Bjerrum (1972) for the relationship between void ratio, e, and effective stress, σ'_v. When the load p_f+p_s is applied to the clay surface, the path *abd* will be followed, but the total compression that occurs can be envisaged by a primary consolidation along path *abc* together with a secondary compression along

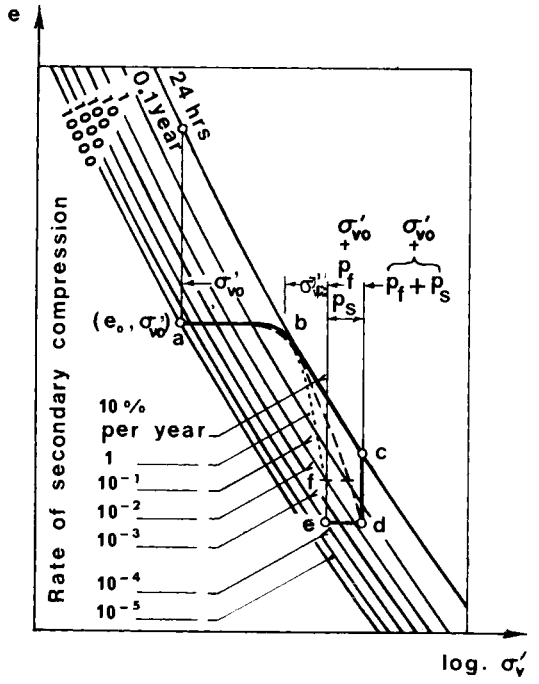


Fig. 9.5. Secondary compression caused by preloading.

path *cd*. Removal of the overload, p_s, moves the clay along path *de*. If, on the other hand, the design load, p_f, is applied alone, the path *bf* is followed while primary consolidation is occurring, and secondary compressions along *fe* take a very long period of time. This figure illustrates clearly that both the magnitude and rate of secondary compression under the design load are reduced by the prior application of an overload. For these effects to be of practical significance, however, the overload must remain in position until well after the excess pore pressures in the layer have dissipated (i.e. until after primary consolidation has ceased).

The effect of preloading on the secondary compression of a normally consolidated soil can be calculated approximately in much the same way as for primary consolidation, on the basis of the two assumptions (Johnson, 1970):

(1) Primary consolidation of magnitude ρ_f is completed under the load p_f in time t_p.

(2) After primary consolidation is completed, secondary compression occurs at time t_s at a rate ρ_s, in accordance with the law:

$$\rho_s = C_\alpha H_p \log \frac{t_s}{t_p} \tag{9.7}$$

where H_p is the thickness of the layer at time t_p, and C_α is the coefficient of secondary compression.

The settlement behaviour of a clay layer under the loads p_f and p_f+p_s is illustrated in Fig. 9.6. If no primary or secondary compressions are to occur under the design load, p_f, the load p_f+p_s must be sustained for a period t_{sR}, such that the total settlement, ρ_{sR}, is the sum of the primary and secondary

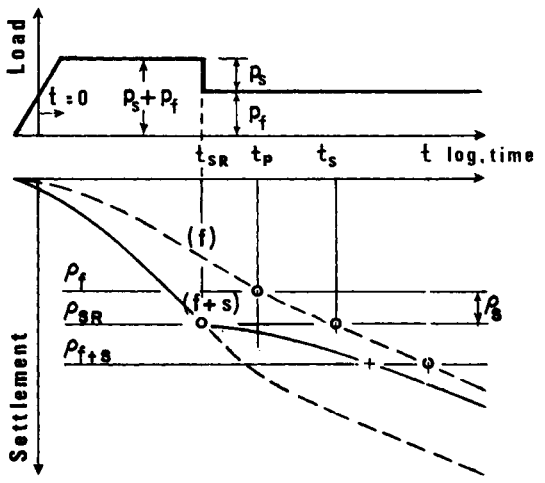


Fig. 9.6. Determination of preload time necessary to eliminate secondary compression under design load.

settlement that would have occurred under the load p_f alone. That is (Johnson, 1970a):

$$\rho_{sR} = \rho_f + \rho_{sc} \tag{9.8}$$

where:

$$\rho_f = \frac{2H}{(1+e_o)} C_c \log \left(\frac{\sigma'_{vo}+p_f}{\sigma'_{vo}}\right) \tag{9.9}$$

$$\rho_s = (H-\rho_f)\, C_\alpha \log \frac{t_s}{t_p} \tag{9.10}$$

The desired settlement, ρ_{sR}, can then be related to the *primary* settlement under the load p_f+p_s by:

$$\rho_{sR} = U_{f+s}\, \rho_{f+s} \tag{9.11}$$

and substitution of Eqs. 9.9, 9.10, 9.11 & 9.1b into Eq. 9.8 leads to the solution:

$$U_{f+s} = \frac{\left(1-C_\alpha \log \frac{t_s}{t_p}\right) \log \left(1+\frac{p_f}{\sigma'_{vo}}\right) + \frac{C_\alpha}{C_c}(1+e_o) \log \frac{t_s}{t_p}}{\log \left[1+\left(\frac{p_f}{\sigma'_{vo}}\right)\left(1+\frac{p_s}{p_f}\right)\right]} \tag{9.12}$$

The overload time, t_{sR}, can then be readily calculated from U_{f+s}. The secondary compression time, t_s, is selected in relation to the life of the structure or as the time required to give a desired magnitude of secondary compression compensation.

9.2.4 Undrained strength gain by preloading

The compressions which occur under preloading result in increases in the undrained shear strength of a clay. The manner in which primary and secondary compressions contribute to this strength gain is illustrated in Fig. 9.7 (Bjerrum, 1972), which shows e-log σ'_v relationships for a clay together with the relationship between undrained shear strength, s_u, and the effective stress, σ'_v. The effects of loading the surface of the clay can be summarized with reference to this figure, thus:

(1) If the loading, p_f, does not exceed the critical pressure, σ'_{vc}, (i.e. if path *ab* is followed), no strength gain is experienced, since the void ratio does not change measureably.

(2) Primary consolidation along path *bc* results in a strength increase from *B* to *C*.

(3) Secondary compression along path *cd* results in a strength increase from *C* to *D*.

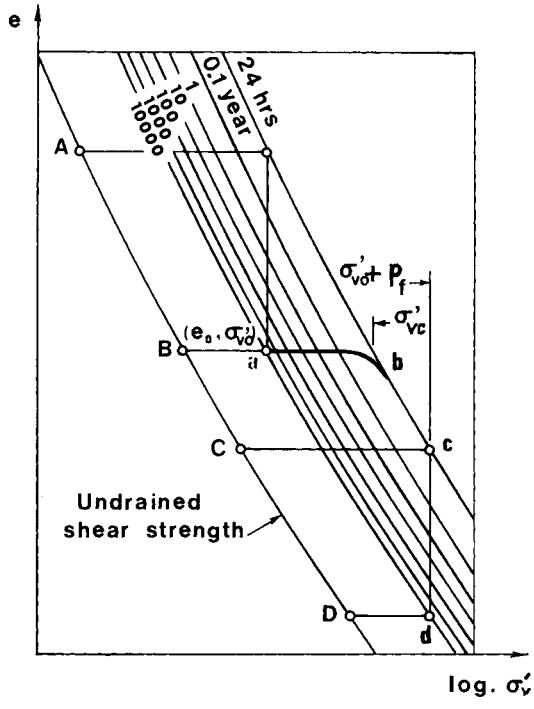
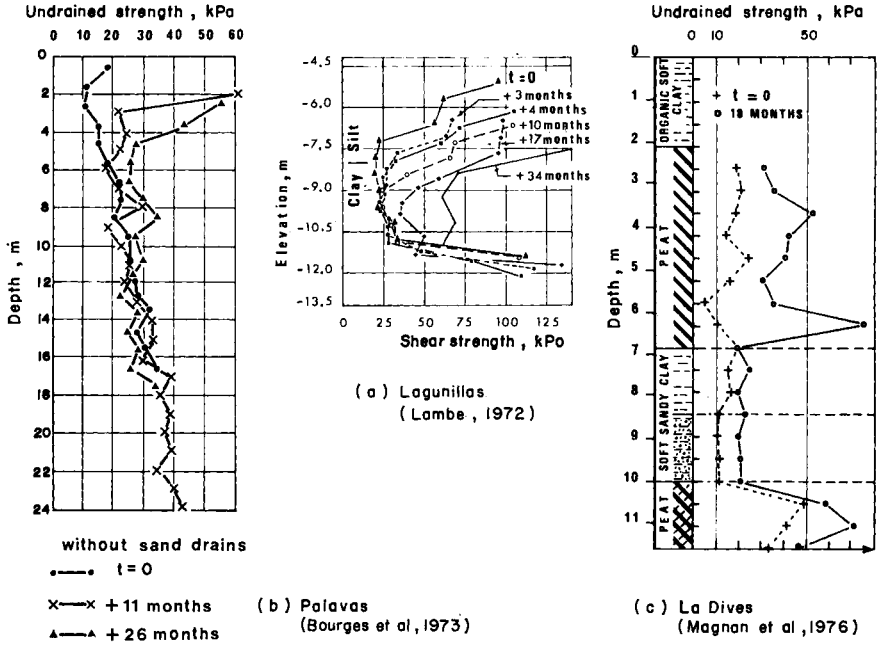


Fig. 9.7. Gain in undrained shear strength caused by preloading.

Strength gains due to primary consolidation are generally large compared to those caused by secondary compression. Because primary consolidation is generally not completed during the period for which an overload is applied, however, the distribution of strength increase can be highly variable throughout a clay layer when the overload is removed.

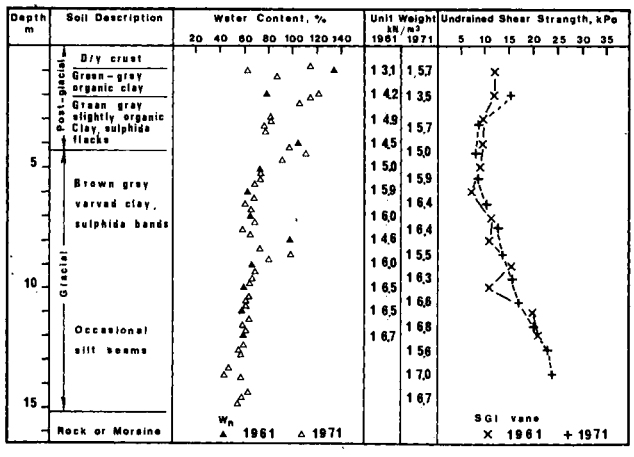
The prediction of strength increases resulting from consolidation are generally based on field established relationships between plasticity index and the ratio s_u/σ'_{vo}, or on undrained strength envelopes obtained from consolidated undrained triaxial tests; triaxial tests generally indicate the strength gain, Δs_u, to be related to the increase in consolidation pressure, $\Delta\sigma'_v$, approximately by $\Delta s_u = 0.2\ \Delta\sigma'_v$. There are several published records of strength gains under fills. Rico et al (1969) reported that the strength of a thick layer of soft volcanic clay increased from 15 kPa to 23 kPa under the centre-line of an embankment placed for 30 months. Lambe (1972) described how an overload of 230 kPa on a 7 m thick layer of silt and soft clay at a reservoir site at Lagunillas, Venezuela, brought about up to 100% strength increase in three years (Fig. 9.8a). Considerable strength increases measured by field vane in a 24 m thick soft silty clay layer were reported by Bourges et al (1973) (Fig. 9.8b) at a location at Palavas, France, where the applied load was increased from



(a) Lagunillas (Lambe , 1972)

(b) Palavas (Bourges et al , 1973)

(c) La Dives (Magnan et al , 1976)



(d) Skå Edeby test embankment (Holtz & Lindskog , 1972)

Fig. 9.8. Examples of the effects of preloading.

75 kPa to 130 kPa in eleven months; after 26 months, the shear strength gain varied from 30 to 50 kPa in the upper few metres to virtually nothing below the middle of the layer. At La Dives, France, Magnan et al (1976) noted appreciable improvements in a peat/soft clay complex after it had been for 18 months under a load of 120 kPa (Fig. 9.8c).

There are some recorded instances of no strength gains having occurred even though considerable compressions were observed. Under the well-known Skå-Edeby full-scale test embankment in Sweden, Holtz & Lindskog (1972) reported that, after ten years, insignificant strength gains had occurred even though a ground surface settlement of 0.6 m had been recorded (Fig. 9.8d). It is probable in this case that the recorded settlement was the result of horizontal creep of the soft clay, since only small water content decreases were measured in the clay and the factor of safety of the embankment against failure was estimated to be as low as 1.5.

9.2.5 Case histories of preloading

Only a few case histories exist of the performance of soft clays treated by means of preloading. Table 9.1 summarizes the recent cases reported in the literature. Among other things, the table shows the available data on the surface heave that occurred when the overload was removed, and the additional settlement that took place after that time. The success of the preloading method is clearly illustrated.

The Author is particularly familiar with the Arles preloading project (Simon & Brigando, 1976) summarized in Table 9.1, and a brief discussion of this will prove useful. Where the Route Nationale 113 bypasses Arles, it runs over recent alluvium which consists of soft clay and peaty clay 10 to 15 m deep, the main properties of which are shown in Fig. 9.9. The original design of a two-span bridge on this material adopted deep foundations, but their high cost led to a decision to use shallow foundations after preloading the site with a 100 m long and 50 m wide embankment of sand and gravel ($\gamma = 22 kN/m^3$). Stability requirements dictated a three-stage construction of the embankment to a height of 8 m, removal of the load taking place after 611 days (Fig. 9.9). The main results of the preloading were:

(1) The engineering properties of the soft soil were greatly improved, the undrained shear strength being doubled in 320 days.
(2) The final settlement of the fill of 1.2 m was greater than the predicted 0.8 m because of the extensive lateral displacements which occurred at the low safety factor of 1.3.
(3) The soil heaved appreciably when the fill was removed and continued to heave during the construction of the bridge foundations.

Because of the effects of the preloading, the bridge has settled only 60 mm during its working life.

Table 9.1. Some case histories of preloading by fill

Location and reference	Soil description	Performance data					
		Permanent load, ρ_f (kPa)	Surcharge load, ρ_s (kPa)	Time to overload removal, t_{sR} (months)	Total settlement under overload, ρ_{sR} (m)	Heave on removal of overload (m)	Additional settlement after removal of overload (m)
Fukuroi, Japan (Ueda, 1971)	Recent, fine alluvium, 20 m thick; w_n=30 to 60%, s_u=50 to 200 kPa, c_v=2.5 to 4.5 × 10^{-7} m^2/sec.	42	84	7	0.39	0.04	≈0
Tampa, USA (Wheeless & Sowers 1972)	Sandy clay, 6 m thick under 5 m of sand	41	174	3.5	0.07	?	≈0
Lagunillas, Venezuela (Lambe, 1972)	Clay and silt, about 10 m thick; s_u=25 to 100 kPa	?	225	36	1.10	?	?
Berthierville, Canada (Samson & Garneau, 1973)	Silty clay, about 60 m thick	160	160	11	1.20	0.04	?
Arles, France (Simon & Brigando, 1976)	Clay and peat, 11 m thick; w_n=20 to 120%, s_u=10 to 20 kPa	?	176	20	1.20	?	0.06

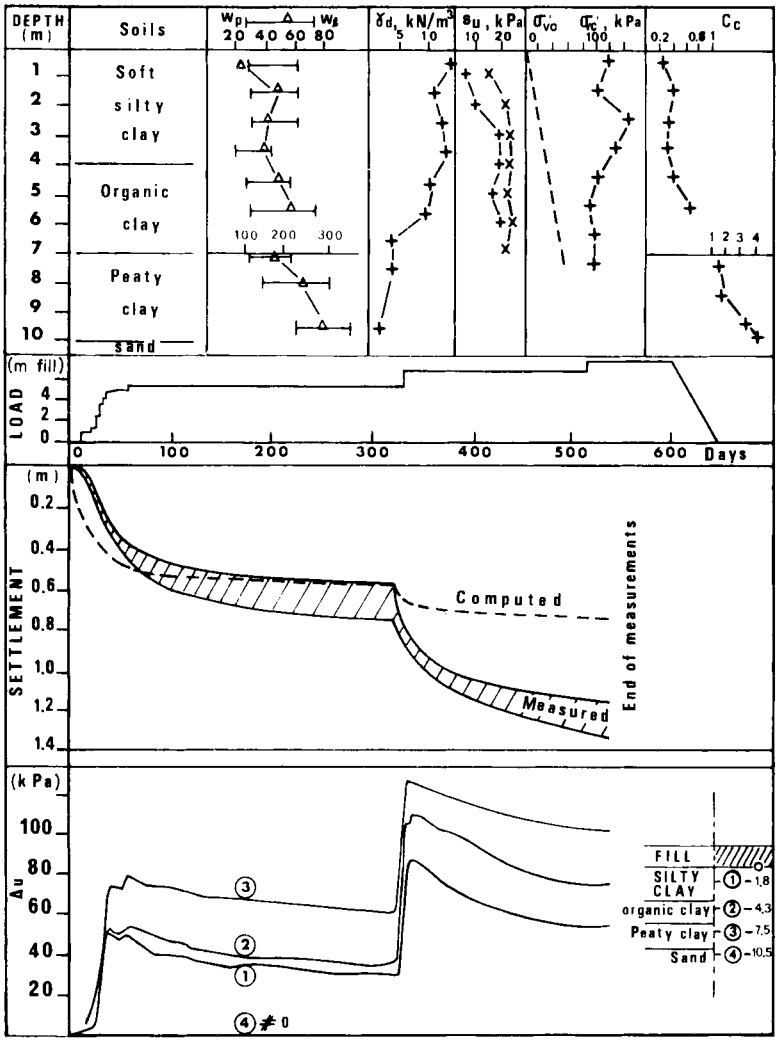


Fig. 9.9. Preloading at Arles (Simon & Brigando, 1976).

9.2.6 *Vacuum preloading*

After carrying out a series of in-situ tests in the 1940s, Kjellmann (1952) finally announced his vacuum preloading method which was then adopted by others. A 150 mm layer of sand is placed on the surface of the soft clay, the layer is covered with an impervious membrane, and a vacuum of 60 to 80 kPa is induced in the sand which acts as an equivalent overload. Vertical drains are often used in conjunction with this method (Fig. 9.10a).

The advantages of the vacuum method are:

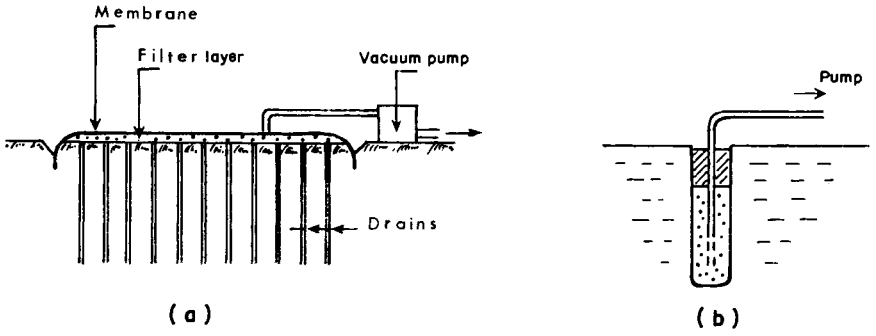


Fig. 9.10. Vacuum preloading: (a) by surface membrane and drains, (b) by vaccuum wells.

(1) there are no problems of embankment stability to consider,
(2) it eliminates the need for backfill material which is usually expensive and often unavailable, and
(3) the installation and removal of the means of applying the preload are readily accomplished.

The method is cheaper, therefore, than the use of fills for preloading. The major difficulty with the method used to revolve around problems of membrane strength and service life, but these problems have now been overcome (Holtz & Wager, 1975).

An alternative to the use of a surface membrane is the provision of a series of wells to which the vacuum is applied (Fig. 9.10b). This vacuum well method can be carried out concurrently with pumping from deep wells that penetrate any available deep, more permeable layers.

In spite of the obvious advantages of vacuum preloading, there appear to be only four comparatively recent published case histories; these are summarized in Table 9.2. In each case, because of the existence of very soft sediments, the use of embankment fills for preloading would have posed serious stability problems.

9.3 VERTICAL DRAINS

9.3.1 General principles

When a layer of soft clay is very thick or the permeability is exceptionally low, the preloading technique is likely to be inefficient when used alone because an inordinately long period of time will be needed to bring about significant compressions. In these circumstances, radical improvements in the preloading time can be effected by the installation of vertical drains to shorten the drainage path under which the clay will consolidate (Fig. 9.11). This is now a commonly employed device with soft clays.

Table 9.2. Case histories of vacuum preloading

Location, structure, reference	Soil description	Performance data	
		Technique employed	Settlement, ρ_{sR} (m)
Alfortville, France; gas-holder foundation. (Solétanche, 1960)	Power station soot, 3 m thick, over sand-gravel mixture.	Membrane method, applied in two stages:	
		(1) 40 to 90 kPa for 40 days	0.24
		(2) 40 kPa, 3 years later, for 60 days concurrently with filling of reservoir	0.10
Philadelphia, USA; runway. (Halton et al, 1965)	Silt and organic clay, 7 m thick, over sands and gravels.	Well method (without membrane), in conjunction with pumping from well-points and sand drains; maximum 40 kPa negative pressure applied for 40 days.	0.18
Inland Sea, Japan; reservoir foundation. (Teshima & Ogawa, 1967)	Clay and silt, 6 m thick, under layer of fine sand lying over sands and gravels.	Well method (without membrane) in conjunction with pumping from well-points and sand drains; vacuum applied for 60 days.	0.3 to 0.5
Brest, France; hydraulic fill. (Paute, 1970)	Clayey silt fill, 6 m thick, over 2 m of slime lying over sands and gravels.	Well method (without membrane); 20 to 45 kPa negative pressure applied for about 30 days.	
		— wells with 10 m spacing	0.03 to 0.12
		— wells with 3.3 m spacing	0.19 to 0.42

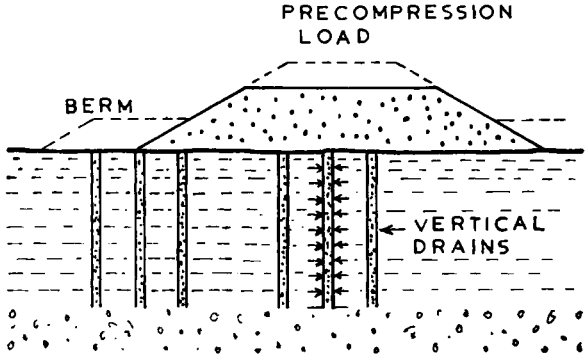


Fig. 9.11. Vertical drain application.

The benefits of vertical drains used in conjunction with preloading are related almost entirely to the acceleration of the primary consolidation of the clay, since they bring about the rapid dissipation of excess pore water pressures. Vertical drains have no direct effect on the rate of secondary compression, but the early completion of primary consolidation brings about the earlier occurrence of significant secondary compressions.

Although the concept of vertical drains was advanced more than a century ago (Moreau, 1832), the method was not applied until the 1930s. Since that time, however, the method has been widely used, and great advances have been made in the way in which the concept is applied to practical problems. The literature on the subject is plentiful, and Magnan (1981) listed over 300 references in a recent bibliography. This section will be limited, therefore, to considerations of the more important published literature and to information published since about 1970; the older literature has been abley summarized in the paper by Johnson (1970b) to which reference should be made. The abundance of literature on vertical drains is a reflection of the widespread use of the technique, the many developments that have been reported, and the complexities of the practical details of the many varieties of drain.

9.3.2 Efficiency of vertical drains

The efficiency of a system of vertical drains is assessed in terms of the acceleration achieved in primary consolidation over the consolidation rate that would have occurred without the drains. In a preloading operation with drains, therefore, the following two factors are of as great importance as they are where drains are not used:

(1) The stress level under the applied load, $\sigma'_{vo}+p$, must be appreciably greater than the critical pressure, σ'_{vc} (Fig. 9.12).

(2) The ratio of the primary settlement, ρ_{pc} to the secondary compression, ρ_{sc}, should be large.

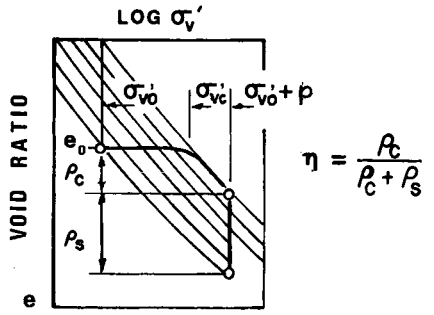


Fig. 9.12. Definition of vertical drain efficiency.

The efficiency factor, η, of vertical drains has been expressed by Bjerrum (1972) as:

$$\eta = \frac{\rho_{pc}}{\rho_{pc} + \rho_{sc}} \tag{9.13}$$

where ρ_{pc} and ρ_{sc} are evaluated as shown in Fig. 9.12. This leads to the expression:

$$\eta = \frac{\int_0^z [\log(\sigma'_{vo} + p) - \log \sigma'_c]\, dz}{\int_0^z [\log(\sigma'_{vo} + p) - \log \sigma'_{vo}]\, dz} \tag{9.14}$$

According to Bjerrum (1972), the satisfactory performance of drains at five sites established the satisfactory value of η as between 0.6 and 0.8.

The need for, and effectiveness of, vertical drains depends to a great extent on the properties of the soil which, in turn, depend largely on geological factors. Of primary importance are the soil permeability and coefficient of consolidation and their variations in space and time. Deposits of recent clays often exhibit horizontal stratification, with frequent thin seams of sand or silt of relatively high permeability. The importance to be ascribed to these macro-structural features has been emphasized by Rowe (1968), who noted that:

(1) Thin lenticular strata of high permeability greatly increase the efficacy of drains, since they act as horizontal drains attached to main arteries.
(2) Continuous thick seams of high permeability material at sufficiently close spacings often render vertical drains unnecessary or greatly reduce their real effectiveness.

Rowe (1968) concluded that the efficiency ascribed to vertical drains is often unwarranted, because inadequate pre-installation investigations had failed to

reveal the presence of horizontal drainage layers within the clay. He advised that an adequate geotechnical investigation when drains are being contemplated should include continuous core sampling of the soil strata, in-situ permeability measurements at low hydraulic heads, and laboratory consolidation tests on large diameter specimens.

The decision as to whether or not vertical drains are employed to aid the rate of settlement of a fill will depend on a number of factors, as will the selection of the particular type of drain. The five types of drain available are: (1) sand drains, (2) cardboard drains, (3) sand wicks, (4) plastic drains, and (5) drainage trenches. The first four of these are discussed in Section 9.3.3, drainage trenches being omitted because they are virtually never used. Although the comparative costs of drain installations are of paramount importance in the selection process, there are other factors that relate to drain performance which must be considered (Rowe, 1968), namely:

(1) smear and distortion of the drain walls, which reduce drain permeability, and
(2) disturbance and lateral deformations of the soft ground resulting from drain installation, which reduce the permeability, coefficient of consolidation and undrained strength of the soil while increasing the pore pressures.

9.3.3 Types of vertical drain

9.3.3.1 Sand drains

Sandd rains have been widely used for over forty years. They range in diameter from 180 to 450 mm, and they have been installed to great depths by a great variety of procedures, the most common of which are the closed-mandrel and open-mandrel methods.

Closed mandrels, which consist of steel tubes closed at the lower end by a loose cap, are driven by percussion or vibration or sometimes are jetted into place before being filled with sand and the tubes extracted. This is a simple inexpensive method which is very popular. Its major drawback, however, is the disturbance caused to the soil by the displacement during installation, which results in decreased strength and permeability and increased settlement. An assessment of the disturbance effect can be made by measuring pore pressure changes, in-situ strength decreases and surface heaves.

Open mandrels are installed by the same means as closed mandrels, the soil in the tube being removed by jetting or by means of an auger before sand is placed in the hole as the tube is removed. Significantly less disturbance results from the installation process than with the closed mandrel method, but the problem of smear still exists.

Although some extensive investigations have been carried out to examine the installation procedures and performance of sand drains (Hansbo, 1960;

Landau, 1966; Johnson, 1970b), the decision as to the working method that should be adopted in a particular situation largely depends upon local conditions and experience.

9.3.3.2 Cardboard drains

The first to suggest the use of driven cardboard drains to replace sand drains was Kjellmann (1948), who had carried out tests over several years. These first cardboard drains were 100 mm wide and 3 mm thick (Fig. 9.13), but other sizes have been used subsequently. They are inserted into the ground by means of a purpose-made mandrel which is then removed. Channels in the cardboard (which occupy about 10% of the cross-sectional area) facilitate the passage of water from the clay to the ground surface. A 100 mm × 3 mm cardboard drain is about equivalent to a 50 mm diameter sand drain.

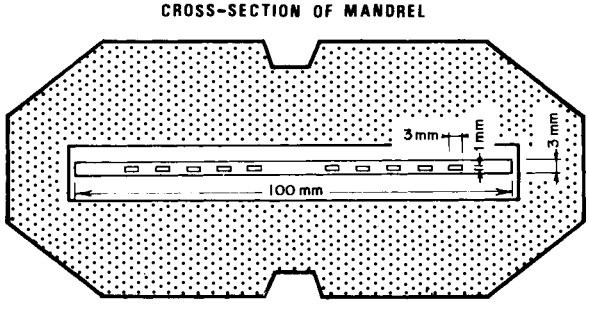


Fig. 9.13. Cross-section of cardboard drain and insertion mandrel.

Cardboard drains have the advantage of ease of installation, which also causes relatively little soil disturbance. They can be very closely spaced if required. The specially processed cardboard has long life, but there is some question about the ability of this material to withstand the large deformations which often result in soft soils.

9.3.3.3 Sand-wicks

Sand-wicks are ready-made small diameter sand drains which are contained in long canvas bags. The most common diameter is about 100 mm. They are usually installed by the closed-mandrel technique described in Section 9.3.3.1 above.

Sand-wicks were first used in India by Dastidar et al (1969), and their satisfactory use in that country has been further described by Subbaraju et al (1973). This type of vertical drain is economical where labour costs are low, since the making and filling of the canvas bags is generally done by hand, and installation in soft clay can be accomplished entirely by hand labour.

9.3.3.4 Plastic drains

Plastic drains have recently been introduced to replace cardboard drains as the most popular 'manufactured' vertical drains. Particularly well-known is the *Geodrain* (Fig. 9.14), which consists of a 100 mm wide × 3 mm thick paper-covered polythene strip which contains channels along both sides (Boman, 1973; Terrafigo, 1976). This configuration achieves a drainage channel cross-sectional area of 220 mm², or more than 70% of the total drain area. The exterior filter paper is chemically impregnated for long life. The mandrel used for installation of Geodrains is shown in Fig. 9.14.

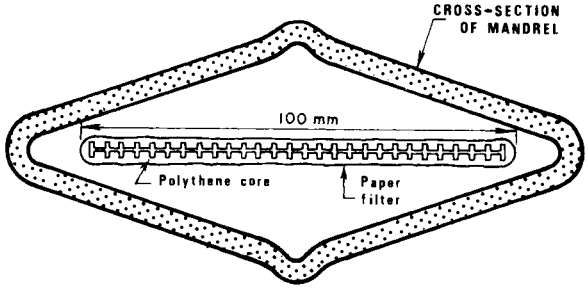


Fig. 9.14. Cross-section of plastic Geodrain and insertion mandrel.

The efficiency of Geodrains has been established at various test sites (Hansbo & Torstensson, 1977), including the well-known Skå-Edeby site. Comparison with sand drains showed that relatively little disturbance occurs with the plastic drains, and that large settlements do not destroy drain continuity. Geodrains at 0.9 m centres were found to be as efficient as 0.18 m diameter sand drains at the same centres (Fig. 9.15).

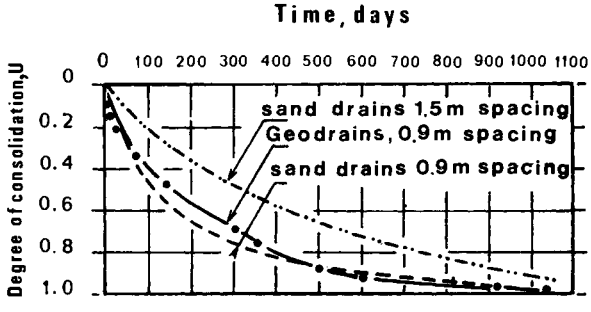


Fig. 9.15. Comparative efficiencies of Geodrains and 0.18 m diameter sand drains.

9.3.4 Design of vertical drains

The theoretical design of vertical drains is based upon the independent behaviour of each drain in the centre of a cylindrical soil mass (Fig. 9.16). The

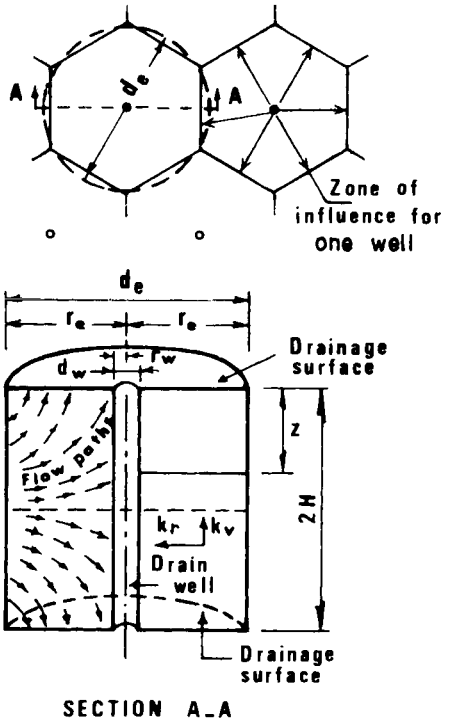


Fig. 9.16. Design principles for vertical drains.

many theories that have been proposed over the last 40 years have been based upon various assumptions about the homogeneity of the soil, the variations with time of the permeability and coefficient of consolidation, the appropriate hydraulic flow law, drain effects (smear, disturbance and well resistance), loading rate, and creep effects. A review of the many assumptions made in the many relevant publications is not possible within the confines of this chapter, but the assumptions made in the recent literature are summarized in Table 9.3. It can be seen that the majority of the publications have dealt with the simplest case of instantaneous load application on a homogeneous soil, without drain effects.

9.3.4.1 Basic solution for instantaneous loading

The basic design method is that proposed by Barron (1947), in which instantaneous loading of a homogeneous soil results in solely radial consolidation (and, therefore, radial flow) under conditions of constant permeability, k_h, and radial consolidation coefficient, c_r. The equation which governs the relationship between the pore pressure, u, the radial distance from the drain, r, and the time, t, is:

Table 9.3. Theoretical solutions for vertical drain performance

			Homogeneous soil												Heterogeneous soil			
			$v=ki$								$v=ki_m$		$v=k(i-i_o)$		$v=ki$			
			Incompressible fluid				Compressible fluid				Incompressible fluid				Incompressible fluid			
			Constant k & c_r		Variable k & c_r		Constant k & c_r		Variable k & c_r		Constant k & c_r		Variable k & c_r		Constant k & c_r		Variable k & c_r	
			No creep	Creep	No creep	Creep	No creep	Creep	No creep	Creep	No creep	Creep	No creep	Creep	No creep	Creep	No creep	Creep
Instantaneous loading	No smear	No well resistance	3	4 9	4 5 11 15 18	16 17	9 19		8	8	7		1		10 14			
		Well resistance	2 3 20															
	Smear	No well resistance	3 4 12 13															
		Well resistance																
Progressive loading	No smear	No well resistance	6 15		6	16 17												
		Well resistance																

Legend

1. Abelev (1973) 2. Aboshi et al (1969) 3. Barron (1947) 4. Berry & Wilkinson (1969) 5. Botea & Nguyen (1971) 6. Chaput & Thomann (1975) 7. Hansbo (1960) 8. Hoang (1975) 9. Kulkarni (1973) 10. Lee & Valiappan (1974) 11. Raymond (1969) 12. Reddy & Murthy (1969) 13. Richart (1959) 14. Rowe (1964) 15. Schiffmann (1968) 16. Suklje & Kožak (1972) 17. Suklje & Simončič (1972) 18. Tan & Simons (1971) 19. Tsytovich et al (1971) 20. Yoshikuni & Nakanado (1974)

$$c_r \left(\frac{\partial^2 u}{\partial r^2} + \frac{1}{r} \frac{\partial u}{\partial r} \right) = \frac{\partial u}{\partial t} \tag{9.15}$$

Also, $u = u_o$ at $t = o$ throughout the soil, and $u = o$ in the drain at any time. The two boundary conditions for which solutions can be obtained are: (1) uniform vertical surcharge on the ground surface ('free strain' condition), and (2) uniform vertical deformation of the surface ('equal strain' condition). The solutions for these two conditions, in fact, give similar results, but that for unform vertical strain lends itself to simpler numerical manipulation. The radial degree of consolidation for this uniform strain condition is:

$$U_r = 1 - \exp \left[- \frac{8T_r}{F(n)} \right] \tag{9.16}$$

where:

$$T_r = \frac{c_r t}{d_e^2} = \text{radial time factor,}$$

$$F(n) = \left(\frac{n^2}{n^2 - 1} \right) \ln n - \left(\frac{3n^2 - 1}{4n^2} \right)$$

$$n = \frac{d_e}{d_w} = \text{ratio of drain spacing to drain diameter.}$$

The variation of U_r with the radial time factor is shown in Fig. 9.17 for a range of n values.

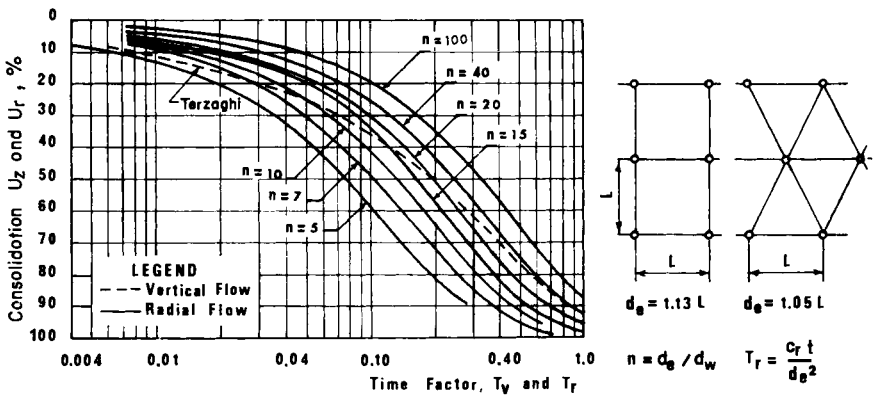


Fig. 9.17. Theoretical solution for the consolidation of clay containing vertical drains (Barron, 1947).

Consolidation of the cylindrical body of soil around a vertical drain is, in fact, three-dimensional and is governed by the equation:

$$c_r \left(\frac{\partial^2 u}{\partial r} + \frac{1}{r}\frac{\partial u}{\partial r}\right) + c_v \frac{\partial u}{\partial z^2} = \frac{\partial u}{\partial t} \tag{9.16}$$

Carillo (1942) had showed, however, that the overall (three-dimensional) degree of consolidation, U^*, was given by:

$$1 - U^* = (1 - U_r)(1 - U_z) \tag{9.17}$$

where U_r is given by Eq. 9.16, and U_z is the average vertical degree of consolidation from Terzaghi's one-dimensional consolidation solution. Terzaghi's curve of U_z versus the vertical time factor, T_v, is shown in Fig. 9.17 together with values of U_r.

9.3.4.2 Solution for gradually applied loading

Early theoretical studies of vertical drains assumed instantaneous loading, but the practical situation is one of gradual loading, and the prediction of the amount of consolidation during the loading stage is of considerable importance. A solution for linearly increasing load has been given by Chaput & Thomann (1975); this is represented in Fig. 9.18. If the average degree of consolidation, U, is defined as the ratio of the settlement at time t when the load reaches $p(t)$ (Fig. 9.18a) to the ultimate settlement that would result from $p(t)$ applied instantly, then during the load application ($o \leqslant t \leqslant t_o$):

$$U = 1 - B(n)\, C\, (\lambda^2 n^2 T_r, T_v) \tag{9.18}$$

where λ_1, $B(n)$ and C are given in the nomagrams in Figs. 9.18b, 9.18c & 9.18d respectively. Once the total load has been applied ($t > t_o$):

$$U = 1 - B(n)\, C\, (\lambda_1^2 n^2 T_{ro}, T_{vo})\, D\, [\lambda_1^2 n^2 (T_r - T_o), (T_v - T_{vo})] \tag{9.19}$$

B & C being obtained as before for $t = t_o$, and D is given in the nomogram in Fig. 9.18e.

9.3.4.3 Effects of soil and drain properties

Most theories relating to vertical drains assume the pore fluid to be incompressible (Table 9.3), whereas the creep of the soil skeleton has been well considered. The soil properties of radial permeability, compressibility and coefficient of consolidation have been considered as constants or have been assumed to vary in some rational fashion with time.

Darcy's law has generally been used for drain theories, with the exceptions of those by Hansbo (1960), who used $v = ki_m$ ($m = 1.5$ was obtained experimentally), and Abelev (1973), who assumed an initial hydraulic gradient of i_o and $v = k(i - i_o)$.

The effects of the drains themselves are a result of: (1) circumferential disturbance and smear due to installation, and (2) the resistance of the drain

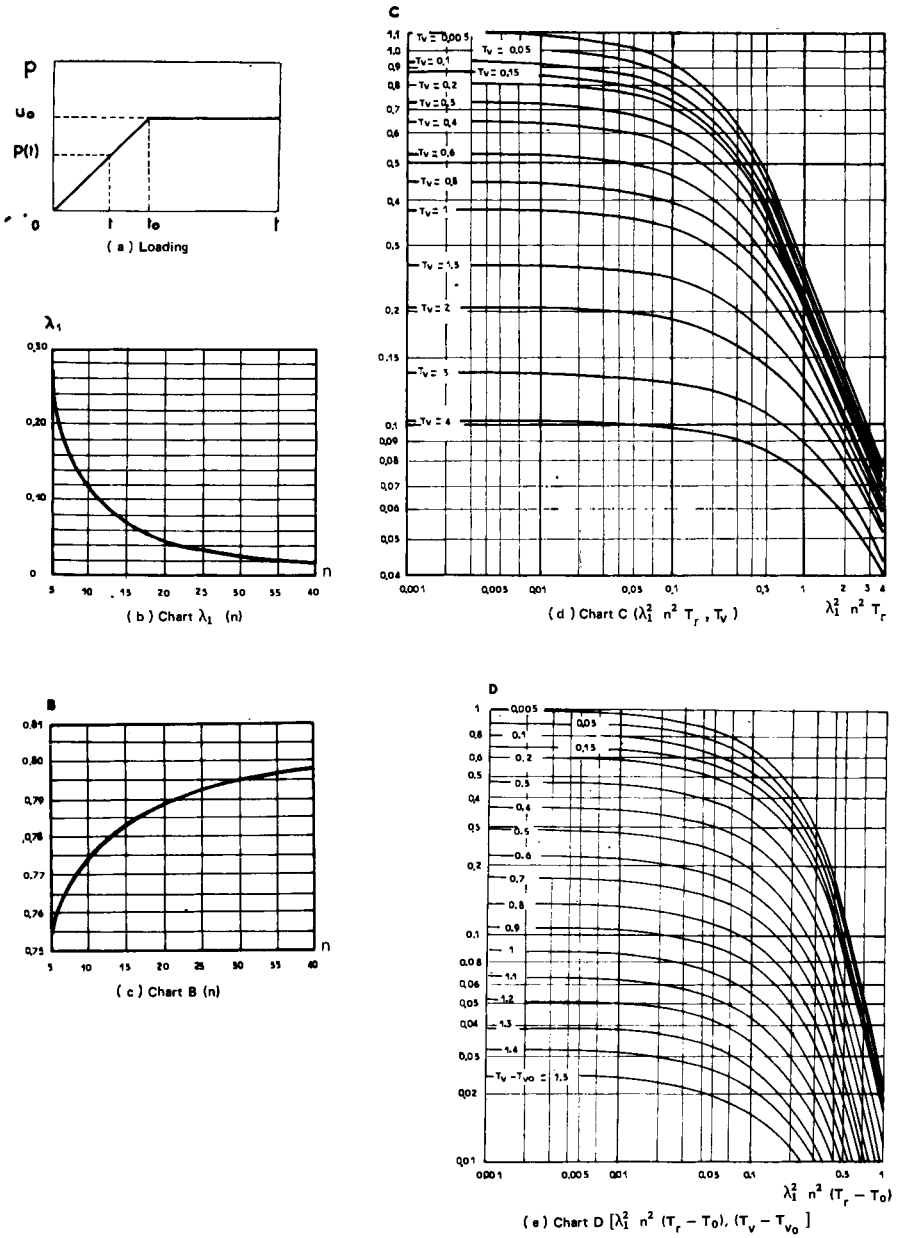


Fig. 9.18. Nomograms for the functions required for the design of vertical drains under progressive loading conditions (Chaput & Thomann, 1975).

to flow. Theories generally assume the drain to be of highly permeable material which provides no resistance to flow, but the resistance can be taken into account (Barron, 1948; Aboshi et al, 1969; Yoshikuni & Nakanodo, 1974). Soil disturbance and drain smear have also been allowed for theoretically.

Circumferential disturbance and smear can be accounted for by considering them to result in the formation of a cylindrical tube of soil with permeability lower than that of the surrounding soil. Because of this tube of reduced permeability, radial flow is inhibited, and the rate of consolidation is decreased. Barron (1947) recognized this problem, and theoretical allowance was made for it by Richart (1959) and Berry & Wilkinson (1969).

The resistance offered by the drains to the vertical flow of water affects the rate of consolidation. Barron studied this effect on radial consolidation, while Yoshikuni & Nakanodo (1974) dealt with three-dimensional consolidation. The results of Yoshikuni & Nakanodo are shown in Fig. 9.19; the notation is defined in Fig. 9.19a, values of U_r are given in Fig. 9.19b for a specific situation, and Figs. 9.19c & d present the times required to reach 50% & 80% degrees of consolidation for a wide range of situations.

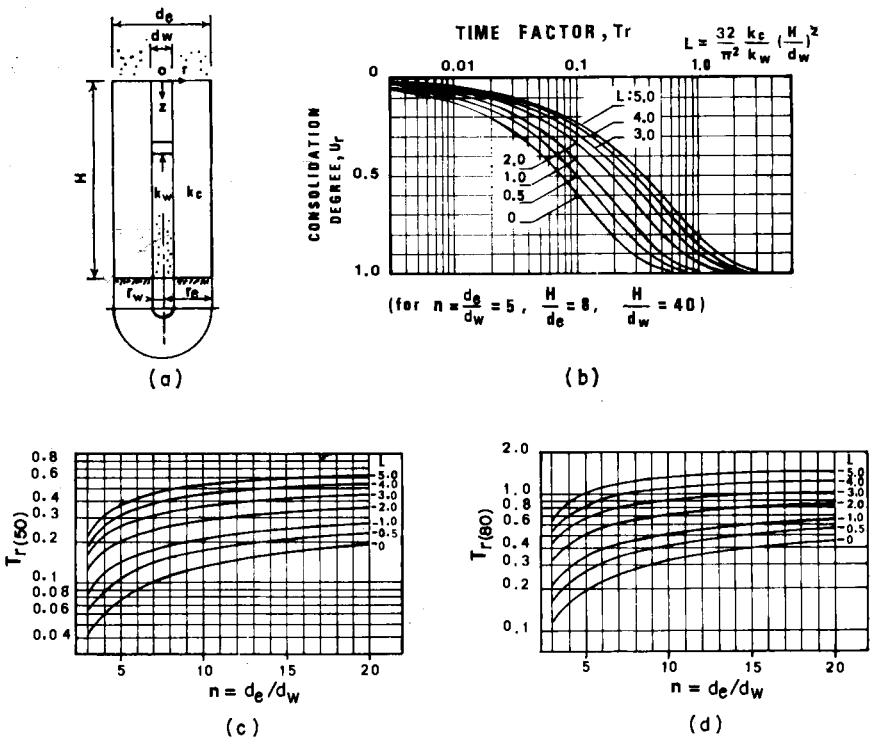


Fig. 9.19. Effect of drain resistance on consolidation rate (Yoshikuni & Nakanodo, 1974): (a) notation, (b) values of U_r, (c) values of $T_{r(50)}$, (d) values of $T_{r(80)}$.

The presence of stratifications in the form of thin seams of silt or sand causes consolidation to occur faster than predicted on the basis of laboratory oedometer tests. In addition to the well-known papers by Rowe (1964, 1968) which consider the influence of stratifications, Lee & Valiappan (1974) produced the nomograph in Fig. 9.20 to illustrate these effects.

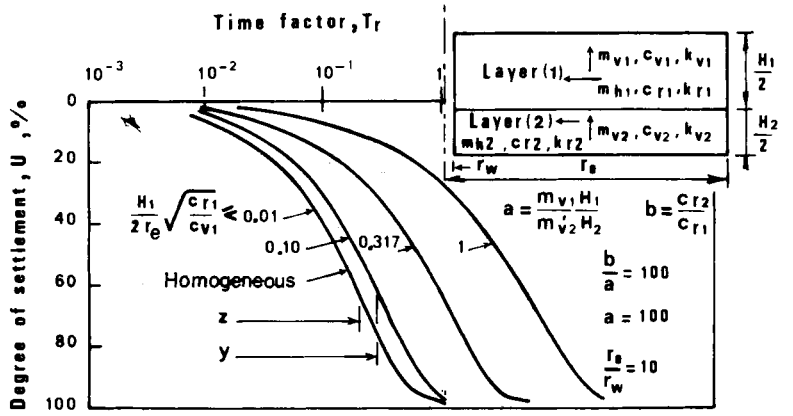


Fig. 9.20. Effect of stratification on vertical drain performance (Lee & Valiappan, 1974).

9.3.5 Soil strength gain

The increase in undrained shear strength, s_u, that results from the primary consolidation that takes place when vertical drains are used in conjunction with a surface load, is the same as that which occurs under the surface load alone (see Section 9.2.4). The presence of the drains, however, causes a more rapid consolidation and consequent faster strength gain throughout a soft clay layer. This obvious assertion is supported by measurements under the Napa River embankment (Su et al, 1973) (Fig. 9.21) and the Palavas embankment (Bourges et al, 1973). At Palavas, vane tests performed on areas with and without vertical drains showed that, without drains, the strength gains were limited to the upper portion of the soft clay, whereas strength increases were measured throughout the clay stratum where drains were installed (Fig. 9.22).

9.3.6 Measurement of c_r

The design procedures for vertical drains all require a knowledge of the radial coefficient of consolidation, c_r, which links vertical compressibility and horizontal permeability, and which controls the radial flow of water into the drain. The accurate determination of this parameter, therefore, is of great importance. Both laboratory and field methods have been proposed for this. Values of c_r are commonly found to be from $2\,c_v$ to $10\,c_v$ because of horizontal stratification of the soil.

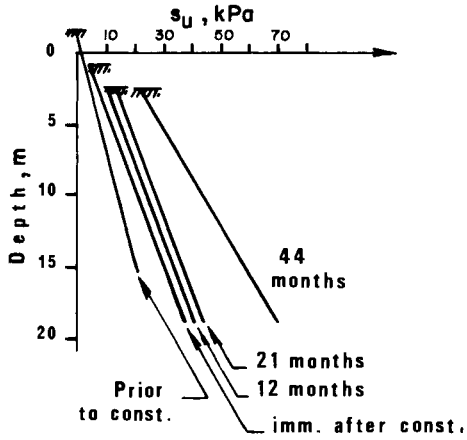


Fig. 9.21. Undrained strength gains at Napa River by use of sand drains (Su et al, 1973).

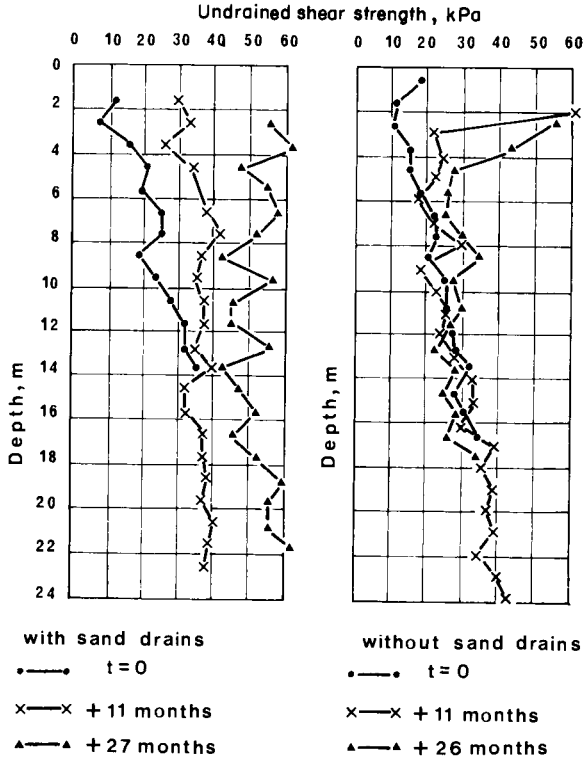


Fig. 9.22. Undrained strength gains at Palavas in areas with and without vertical drains (Bourges et al, 1973).

Laboratory tests to measure c_r employ an oedometer in which the clay specimen is placed with a vertical drain at its centre. Vertical loads are applied as in the conventional oedometer tests, and consolidation rate (and, if required, pore pressure dissipation rate) is measured. Extensive use of this method has been made by Hansbo (1960), Shields & Rowe (1965), Rowe & Barden (1966), Rowe (1968), Berry & Wilkinson (1969) and Paute (1973b).

The experience gained with laboratory measurements of c_r leads to the conclusions:

(1) Large oedometer specimens must be used to ensure that account is taken of stratification of a soil stratum. Rowe (1968) advocated that 254 mm diameter and 63 mm thick specimens be used, while Paute (1973b) reported the use of 141 × 40 mm specimens.

(2) The agreement between laboratory measurements and the theory of Barron (1947) improves as the degree of consolidation increases (Fig. 9.23) (Paute, 1973b).

(3) Variations in c_r and k_r during the consolidation process do not invalidate the application of Barron's theory to practical problems (Berry & Wilkinson, 1969).

(4) The efficiency of a drain design can be approximately assessed by means of laboratory tests.

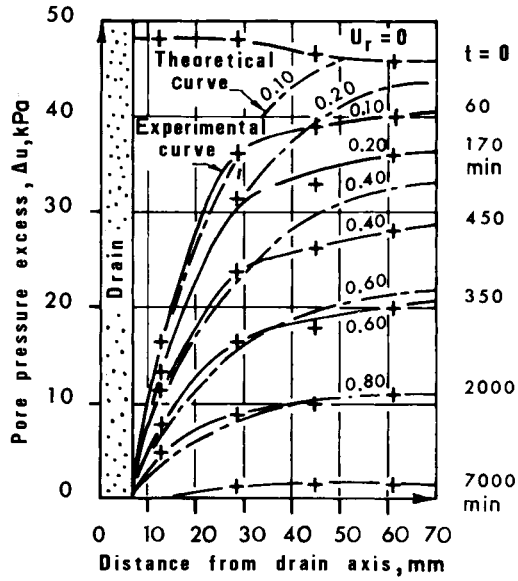


Fig. 9.23. Comparison between oedometer test results and Barron's theory for sand drains (Paute, 1973b).

Table 9.4. Case histories of vertical drain applications

Location and reference	Structure and soil description	Drain system	Comments on performance
Fukuroi, Japan. (Ueda, 1971).	Embankment 10 m high (of which 3 m were overload), on 5 to 8 m peat and 7 m soft clay (s_u=20 to 60 kPa).	Sand drains, 0.4 m diam. at 1.2 m centres.	Failure occurred at an embankment height of 6 m.
Massachusetts, USA. (Aldrich & Johnson, 1972).	Test embankment 6 m high, on about 18 m silty to organic clays.	1) Three test areas with sand drains 0.45 m diam. at 3 to 4.2 m centres: a) auger-bored drains, b) driven drains, c) jetted drains. 2) Test area with the same drains arranged differently.	With drains at 3 m, in-situ c_r values lower than for 4.2 m spacing: concluded that disturbances increased from jetted drains to auger-bored drains to driven drains. Settlements in driven-drain area were greater than in other areas.
San Francisco, USA. (Margason & Arango, 1972).	Embankment in the Bay, on 6 to 12 m soft clay. Observed for two years.	Jetted sand drains, 0.3 m diam. at 3 to 4.5 m centres.	Good results. Area with drains at 3 m settled 20 to 25% faster than that with drains at 4.5 m. Settlements agreed well with conventional theories, but not so pore water pressures.
Skå-Edeby, Sweden. (Holtz & Broms, 1972).	Test embankments, on about 10 m soft clay (s_u=5 to 10 kPa). Observed for 14 years.	1) Driven sand drains, 0.18 m diam. at 0.9 m, 1.5 m and 2.2 m centres. 2) Drainless reference area.	Disturbances due to the drains were small: close spacing more efficient. Results support Hansbo's theory with n=1.5.
Massachusetts, USA. (Ladd et al, 1972).	Approach ramp 10 m high, on 10 m chiefly soft and sensitive clay. Observed for two years.	1) Sand drains (jetted open-mandrel), 0.3 m diam. at centres from 2.7 to 4.8 m according to test area. 2) Drainless reference area.	Good results. Consolidation virtually completed within 200 to 400 days in various test areas, but U<70% in drainless area after 700 days. c_r/c_v in-situ found to be equal to 2, values of c_r and c_v being much higher than laboratory.
Napa River, USA. (Su et al, 1973).	Access ramp to a structure 7 m in height, on 15 m of soft silty clay. Observed for four years.	Driven sand drains, 0.45 m diam.	Good results. Greatly increased strengths compatible with increased pore water pressure and settlements.
Hampton Roads, USA. (Kuesel et al, 1973).	Precompression embankment 9 m high, on about 20 m soft clay. Observed for about	Jetted sand drains with layout pattern from 1.8×2.1 m to 2.7×3 m.	Good results. c_r found to be equal to c_v and, on this basis, measured settlements agreed with design calculations.

[illegible] (Croce et al, 1973).	[illegible] 30 m more or less organic clay.	[illegible] about 3 m centres.	[illegible] pore pressures of approx. 20 kPa that virtually vanished within 70 days.
Oxford, England. (Murray, 1973).	Embankment 3.4 m high, on 18 m soft clay. Observed for one year.	1) Driven sand drains, 0.3 m diam. at 1.8 m centres. 2) Drainless reference area.	Settlements occurred as quickly with or without drains, laboratory testing having underestimated c_r and c_v.
Vizag, India. (Subbaraju et al, 1973).	Stockpiled ore 9 m high, on 12 m soft clay.	1) Test area with open-mandrel sand-drains, 0.36 m diam. at 2.14 m centres. 2) Test area with sand-wicks, 0.06 m diam. at 2.14 m centres. 3) Drainless reference area.	Gains in strength were largest with sand-drains, lower with sand-wicks, negligible in reference area.
Cran, France. (Paute, 1973a).	Access ramp to structure about 3 m high, over 17 m soft clays. Observed for 3.5 years.	Driven sand drains, 0.4 m diam. and 13 m deep at 4 m centres.	Good results. Settlements slightly less than calculated using method of finite differences. Decreased moisture content and increased cohesion noteworthy in upper 5 m.
Palavas, France. (Bourges et al, 1973).	Test embankment 7 m high, on 22 m soft clay (s_u=10 to 30 kPa). Observed for three years.	1) Sand drains (jetted open-mandrel), 0.3 m diam. at 4 m centres. 2) Drainless reference area.	Good results. Settlements much more rapid in drain area (2.8 m against 1.9 m). Appreciable uniform cohesion increase throughout layer.
Narbonne, France. (Mieussens & Ducasse, 1973).	Test embankment 5.5 m high, on 15 m various soft clays interstratified with sand. Observed for 1.5 years.	1) Test area with cardboard drains at 3 m centres. 2) Test area with sand drains (jetted open-mandrel), 0.3 m diam. at 4 m centres. 3) Drainless reference area.	Unsatisfactory results. Pore water pressures dissipated slightly faster in drain areas, but settlements much the same in three areas.
Bordeaux, France. (Queyroi et al, 1977).	Embankment 6 m high, on 12 m soft organic clay. Observed for two years.	Auger-bored sand drains, 0.3 m diam. at 2 m centres.	Good results. Measured settlements agreed with those estimated on basis of c_r measured in-situ.
Skå-Edeby, Sweden. (Hansbo & Tosensson, 1977).	Test embankments 1.5 m high, on 10 m soft clays. Observed for three years.	1) Geodrains at 0.9 m centres. 2) Sand drain reference area. 3) Drainless reference area.	Results for Geodrains equivalent to results from sand drains 0.18 m diam. at 0.9 m centres.

Two methods have been reported recently for the in-situ measurement of c_r. Both of these, however, suffer from the fact that c_r is evaluated from permeameter tests and not from rates of compression that occur vertically under stress increases. Torstensson (1975) deduced c_r from the rate of dissipation of excess pore water pressure from a piezometer, c_r being deduced from nomograms. Mieussens & Ducasse (1977) employed a self-boring permeameter (Fig. 9.24a) for a similar purpose, a negative excess pore pressure being maintained as a constant head to cause the consolidation of a large volume of soil. It can be shown that c_r is given by:

$$Q = 2\pi b \cdot \frac{k_r}{\gamma_w}\left[1 + \frac{2}{\sqrt{\pi c_r \frac{t}{a^2}}}\right] \tag{9.17}$$

The comparative results of field and laboratory measurements of c_r shown in Fig. 9.24b (Mieussens & Ducasse, 1977) indicate that in-situ values are higher than laboratory values. This is almost certainly because laboratory specimens cannot adequately model a large stratified mass of soil.

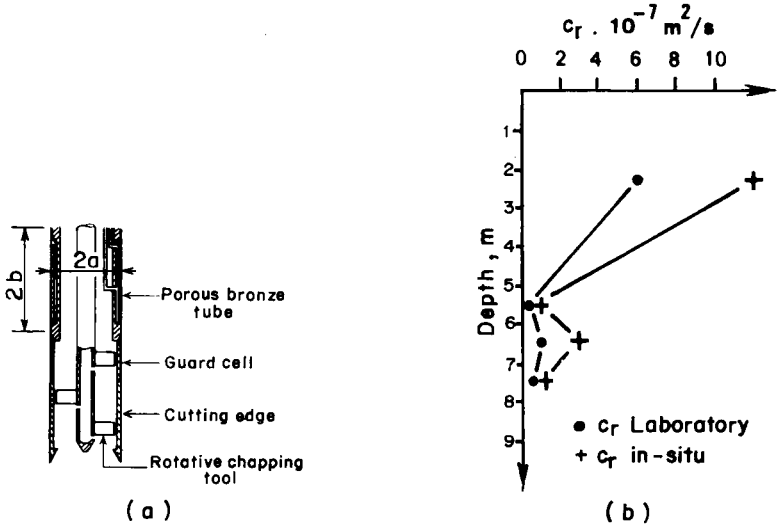


Fig. 9.24. Measurements of coefficient of radial consolidation by Mieussens & Ducasse (1977): (a) self-boring permeameter, (b) measured values of c_r.

9.3.7 Case histories of vertical drains

A number of case histories involving the use of vertical drains in soft soils were described by Johnson (1970b) and Bjerrum (1972). Details of recently published cases of vertical drain applications are compiled in Table 9.4, from which the conclusions can be drawn:

(1) Vertical drains are generally very effective except where they are installed in organic soils, in highly stratified soils, or where serious stability problems exist.

(2) The installation of sand drains by the open-mandrel method results in more efficient drains than does the closed-mandrel method.

(3) Plastic drains are increasingly being used.

(4) Predictions of rates of consolidation where drains are installed cannot be made reliably, because of the difficulties of determining a representative value of c_r and of accounting for the effects (disturbance and smear) of the drain installation.

The performance of the Bosse-Galin embankment on the Boulevard Nord in Bordeaux has been reported in some detail (Queyroi et al, 1977). The 6 m high railway embankment was built on a 12 m thick deposit of soft organic clay, with properties: $w_n=70\%$, $w_L=90\%$, $I_p=50\%$, s_u(vane)$=15$ kPa. The values of c_v determined in the laboratory ranged from 1×10^{-7} to 6×10^{-7} m^2/sec., but about twice these values were measured in-situ with a self-boring permeameter. Because of the extremely low shear strength of the clay, four-stage embankment construction was employed after 300 mm diameter auger-bored sand drains were installed at 3 m centres. The high efficiency of the drains is demonstrated in Fig. 9.25, as is the excellent agreement obtained between the actual rates of settlement and those predicted on the basis of the c_v values measured in-situ.

9.4 DYNAMIC COMPACTION

9.4.1 General principles

The process known as *dynamic compaction,* sometimes referred to as *dynamic consolidation,* consists in the densification of soil by the application of very heavy blows to the ground surface by dropping a heavy weight (rammer). The technique, which has been described by Menard (1972, 1974), was at first applied only to granular soils, but it has been used recently on silts, peats and even soft clays. Its purpose is no different from that of the two techniques described in Sections 9.2 & 9.3, i.e. to increase the shear strength of the soil and to reduce the settlements that will occur under the structure to be built at the site.

The rammer is generally lifted and dropped by a crane (dragline), several blows being applied at one spot; a regular pattern of indentations is achieved, as shown in the photograph in Fig. 9.26. Prior to compaction, a layer of granular material 1 to 2 m thick is spread over the ground surface. On completion of the initial compaction pattern, the surface of the fill is levelled, and a second stage of compaction is applied in the same manner after an interval

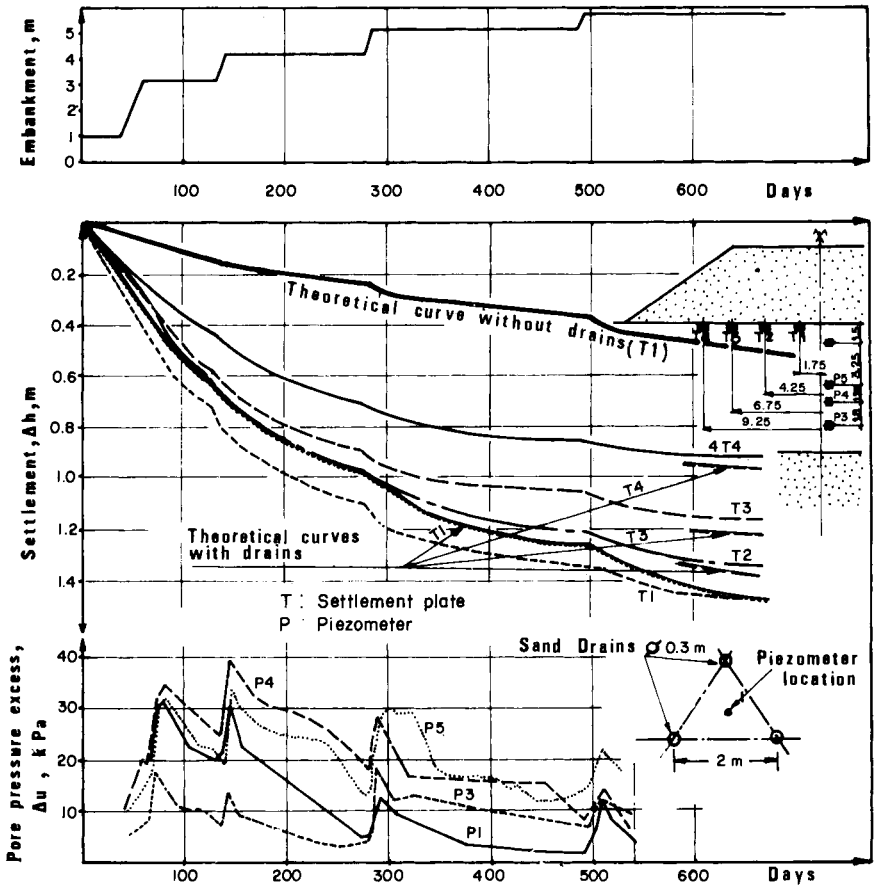


Fig. 9.25. Comparison of measured and predicted performance of sand drains under the embankment at Bosse-Galin (Queyroi et al, 1977).

of time sufficient to permit dissipation of the excess pore pressures generated by the first stage. Further stages are carried out if necessary.

Rammers generally vary in weight from 120 kN to about 200 kN, and heights of fall of up to 20 m are common. Weights of 400 kN falling through 40 m have been used for higher performance, and an uncommonly large crane in Nice, France, drops a 2000 kN weight from a height of 23 m.

9.4.2 Design for dynamic compaction

The 'design' associated with dynamic compaction is largely empirical and is based upon previous experience with the process (Scott & Pearce, 1975). With respect to soft clays, Menard & Broise (1975) suggested that the technique is effective for the following reasons:

Fig. 9.26. Dynamic compaction.

(1) Many soft clays are not fully saturated, and the small percentage of gas in the voids is dissolved in the pore water under the hammer impact, thus reducing the void volume.

(2) Soft clays often liquify under impact.

(3) The permeability is increased during compaction because of liquifaction, fissuring and shearing, and rapid dissipation of the compaction pore pressures occurs.

(4) Thixotropic strength gain follows the shear strength decrease caused by compaction.

The compression of a soft clay during dynamic compaction and its subsequent consolidation characteristics under static loads have been demonstrated in the laboratory by Magnan & Dang (1977). A shock during an oedometer test can cause an immediate volume decrease, and the clay then becomes less compressible at stresses above that at which the shock took place (Fig. 9.27). Experimentation has revealed, however, that the extent to which thixotropy exists varies greatly and is dependent upon the clay mineralogy; it seems that thixotropy is completely absent where montmorillonite or sodium ions are present (Biarez et al, 1977).

More experience is needed on the application of dynamic compaction to soft clays. At the present time, the suitability of the method in a given

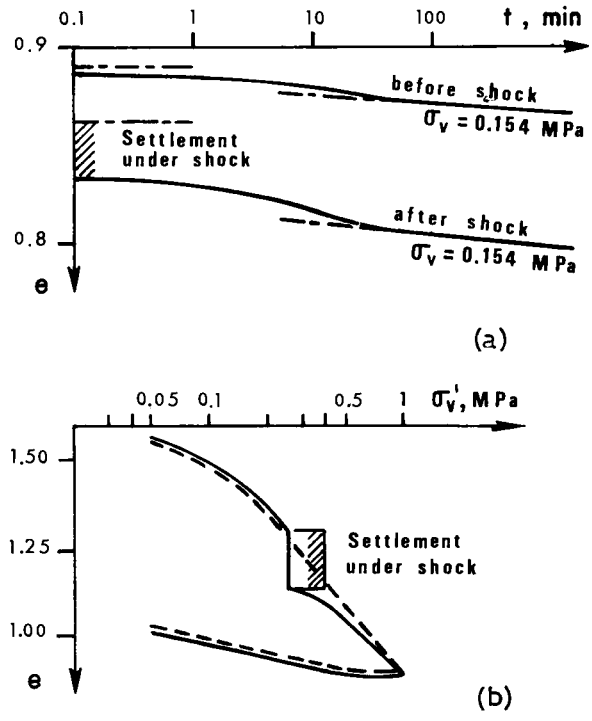


Fig. 9.27. Effect of shock on clay consolidation characteristics (Magnan & Dang, 1977).

instance, and a suitable weight and height of fall for the rammer, can only be judged on the basis of a field trial on a sufficiently large test area.

9.4.3 Case histories of dynamic compaction

Because the dyanmic compaction technique has been applied to soft clay deposits only relatively recently, only a few published accounts exist of its use in this context. Table 9.5 lists six relevant case histories, three of which specifically deal with soft clays, while the others record the application of dynamic compaction to clayey silts of low shear strength. The following comments on the six cases are pertinent:

(1) At Embourg, a test area divided into four sections was used, different energy levels being used on each section. The settlements increased as the energy increased, and the engineering properties of the soil were appreciably improved by the dynamic compaction.

(2) At the site of the Clichy Bridge, 8 m of fill were placed in the River Seine over 3 m of silt. Dynamic compaction was used mainly to com-

pact the fill, but some slight improvement in the engineering properties of the underlying silt was measured.

(3) The site at Ambes was used as a test area for dynamic compaction. Measurements immediately after compaction showed substantial improvements in the engineering properties of the soft clays. After an interval of one year, further improvements were recorded. Observations of pore pressures showed the mid-layer dissipation to have been 25 kPa within 30 days after compaction.

(4) A series of test areas was used at Ogawara to investigate a range of dynamic compaction techniques, but only two of the areas are represented in Table 9.5. The first of the two in the table was later used for the construction process, since the undrained shear strength of the soil was doubled during the test by this technique, and the higher energy application of the second technique was thought to be unnecessary.

(5) In Lisbon, the dynamic compaction was carried out along the 27 km of a motorway site. The high energy employed brought about appreciable improvements in the engineering properties of the silty soil.

(6) The engineering properties of the silty soil at Ilha dos Amores Santos were very poor before the dynamic compaction was carried out, the undrained shear strength being less than 10 kPa at the ground surface. The results of the dynamic compaction were compared with the strength gains experienced by the soil beneath a coal stockpile adjacent to the site, and it was found that, although the compaction resulted in slightly lower strength gains, these occurred much more rapidly than did those beneath the stockpile.

9.5 ELECTRO-OSMOSIS

9.5.1 General principles

More than 150 year ago, it was established by experimentation that an electric current flowing through a moist soil caused the pore water to flow from the positive pole (anode) to the negative pole (cathode); at the same time, electrolysis occurred, with the anions and cations of the dissolved salts moving towards the anode and cathode respectively. This process is known as *electro-osmosis*.

The part of the electro-mosis process by which the pore water moves under the influence of an electric potential is known as *electro-drainage,* and its effect on fine-grained soil is to decrease the water content and, hence, to bring about consolidation and increased undrained shear strength. This technique is most commonly employed where the desired soil improvement is 'local' in nature, e.g. to improve the stability of cuts and trenches, and to increase the bearing

Table 9.5. Case histories of dynamic compaction

Location and reference	Foundation soil							Surface fill thickness (m)	Dynamic compaction details				Results of the dynamic compaction					
	Soil type	Depth (m)	w_p (%)	I_p (%)	w_n (%)	k (m/sec)	c_v (m²/sec)		Weight (kN)	Drop (m)	Specific power (kN/m²)	Compaction time (days)	Settlement (mm)		p_l or s_u (kPa)		E_m (kPa)	
													total	found.	before	after	before	after
Embourg, Belgium. (Van Wanbeke & De Beer, 1973)	silt	3	22	13	30	1.3×10^{-9}		2	80 80 80 80	5–15 10 10 10 5 10 5 10	800 + 410 880 + 740 860 + 790 550 + 410	15 15 15 15	210, 150 } 360 240, 130 } 370 330, 100 } 430 220, 70 } 290	150 average for 1st stage, 60 average for 2nd stage	110	420 for 1st stage, 480 for 2nd stage	700	5500 for 1st stage, 4800 for 2nd stage
Pont de Clichy, France. (Gigan, 1977)	organic silt (beneath 7 m fill)	3	70	20	63	1.3×10^{-5}	5×10^{-5}	1	120	22	700 + 550 + 400 + 200	10 7 4			520	730 670 730 650 (after 10 months)	3700	5000 4700 5000 4400 (after 10 months)
Ambes, France. (Ménard, 1976)	soft clays	10 + 8	21 22	25 39	48 64		3×10^{-7} to $2.5 \times$	0.5	120 120 120 170	16 16 16 20	384 + 307 + 235 + 204		180, 150, 120, 190 } 730		120 (−2m) to 450	210 (−2m) to 520		

Ogawara, Japan. (Ménard, 1976)	soft clays (volcanic origin)	7	80	40				0.5	90 and 120 90 and 120	27 (max) 27 (max)	500 + 500 + 200 1000 +1000 + 200	30 2 30 2	320, 20 } 340 } 20		s_u 50 60	s_u 100 970		
Lisbon, Portugal. (Ménard, 1976)	silt	27	35	35	66			1 + 1	160	22	1750 +1750	60	460, 240 } 700		50 (−4m) 100 (−7m) 300 (−15m)	420 (−4m) 300 (−7m) 300 (−15m) (after 8 months)	500 (−4m)	4500 (−4m) (after 8 months)
Ilha dos Amores Santos, Brasil. (Project document)	silt	20	40	95		10^{-9}		2	200	20	350 + 700	25	800, 560 } 1360		s_u 3 to 15 p_l 3 to 4	s_u 12 to 25 p_l 10 to 12	?	?

Notation. s_u = undrained shear strength: p_l & E_m = pressuremeter parameters.

capacity of piles (used as anode in the process). A local *increase* in water content is also sometimes intentionally brought about to decrease the resistance to the extraction of temporary sheet piles; in such cases, the piles are used as cathode in the electro-osmosis.

Electro-osmosis may also be employed as a means of carrying out chemical injection in a soil which is of too low a permeability for injection by conventional means to be feasible. This aspect of electro-osmosis, by which the ions migrate under the electric potential, is termed *electro-injection* or *electrochemical treatment*. The anode either dissolves or is fed with the appropriate ion solution to bring about a chemical change in the soil (and a strength increase) without alteration of the soil volume or structure. This provides a means for the in-situ strengthening of the soil beneath existing structures built on the ground surface which must not be disturbed.

Electro-osmosis gives rise to a range of more or less complex physicochemical effects. These have been dealt with in a large number of publications, but the survey by Mitchell (1970) adequately discusses all of these. Much less extensive is the literature relating to the measured effects of electro-osmosis on soil properties and to its application to engineering projects. Emphasis herein will be placed, therefore, on these applied aspects of the process.

9.5.2 Consolidation by electro-drainage

The electric potential between two electrodes in a body of soil causes the ions and the free water within the outer layer of the particle double-layer to be set in motion in the soil pores. The electro-drainage flow rate, v_e, of the pore water can be estimated theoretically for different sets of assumptions: the reader is referred to the papers by Esrig & Majtenyi (1965) and Gray & Mitchell (1967) for a treatment of the application of double-layer theory to electro-drainage.

The approximate solution for electro-drainage suggested by Casagrande (1937) is still in common engineering use. This gives the flow rate as:

$$v_e = k_e\, i_e = k_e \frac{\partial V}{\partial x} \tag{9.18}$$

where i_e is the potential gradient, and k_e is the coefficient of electro-osmotic permeability, given by:

$$k_e = \frac{D_e r_v^2 \zeta}{8\, \eta_f} \tag{9.19}$$

where D_e is the dielectric constant of the pore fluid, r_v is the radius of the voids, η_f is the fluid viscosity, and ζ is the zeta potential. Casagrande measured values of k_e in the laboratory for various soil types under conditions of one-, two- and three-dimensional flow, and he established a mean value of approximately 5×10^{-3} mm^2/sec/volt.

The application of surface loading to a soil of hydraulic permeability k results in the development of excess pore pressures which dissipate as flow takes place under Dracy's law at a velocity given by $v=ki$, and consolidation occurs. Where electro-osmosis is employed in conjunction with surface loading, the rate of flow for a one-dimensional situation is given by:

$$v = k_e i_e + ki = -k_e \frac{\partial V}{\partial x} - \frac{k}{\gamma_w} \frac{\partial u}{\partial x} \tag{9.20}$$

which leads to:

$$k_e \frac{\partial^2 V}{\partial x^2} + \frac{k}{\gamma_w} \frac{\partial^2 u}{\partial x^2} = m_v \frac{\partial u}{\partial t} \tag{9.21}$$

This can be rewritten as:

$$c_v \frac{\partial^2 \xi}{\partial t^2} = \frac{\partial \xi}{\partial t} \tag{9.22}$$

where:

$$c_v = \frac{k}{m_v \gamma_w} = \text{coefficient of consolidation} \tag{9.23}$$

$$\xi = \frac{k_e}{k_H} \gamma_w V + u \tag{9.24}$$

Esrig (1968) studied the theoretical pore pressure distribution in soils undergoing electro-osmotic consolidation and showed that the pore pressure varied linearly between the electrodes after a long period of time, the magnitude of the pressure difference between the two being:

$$u(x) = \frac{k_e}{k} \gamma_w V(x) \tag{9.25}$$

where $V(x)$ is the potential difference applied across the electrodes. This result indicates the importance of the ratio k_e/k to the effectiveness of the electro-osmotic treatment of a particular soil. It should be noted, however, that the *rate* of consolidation is governed by c_v (i.e. by m_v and k) and is independent of k_e.

The theoretical studies of electro-osmosis by Esrig (1968), Lewis & Humpheson (1973) and Wan & Mitchell (1975, 1976) provided explanation for the earlier experimental observations of Mise (1961).

One of the first reports of the practical use of electro-osmosis was that by Bjerrum et al (1967) which described the improvement of a 12 m thick sensitive clay at Ås in Norway. The water contents in the vicinity of the anode decreased significantly, resulting in appreciable undrained strength increases in the soft

clay (Fig. 9.28), while almost no changes occurred near the cathode. The effects of the electro-osmosis were also felt in terms of increased plasticity of the clay towards the anode. The laboratory test results obtained later by Caron (1971) for a remoulded soft clay (Fig. 9.29) were of the same form as those of Bjerrum et al.

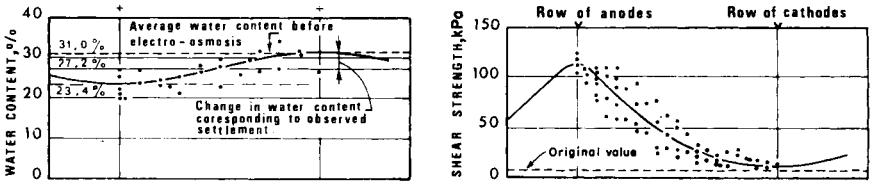


Fig. 9.28. Electro-drainage on sensitive clay at Ås (Bjerrum et al, 1967).

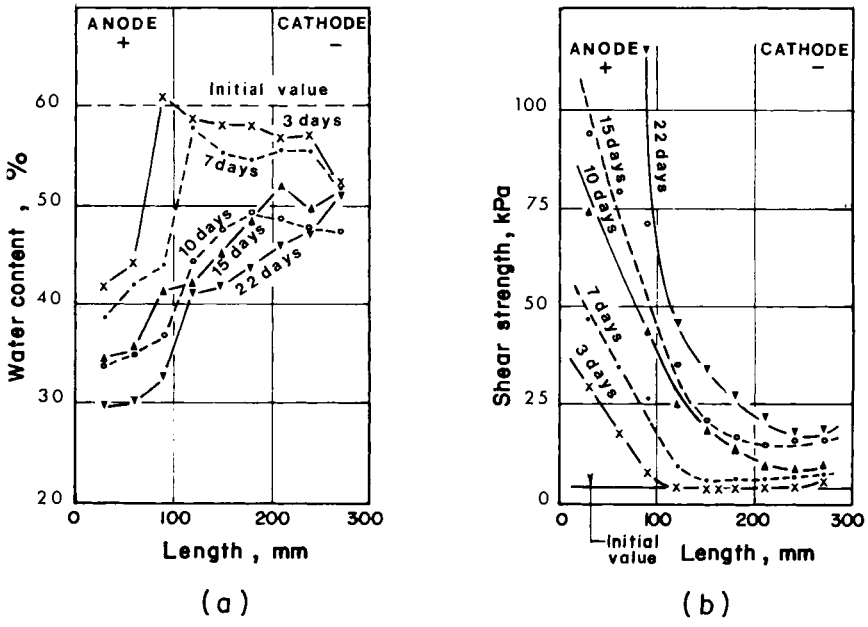


Fig. 9.29. Results of electro-drainage laboratory tests on soft clay (Caron, 1971).

9.5.3 Electro-injection

The chemical changes which are brought about by electro-chemical treatment are often so complex that it has been difficult to employ any 'design' method other than to measure the improvement in soil properties brought about by the treatment in field trials or in laboratory experiments. Such tests are aimed at quantifying the improvements in soil properties by different chemicals and electrical parameters.

The tests conducted by Caron (1971) to determine the effects of electro-chemical treatment on the engineering properties of a soft clay from near Bordeaux, France, is typical of the laboratory studies carried out. Caron used a 300 mm long box of the remoulded soil (water content of 60%) across which a potential difference of 12 volts was applied through flat graphite electrodes. Solutions of 35% sodium silicate and 10% ammonium chloride were fed in at the anode in two separate tests. The shear strengths measured by vane tests are illustrated in Fig. 9.30 and these are compared with the strengths attained after 15 days of electro-drainage alone in a separate experiment. Although the electro-injection with sodium silicate resulted in higher strengths than electro-drainage alone, it should be remembered that the latter process was one third of the cost of the electro-injection.

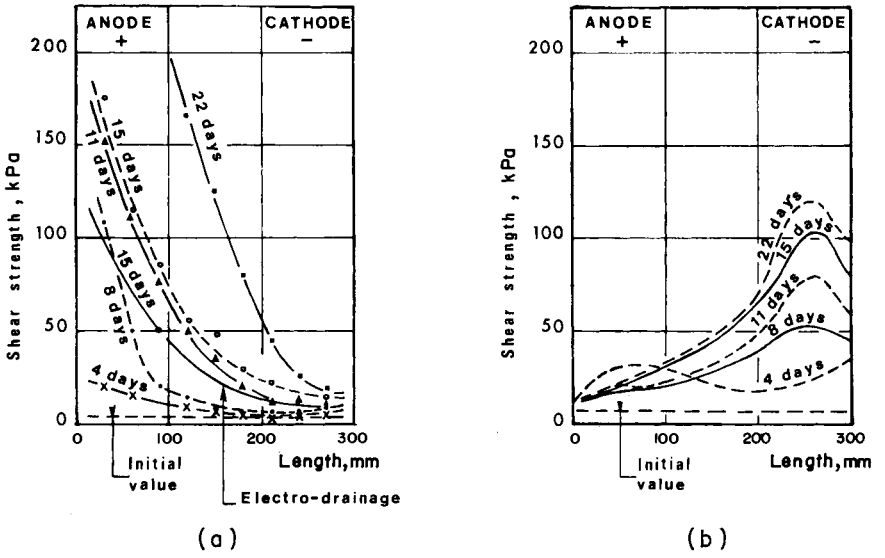


Fig. 9.30. Results of electro-injection laboratory tests on soft clay (Caron, 1971): (a) with 35% sodium silicate, (b) with 10% ammonium chloride.

9.5.4 Case histories of electro-osmosis

Assembled in Table 9.6 are brief details of the nine case histories of electro-osmosis treatment of soft clays that the Author was able to find in the technical literature. The small number is a clear indication of the infrequent use of the technique; this is particularly true of electro-injection, there being only three recorded cases of its engineering application. The table includes no Russian cases, although it is known that electro-osmosis has been relatively widely used in the USSR for the treatment of silts.

The costs involved in electro-osmosis are not easy to assess from the case histories. The electricity consumed in electro-drainage increases with soil

Table 9.6. Case histories of electro-osmosis

Location and reference	Object of treatment	Soil treated	Procedure	Results and comments
Ås, Norway. (Bjerrum et al, 1967)	To increase strength prior to earthworks.	Sensitive clay, about about 12 m thick.	*Electro-drainage:* 65 V/m at 250A for 4 months.	Satisfactory treatment of 2700 m^3 soil; w dropped from 31 to 17%; s_u increased from 9 to 38 kPa.
Warren, USA. (Fetzer, 1967)	Treatment of West-Branch Dam foundation (under construction).	Fairly soft silty clay, 18 m thick.	*Electro-drainage:* 8–25 V/m at 40A for 10 to 12 months.	Satisfactory treatment of 580,000 m^3 soil; excess pore pressure greatly reduced.
Singapore. (Chappell & Burton, 1975)	Treatment of embankment and its foundation on creeping soil.	Fill and soft clay, 5 m thick.	*Electro-drainage* with local hardening next to the anode: 14 V/m at 25–30A for at least 9 days.	Satisfactory; w of soft clay decreased from 72 to 35%; s_u increased from 50 to 144 kPa.
Bordeaux, France. (Caron, 1968)	Treatment prior to excavation.	Soft clay, 9 m thick.	*Electro-drainage:* 16–25 V/m at 20A for 3 months.	Satisfactory treatment of 1000 m^3 soil.
Tunnis, Tunisia. (Caron, 1968)	Foundation soil treatment prior to construction of reservoirs.	Very soft clay, 15 m thick.	*Electro-injection:* ammonium chloride.	Satisfactory; s_u increased from 6 to 15 kPa.
Seattle, USA. (Dearstyne & Newman, 1963)	Treatment of foundation soil under existing runway.	Softed clay (through pumping), 1.5 m deep.	*Electro-injection:* H.226, with 6 V/m at 4A for 15 days.	Satisfactory.
Antwerp, Belgium. (De Beer & Wallays, 1965)	Treatment prior to excavation.	Very soft clay (w_n=120–220%), 7 m thick.	*Electro-drainage:* 10 V/m at 45A for one month.	Satisfactory.
Angers, France. (Caron, 1971; Peignaud, 1977)	To increase soil strength under bridge abutment.	Soft to stiff clay, 10 m thick.	*Electro-injection:* sodium silicate, with 28 V/m at 30A for maximum 3 months.	Unsatisfactory—major implementation problems.
Dighi, India. (Sridharan, 1972)	To harden soil around a pile.	Soft clay around a test pile 150 mm diameter.	*Electro-drainage:* 90 V/m at 8A for 72 hours.	Satisfactory.

conductivity and exceeds that required for electro-injection, but no chemical costs are involved. The electricity consumption for the cases in Table 9.6 increased from as little as 0.5 kW. hr/m^3 of soil in Singapore to 17 kW. hr in Norway; if the treatment at Angers had been fully completed, this would have required an incredible 200 kW. hr/m^3. The worldwide power price escalation will probably prevent any further development of electro-osmosis and will limit its use even further.

The circumstances in which electro-osmosis has been used are clearly where virtually all other methods were impossible to apply. The results achieved by the method are generally thought to have been satisfactory.

There are recorded instances of where field results obtained with electro-osmosis did not match those obtained on pilot tests in the laboratory, and these emphasise the impossibility of accounting completely for field conditions in laboratory experiments. A good example of the problems encountered in the field can be found in the published details of the electro-injection used to improve the soft clay beneath the abutment of the Basse Chaîne bridge in Angers, France (Caron, 1971; Peignaud, 1977). The abutment was built on shallow footings founded mainly on soft clay with shear strengths as low as 20 kPa (Fig. 9.31), and this resulted in horizontal movements of the bridge of up to 90 mm in the eight years after its completion. After it was decided that soil improvement was essential, a series of laboratory tests was carried out on the remoulded clay to test the likely effectiveness of electro-injection. Graphite electrodes were used to apply a 35% solution of sodium silicate, and the results (Fig. 9.32) indicated that the treatment was highly successful. When the same technique was used in the field, however, the results were disappointing. There were large silicate losses by gravity into the sand layer, and silica which was deposited on the stainless steel cables used as the anode

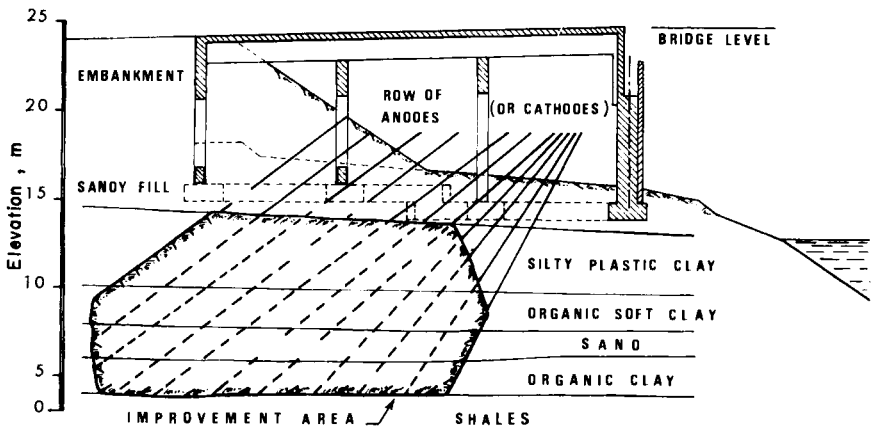


Fig. 9.31. Electro-injection for the Basse Chaîne bridge (Caron, 1971; Peignaud, 1977).

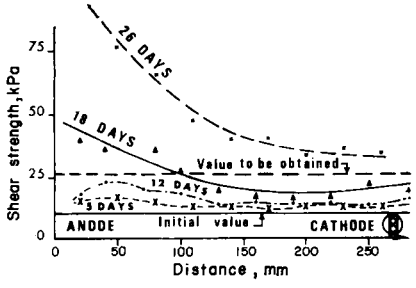


Fig. 9.32. Results of electro-injection laboratory tests on soft clay at Basse Chaine bridge (Caron, 1971).

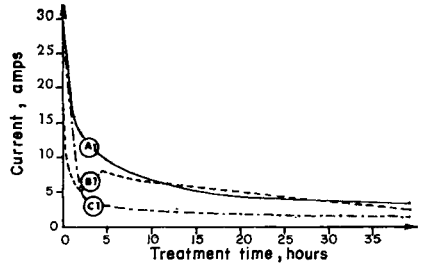


Fig. 9.33. Current reductions during electro-injection at Basse Chaîne bridge (Caron, 1971; Peignaud, 1977).

greatly reduced the current between the electrodes (Fig. 9.33). As a result of these difficulties, the electro-injection was abandoned.

9.6 STONE COLUMNS

9.6.1 General principles

A fairly recent technique, which originated in the 1960s, for the improvement of soft clay foundations is the installation of stone columns. This method consists in forming vertical holes in the ground which are filled with crushed rock to form columns or 'piles' confined by the soil. These columns fulfil the two function (1) that they provide strength reinforcement to the soil, and (2) that they act as vertical drains to enable consolidation to occur quickly under surface loading. Because of the relatively high modulus of the columns, a large proportion of the vertical load applied to the ground surface is transferred to the columns. The beneficial effects of stone columns have been clearly demonstrated by tests of the kind carried out by Engelhardt et al (1974) to compare load-settlement characteristics of a deposit of soft clay with and without stone column reinforcement (Fig. 9.34.).

Construction of stone columns is carried out in two stages. A borehole is drilled to the required depth using a vibratory drive element *(vibroflot* or *torpedo)* which consists of a pointed cylinder, 2 to 5 m long and 0.3 to 0.4 m diameter (Fig. 9.35a), which is vibrated in a horizontal plane by means of eccentric weights housed in the torpedo and driven by a fluid drive or an electric motor. This penetrates under its own weight, often in conjunction with water jetting (Fig. 9.35b). The resulting hole size depends on the consistency of the soil, but this generally ends up at between about 0.7 m and 0.9 m. The torpedo is removed and the hole is filled with crushed rock, the torpedo being used to compact the column in layers. In some working processes, cement grout is used to fill the voids to form a pile in the ground; in others,

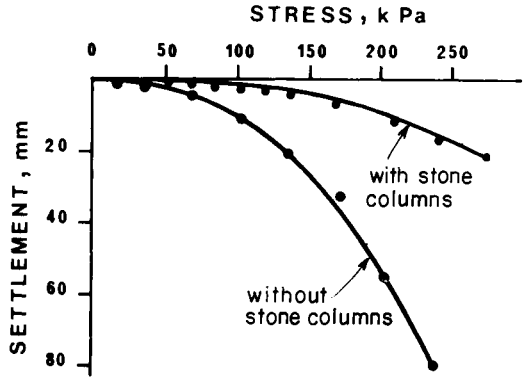


Fig. 9.34. Effectiveness of stone columns (Engelhardt et al, 1974).

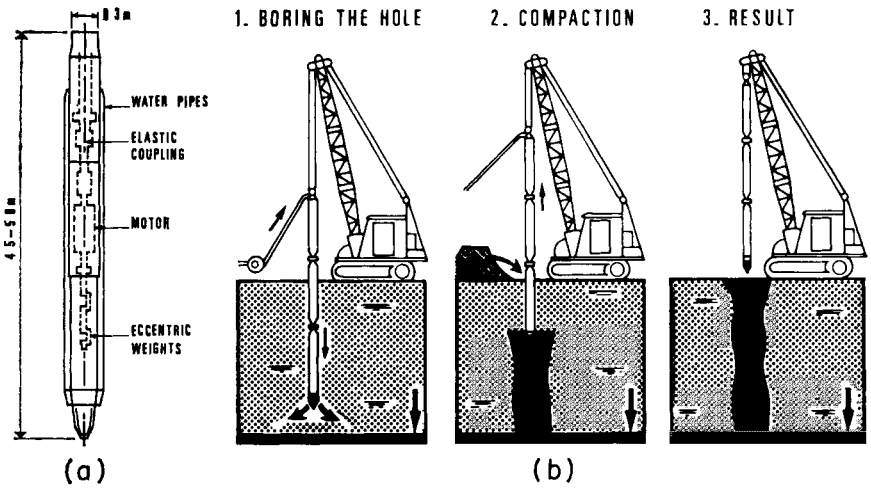


Fig. 9.35. Installation equipment and procedure for stone column construction: (a) torpedo for hole formation, (b) column construction.

compacted sand piles of the type described by Abelev (1973) and Tanimoto (1973) are installed.

The load that can be carried by a single stone column depends on the material and diameter of the column, and on the stiffness of the soil in which it is installed. Allowable loads usually range from 100 to 300 kN.

9.6.2 Design of stone columns

A stone column is subjected to stress conditions much like those imposed in the standard triaxial test (Fig. 9.36). A vertical effective stress, σ'_v, is applied by ground surface loading, and a radial effective stress, σ'_r, results from the

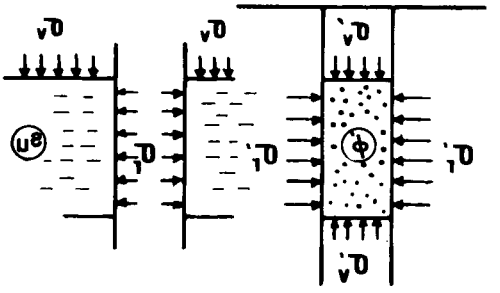


Fig. 9.36. Stresses acting on a stone column.

horizontal reaction of the ground. The factors which govern the soil-column behaviour, therefore, are (Hughes et al, 1975):

(1) undrained shear strength of the soil,
(2) in-situ lateral stress of the soil,
(3) radial stress-strain characteristics of the soil,
(4) initial column dimensions, and
(5) stress-strain characteristics and angle of internal friction, φ', of the column material.

The consolidation of the soil which occurs as the surface load is applied is neglected.

The maximum vertical effective stress that the column can sustain, σ'_{vf}, is reached when the soil fails radially because the maximum radial stress it can develop, σ'_{rf}, is reached. The failure condition which relates the vertical and radial stresses is:

$$\sigma'_{vf} = \frac{1+\sin\varphi'}{1-\sin\varphi'}\,\sigma'_{rf} \tag{9.26}$$

The value of σ'_{rf} can be expressed in terms of the initial radial stress, σ'_{ro}, as:

$$\sigma'_{rf} - \sigma'_{ro} = Ks_u \tag{9.27}$$

which gives:

$$\sigma'_{vf} = \frac{1+\sin\varphi'}{1-\sin\varphi'}\,(\sigma_{ro}-u+Ks_u) \tag{9.28}$$

where s_u is the undrained shear strength of the clay, σ_{ro} is the initial total radial stress, u is the pore pressure, and φ' is the angle of internal friction of the material of the column.

Measurements carried out by Hughes & Withers (1974) in soft clay using a self-boring pressuremeter *(Camkometer)* yielded values of K equal to about 4.0, whereas Ménard (1965) had earlier used a conventional pressurementer

to obtain values of about 5.5. The full-scale loading tests on stone columns performed by Hughes et al (1975) at Canvey Island verified the reliability of Eq. 9.28 as a design criterion and supported a value of K of about 4.0.

9.6.3 Case histories of stone columns

Case histories of the use of stone columns in soft soils are summarized in Table 9.7. It can be seen that stone columns are generally only employed in thick deposits of soil of very low shear strength, and that the columns themselves often do not penetrate the layer of soft soil. Column diameters vary between only 0.7 m and 0.9 m, and the density of installation is from one per 1.1 m^2 to one per 5.7 m^2.

The results achieved using columns demonstrate clearly the reduced settlements and increased stability of embankments and slopes. Table 9.7 also emphasizes the increased undrained shear strengths of the soft clays, a factor of which insufficient account is taken at the design stage. The East Brent case (McKenna et al, 1975) provides evidence that a good installation technique is important to the efficient functioning of the columns.

The case history from Rouen, France, provides detailed information of the effects of the stone columns installed under the embankment at the site (Vautrain, 1977). Observations were made of the pore pressures in the soft clay and the vertical stress distribution at the ground surface during and after the construction of the embankment (Fig. 9.37). Whereas the embankment imposed an average vertical stress of 180 kPa, it was found that the columns

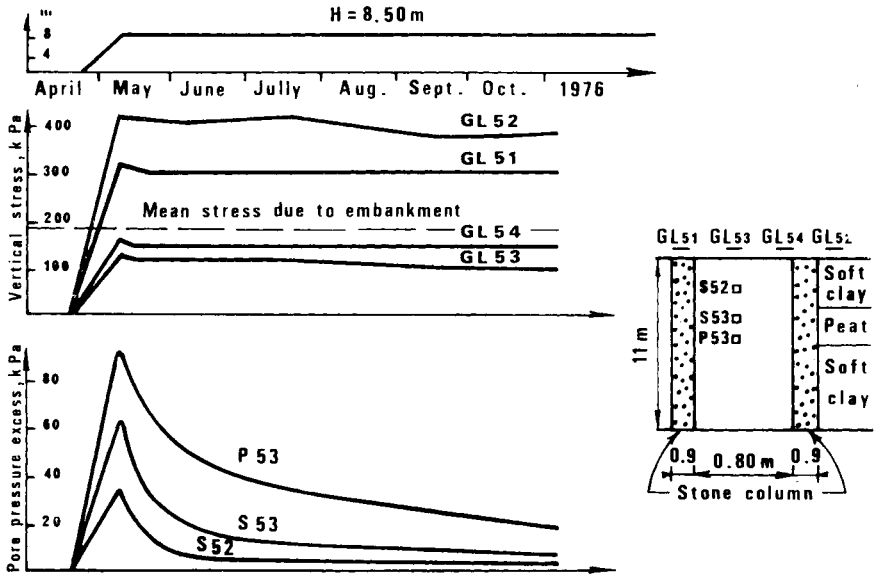


Fig. 9.37. Performance of stone columns in soft clay at Rouen (Vautrain, 1977).

Table 9.7. Case histories of stone column applications

Building site	Foundation soil	Column data	Results
Canvey Island, England. (Hughes et al, 1975)	7 m soft clay under 2 m overconsolidated clay (s_u=15 to 30 kPa).	Loading test applied to column 10 m long, 0.66 m diam. Angle of internal friction (assumed)=38°.	Ultimate load=200 kN; maximum settlement =0.17 m (0.01 m under 100 kN). Very good agreement between measurements and design data of Hughes et al on basis of column's actual diameter (0.73 m).
Freyming A34, France. (Bachy, 1976)	8 m peaty clay. (Menard pressuremeter: p_l=270 kPa, E_m=2,000 kPa).	Columns under foundations and embankment. 1 column per m².	Settlement under loads less than 100 mm. After improvement process, p_l=20,000 kPa, E_m=54,000 kPa.
Argenteuil RB 311, France. (Jardin, 1974)	3.5 m soft organic clay (s_u=10 to 33 kPa).	Columns under embankment slope, 5.5 m long × 0.8 m diam. 1 column per 3.5 m².	No settlement or stability problems encountered; after improvement process, s_u=40 kPa.
Cagnes A8, France. (Cassan, 1974)	36 m soft silty clay (s_u=10 to 70 kPa).	Columns under viaduct abutment and approach embankment, 18 m long × 0.9 m diam. 1 column per 2.8 m².	Average soil strength increase of 70%. Trouble-free behaviour of abutment.
Konstanz, Germany. (Engelhardt et al, 1974)	15 m soft and low-consistency silty clay.	Test area with 12 columns, 12 m deep × 0.8 m diam. 1 column per 1.1 m².	Greatly decreased settlements: under stress of 200 kPa, 55 mm prior to and 10 mm after improvement process.
East Brent, England. (MacKenna et al, 1975)	12 m silty soft clay (s_u=15 to 50 kPa) over 16 m silty sand.	Columns under embankment abutment, 11.3 m long × 0.8 m diam. 1 column per 5.7 m².	No reduction in settlements, nor in pore water pressures (might be due to columns being clogged by mud).
Rouen, France. (Vautrain, 1977)	About 10 m soft soils: clay, peat and chalk (s_u=20 to 50 kPa).	Columns under reinforced-earth embankment, 10 m long × 0.9 m diam. 1 column per 3 m².	Greatest settlement measured=0.4 m (0.9 m. without improvement treatment), greater than anticipated. Test apparently carried out to column ultimate strength.

supported stresses of 300 to 400 kPa compared with those measured at the clay surface of 120 to 160 kPa (Fig. 9.37a). The excess pore pressures reached maximum values of only 30 to 90 kPa (compared to the maximum applied stresses of 120 to 160 kPa) and they dissipated very rapidly (Fig. 9.37b). The efficiency of stone columns is well illustrated by this case.

9.7 LIME PILES AND COLUMNS

Because of the low permeability of most soft clay deposits, conventional chemical grouting is ineffective in such cases. The electro-injection technique described in Section 9.5 is one method by which the soil properties can be changed physico-chemically, and another is the use of lime stabilization. Lime stabilization of silts and clays is a well-known process of long standing which has been codified by some bodies for road engineering purposes (e.g. Ministère de l'Equipement, 1972), but the process is confined to about the top 0.3 m of the ground surface.

The fundamental action of lime on soft clays is three-fold, namely:

(1) The water content of the clay is decreased by the fast hydration reaction of the lime to form $Ca(OH)_2$.
(2) The plasticity index is decreased by the input of Ca ions and the flocculation of the clay particles.
(3) The lime reacts slowly with the clay to change the clay structure by the formation of aluminates and hydrated calcium silicates.

In the 1960s, the lime stabilization of the surface 1 m of soft clay was achieved in the USA and Germany by filling 100 mm diameter prebored holes with lime. These *lime piles* have also been used to deal with great thicknesses of clay in the USSR (Abelev, 1973). In Sweden, *lime columns* (i.e. columns of soft clay stabilized with lime) have recently been introduced (Assarson et al, 1976; Broms & Bowman, 1977). This technique consists in the in-situ mixing of about 6% quicklime (CaO) with the soft clay by means of an auger (Fig. 9.38). The resulting lime columns have about 50 times the untreated soil strength one year after treatment, one third of the strength gain being achieved in one month and one half in two months. The piles are also more permeable than the untreated soil, and they perform as vertical drains to aid consolidation.

Very few instances of the use of lime piles or columns have been described in the literature, partly because of the very recent development of this technique. Abelev (1973) has described the improvements brought about at Barnaul and Buston in the USSR. At Barnaul, a soft silty clay stratum 30 m thick ($\gamma = 15$–16 kN/m^3, $w_n = 23$–26%) was treated with quicklime piles 0.5 m diameter and 7 m long at 2 m centres. The water content of the clay was decreased by an average of 5% (Fig. 9.39); the shear strength parameters

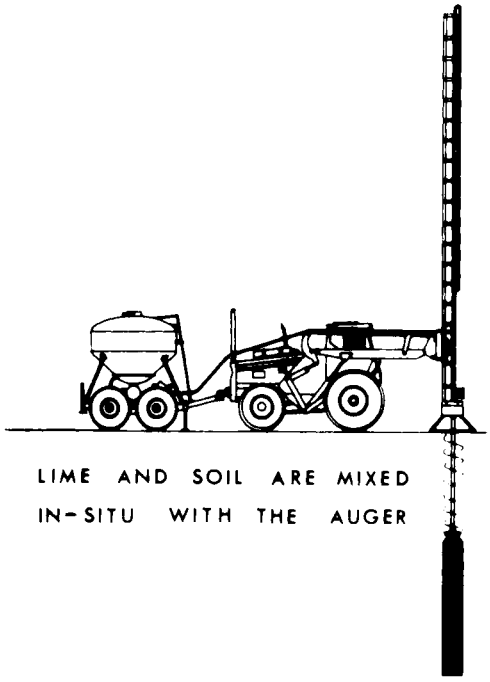


Fig. 9.38. Formation of a lime column.

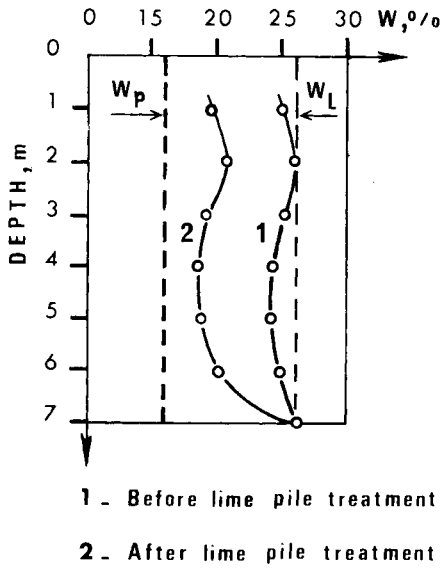


Fig. 9.39. Effect of lime piles on water content of soft clay at Barnaul (Abelev, 1973).

increased from $c' = 12$–21 kPa and $\varphi' = 13$–$14°$ to $c' = 30$–37 kPa and $\varphi' = 17$–$20°$; and the soil modulus mid-way between the piles increased from 1,000–1,600 kPa to 2,900–3,800 kPa. At Buston, loading tests were carried out before and after the treatment of a silty clay ($\gamma = 17$–18 kN/m^3, $w_n = 25$–28%) with quicklime piles 0.265 m diameter and 5 m long. The plate bearing tests performed 36 days after the treatment (Fig. 9.40) clearly demonstrate the effectiveness of the treatment.

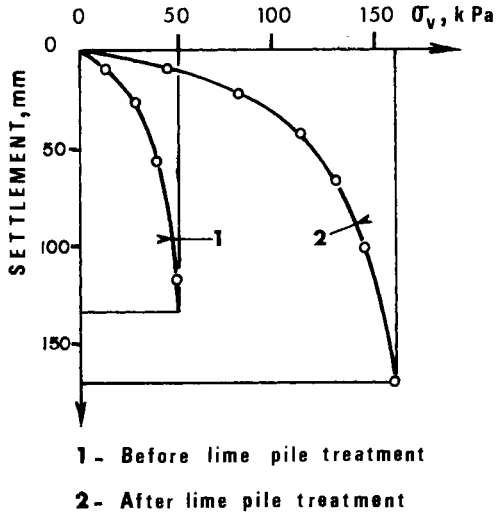


Fig. 9.40. Effect of lime piles on plate bearing tests on soft clay at Buston (Abelev, 1973).

9.8 GROUND FREEZING

The process of ground freezing is employed in some circumstances to aid the construction of certain underground facilities, such as tunnels, shafts and caissons (Fougeot & Rouault, 1969; Potevin, 1972). It is not intended to bring about permanently improved mechanical properties of a soil but merely to effect improvements during construction. Ground freezing is not in common use in any soil type (because of the high cost) and its use in soft clays is very rare. It will be dealt with, therefore, only briefly here.

When a mass of soil is frozen, its strength is increased and its permeability decreased. Very high shear strengths are achieved (Vialov, 1966; Sanger, 1968). Its mechanical properties are similar to those of ice, its all important creep characteristics under load being temperature dependent (Fig. 9.41) (Sanger, 1968; Maishman, 1975).

The freezing process in the field has traditionally been carried out by means of brine circulation through a system of pipes, but the most recently developed process employs liquid nitrogen (boiling point $-196°$C).

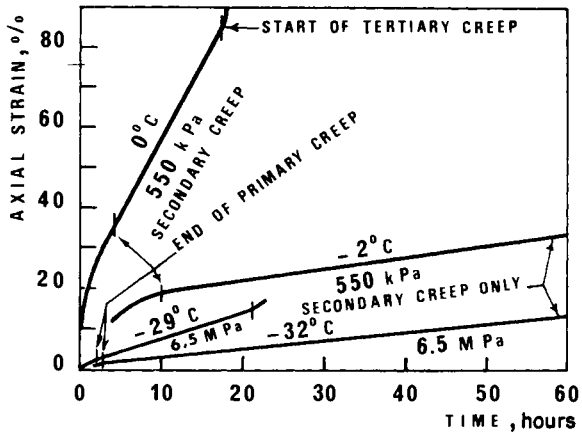


Fig. 9.41. Creep characteristics of a frozen silty clay (Sanger, 1968).

A typical example of the use of ground freezing is its application to the driving of a heading through very soft silty clay below the water-table at a lake at Le Bourget (Chapeau & Dupuy, 1976). Figure 9.42 shows the ground profile, the configuration of the heading chamber and the volume of soil frozen. The freezing equipment consisted of a bundle of 17 cooling pipes 20 m long and 0.5 m apart through which liquid nitrogen was circulated. A 0.7 m thick vault of frozen soil around the chamber location was completed in three days, 1200 litres of liquid nitrogen being used per m^3 of frozen ground to maintain the temperature at between -1 & -3°C. The construction process was completely trouble-free apart from 10 mm of creep that occurred in the eight days required to excavate the chamber.

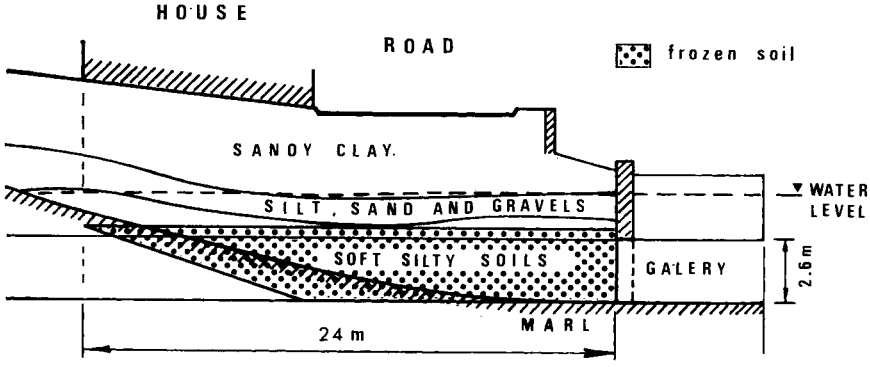


Fig. 9.42. Ground freezing for the construction of a heading at Le Bourget (Chapeau & Dupuy, 1976).

9.9 CONCLUSIONS

The methods for the improvement of the engineering properties of soft clays can be divided into three groups. The first consists of those methods which bring about consolidation directly, namely preloading and vertical drains; they tend to be the most suitable means of dealing with large masses of clay most simply and economically. The second group, which includes stone columns and lime columns, encompasses those processes which act to physically reinforce or stiffen the soil. The last group consists of specialized, expensive processes which treat relatively small volumes of soil; electro-osmosis and freezing are in this category.

Techniques available for soft soil improvement are still being developed. Existing processes for other soil types are also being adopted for use with soft clays. Experience indicates clearly that the 'design' aspect of soil improvement is not at all satisfactory, and field trials are generally necessary to ascertain the improvements to be achieved by the adoption of a particular method. There is certainly ample scope for future research and development on the subject.

ACKNOWLEDGEMENTS

The Author sincerely thanks Miss M.L. Lemesle of the Technical Information Service of the Laboratoire Central des Ponts et Chaussées for the extensive help she gave him. He also thanks his colleagues at the LCPC, Messrs F. Blondeau, J.P. Magnan and M. Peignaud. He is also grateful to Mr C. Caron of GEIC, Mr J.M. Cognon of PTS Ménard, Mr F. Contal of l'Air Liquide, and Mr G. Schultze of SIF Bachy, with all of whom he had fruitful discussions.

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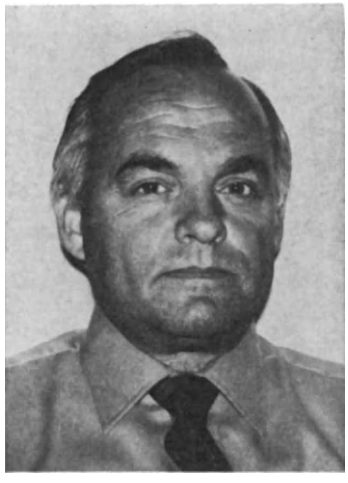
Chapter 10

FIELD INSTRUMENTATION FOR SOFT CLAY

by

ELMO DIBIAGIO and FRANK MYRVOLL



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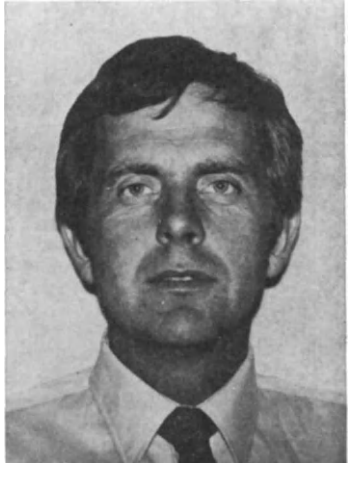
Dr DiBiagio has over 22 years experience related to full-scale geotechnical and structural instrumentation projects. This experience encompasses the design of instruments, the development of measurement techniques, and the planning and implementation of instrumentation projects. He has published some 37 technical papers.



Frank Myrvoll has spent all his working life with the Norwegian Geotechnical Institute, which he joined in 1966 after graduating with an MSc in Civil Engineering from the Technical University of Norway. After working for three years on the design and construction of embankment dams in Norway, he was then engaged for two years on foundation problems in soft clay. The last ten years have been spent on geotechnical and structural instrumentation, and he is now Senior Engineer in the Instrumentation Section of NGI.

Mr Myrvoll's instrumentation experience covers development and measurement techniques, and planning and implementation. He has been involved in instrumentation projects associated with piles, slurry trench walls, deep excavations, tunnels and embankments, as well as with structural and foundation monitoring of concrete gravity structures in the North Sea. He has published nine technical papers.

Chapter 10

FIELD INSTRUMENTATION FOR SOFT CLAY

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10.1 INTRODUCTION

The pace of scientific achievement is accelerating in the field of soil mechanics and foundation engineering. This is readily apparent from even a cursory glance through the literature or a quick look at the topics covered by the multitude of geotechnical conferences held each year throughout the world. There are a number of significant reasons for this rapid development. First of all, more workers are contributing to the general advancement of the state-of-the-art; secondly, we have acquired through the years a better understanding of the fundamentals that are important to the development of this relatively young scientific field; and thirdly, because of technological advances in other fields, we have at our disposal a larger and better selection of the tools and facilities that are needed to advance the frontiers of knowledge in our own field. One of the tools that has contributed significantly to the technological advances in soil mechanics and foundation engineering is instrumentation.

Geotechnical instrumentation can be defined in a number of different ways depending on the viewpoint taken. For some people, *instrumentation* refers to the various mechanical, hydraulic and electrical devices that are used to make physical measurements; others interpret *instrumentation* simply as the physical activity associated with the installation of measuring devices.

As commonly practiced in geotechnical engineering, however, instrumentation is more than just hardware and installation work. A more appropriate definition is, perhaps:

> *Geotechnical instrumentation is a combination of activities, consisting of part philosophy and know-how, part insight and experience, and part measurement technology and devices, that is used to obtain qualitative and quantitative information needed to assess or solve a geotechnical problem.*

As indicated in Fig. 10.1, the fundamental objective of geotechnical instrumentation is to provide the information that research workers, designers and contractors need in order to carry out their work. In this respect, instrumentation should be considered as a tool that extends the capabilities of an experienced observer, but it cannot replace him.

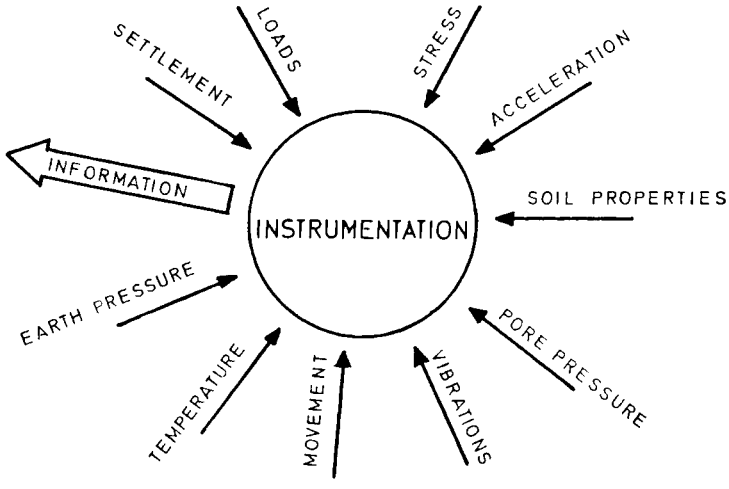

Fig. 10.1. Function of geotechnical instrumentation.

Because instrumentation provides a means of acquiring information, and because information is vital to the advancement of any field of science, it is not surprising that instrumentation has become one of the most important implements in the geotechnical tool box. The recognition of this fact, together with significant improvements and advances in measurement technology, has contributed to the present day situation, wherein instrumentation is exceedingly popular and in widespread use.

Present day emphasis on instrumentation over a broad range of applications, together with the increased complexity of modern measuring devices and installation techniques, requires that much of the instrumentation work must be planned, executed and followed-up by specially trained personnel. As a result of this trend, the instrumentation specialist has come into being in our profession. Competence in measurement technology alone, however, does not guarantee that the results of an instrumentation programme will be useful to the solution of the geotechnical problem being investigated unless the geotechnical problem itself is well understood and the objectives of the instrumentation programme are clearly defined. It is the responsibility of the geotechnical expert to establish these objectives, whereas it is the role of the instrumentation specialist in the design and execution of an instrumentation project to propose and use the simplest systems and methods that will yield the required information, reliably and at minimum cost.

If the results of an instrumentation programme are to be of any practical value to the practicing engineer, or if they are to have any significant bearing on the development of our profession by contributing positively to a better understanding of fundamental principles and thus advance the state-of-the-art, at least three essential conditions must be satisfied. These are:

(1) The purpose of the instrumentation programme and the underlying geotechnical problems that create the need for instrumentation must be clearly defined and understood. Instrumentation without any real purpose is only a waste of time and resources.
(2) The instrumentation programme must be comprehensive and carefully planned to include observations or measurements of all the quantities that, to the best of our knowledge, enter into the problem under study. Although incomplete data is better than no data, at best it will only produce incomplete results.
(3) The data collected during the course of an instrumentation programme must be reduced to a convenient form, interpreted or analyzed by competent engineers, and the results, whether positive or negative, must be made available to the profession without unnecessary delay. Data accumulated in a filing cabinet does not advance the state-of-the-art for the entire profession.

The application of instrumentation to problems dealing with soft clay does not differ significantly from cases where other geotechnical materials are involved. The underlying reasons for initiating instrumentation projects, the manner in which they are carried out, the goals and the pitfalls are essentially the same regardless of the material. Fundamental rules and concepts that govern the use of geotechnical instrumentation are applicable to most soils, although there can be marked dissimilarities in detail between the use of instrumentation in soft clay compared to its applications in other materials. These differences may be quite important. For instance, soft clay deposits are generally more homogeneous than other soil media, so that the amount of instrumentation needed in clay is often considerably less than that required in other materials. Further, the installation of instruments in soft clay is usually easier, less complicated and less expensive, and relatively light and simple equipment can often be used to press instruments into the natural ground without expensive and time-consuming pre-boring.

The amount of soil disturbance caused by the installation of instruments in soft clay can be substantially less than in other materials, and this can be a significant factor for some instrumentation applications. Certain types of measurements that are essentially impossible to realize in hard clay or granular soils, such as direct measurements of in-situ stresses within a soil medium, may have a better chance of success in soft clay. For example, it is sometimes possible to develop tools and installation procedures that can be used to insert special instruments into soft clay with little disturbance. Recent attempts to measure in-situ values of K_0 in soft clay have been successful for this reason.

The design of instruments for use in soft clay is often easier than for other materials. Because of the particle size of clay, it is generally not difficult to develop instruments that, for geotechnical reasons, must satisfy basic design criteria that are a function of the grain size of the medium. It is much easier,

for example, to construct a membrane-type earth pressure cell that satisfies accepted geometrical design requirements, such as size and permissible diameter/deflection ratio, for use in clay than for use in gravel. On the other hand, deformations associated with soft clay can be so large that it is difficult to provide instrumentation capable of operating over the full range of deformation encountered, or to do installation work in such a way as to avoid damage to instruments and connecting hydraulic lines or electrical cables as a result of these large movements.

10.2 PLANNING OF INSTRUMENTATION PROGRAMMES IN SOFT CLAY

10.2.1 Objectives of instrumentation programme

The principal objective of any geotechnical instrumentation project is to provide quantitative and qualitative information on the parameters that are relevant to the engineering problem on hand, regardless of the type of soil medium. Instrumentation programmes are generally initiated either as part of a research project or as a means of obtaining information necessary for the successful design and execution of a construction project, or to verify that the performance of a completed engineering work is acceptable and in agreement with predicted behaviour. Sometimes the favourable situation exists wherein the motivation for implementing an instrumentation project is a combination of basic research and immediate practical application.

Experience has shown that nearly all instrumentation applications can be divided into four main categories or combinations of these depending on the objectives of the project, namely:

(1) *Instrumentation for site investigation and evaluation of soil parameters.* The ability to obtain an accurate and complete picture of site conditions and of the engineering properties of the materials encountered improves continuously through the increased use of field instrumentation and special devices. Examples of this category of instrumentation application are: in -situ determinations of stress-strain and strength properties, permeability and in-situ stresses, and the use of penetration tests, plate tests and pile loading tests.

(2) *Instrumentation for construction control.* Probably the most common applications of instrumentation fall into the category of construction control. They may consist of a simple documentation of ground movements, changes in ground water level, intensity of vibrations, etc, in order to protect the owner or contractor in case of claims for damage incurred during construction. In other cases, control observations may be necessary to verify that the project is being carried out according to the specifications in the contract documents. The most important reason for control instrumentation is to obtain information which can

be used to evaluate the safety of a project during construction, and thereby avoid injury, loss of life or failure of the project. When instrumentation is used for this purpose, the measurement provide a hazard warning if abnormal conditions arise, and they serve as a basis for initiating contingency action if the situation is judged to be critical. Instrumentation can be used in a somewhat similar manner to provide the input that is required for a 'design as you go' approach to an engineering problem.

(3) *Instrumentation for performance measurements.* Quantitative and qualitative information about the behaviour of completed structures is necessary if the state-of-the-art is to be advanced. Performance measurements provide vital feedback to the design engineer, and they enable him to evaluate his design assumptions and field predictions and to modify his procedures to obtain a better design on future applications.

(4) *Instrumentation of full-scale tests.* Full-scale field tests and large models generally require extensive amounts of instrumentation. One common application is the validation of design or construction concepts before they are put into general use, and tests of this kind are often the only way to arrive at satisfactory solutions to many engineering problems. Tests in this category include those on footings or piles, trial embankments, and full-scale tests to evaluate a new construction technique or to investigate the applicability of existing construction methods for projects where the soil conditions differ from past experience.

10.2.2 Parameters most frequently measured

Although instrumentation may not be necessary for all geotechnical projects, it is relatively easy to argue that nearly every conceivable type of project can benefit from some form of instrumentation. It is not surprising, therefore, that the number and types of instrumentation application reported in the literature are extremely large and varied. Even though a long list would be necessary to catalogue the different types of instrumentation applications encountered in practice, the physical quantities measured are surprisingly few in number. Apart from direct field measurements of engineering properties such as shear strength, compressibility, permeability and in-situ stresses, the most frequently observed quantities are force, stress and strain, linear and angular displacements, and pressure. Measurement of these physical quantities is all that is needed to arrive at the parameters relevant to most geotechnical problems in soft clay, namely:

(1) surface and subsurface ground movements,
(2) settlement, tilt and horizontal displacement of structures,
(3) stress, strain and load in structural elements,

(4) pore pressure in the ground and on boundaries of structures, and
(5) earth pressures on contact surfaces between soil and structure.

In addition to these basic parameters, the need sometimes arises for supplementary measurements, such as ground temperature, frost penetration, flow of water, vibrations and accelerations.

Although present day engineers have at their disposal an ever increasing number of modern instruments and techniques which can be used for instrumentation purposes, it is still not possible to measure satisfactorily all the quantities that are needed. Table 10.1 contains a list of the physical measurements that are frequently included in instrumentation programmes in soft clay. For each entry in the table, an attempt has been made by the Authors to comment on relative frequency of occurrence, types of project involved and level of confidence in the results of measurements. Additional information concerning confidence in measured data can be found in the paper by DiBiagio (1975).

10.2.3 Selection of instrumentation systems

People who do instrumentation work must eventually decide the type of instrument best suited for their particular application. This in itself is not always a simple matter. As more and more emphasis has been placed on instrumentation projects, there has been a corresponding increase in the number of different measurement techniques employed in instrumentation schemes. At the same time, the number and types of instruments available has increased, literally by leaps and bounds, to the extent that the geotechnical instrumentation market is presently flooded with an inventory of equipment that ranges from simple, inexpensive mechanical devices to highly complex and expensive electronic instruments. The net result of this development is very favourable, on the whole, because the chances of carrying out a successful instrumentation project are increased if there is a larger selection of instruments to choose from. This abundance of instrumentation creates, on the other hand, another problem that is not always easy to solve: namely, how does the engineer select the proper instrument for the job on hand?

For the majority of prospective users, the choice of instrument is restricted to commercially available equipment. Others who have facilities to develop and produce instruments have a definite advantage, but they nevertheless must determine the practical limit of their own development work. Commercial instruments do not always satisfy the physical or operational requirements of special applications, especially research projects. Furthermore, delivery times are determined by the manufacturer and not by the user, and these can be totally unacceptable in some instances; for example, for unexpected or emergency applications of the 'fire-alarm' type.

Typical instruments used in field instrumentation projects by the Authors at the Norwegian Geotechnical Institute are shown in Fig. 10.2.

Table 10.1. Review of commonly measured parameters for construction projects in soft clay

Physical measurement	Occurrence	Typical application	Typical method of measurement	Reliability
Settlements of ground surface and structures	Frequent	Excavations	Surveying	Very good
Subsurface settlements	Occasional	Tunnels	Extensometers	Good
Horizontal displacements at ground surface	Occasional	Embankments	Surveying	Good
Horizontal displacements within clay mass	Frequent	Excavations	Inclinometer	Good
Elevation of ground water-table	Frequent	Cut slopes	Observation wells	Very good
Pore pressures within clay mass	Frequent	Embankments	Piezometers	Good
Pore pressures on boundaries of structures	Occasional	Retaining walls	Piezometers	Very good
Earth pressures within clay mass	Seldom	Embankments	Transducers	Poor
Earth pressures on boundaries of structures	Seldom	Retaining walls	Transducers	Good
Total load on single pile	Frequent	Pile foundations	Load cell	Very good
Pile point resistance	Seldom	Friction piles	Load cell	Good
Load distribution in pile	Seldom	Pile foundations	Strain gauges	Fair
Forces on struts	Frequent	Excavations	Strain gauges	Good
Forces on ground anchors	Occasional	Excavations	Hydraulic jacks	Good
Forces on tunnel liners and supports	Seldom	Tunnels	Strain gauges	Fair
Stresses in reinforcing steel	Seldom	Concrete piles	Strain gauges	Good
Stresses in concrete	Seldom	In-situ piles	Embedment gauges	Fair

Fig. 10.2. Typical field instruments used at the Norwegian Geotechnical Institute.

As a general rule, it is sound instrumentation policy to utilize the simplest type of instrumentation system that will enable the desired data to be obtained. For this reason, unsophisticated but reliable mechanical and hydraulic systems are often preferred and are employed quite frequently. Because of accessibility problems, however, or because frequent observations are required over a very long interval of time, or simply for convenience and expediency, it is often an advantage to have a more sophisticated measuring system or one that will permit remote observations. The selection of the most preferable instrumentation system and its components is usually made on the basis of:

(1) costs involved,

(2) required accuracy,

(3) environmental and operational conditions, and

(4) qualifications of the personnel who are to install and use the proposed instrumentation.

When evaluating a contemplated instrumentation system, the total cost of the system has to be considered. This should include not only the cost of the instruments, but installation costs and the cost of carrying out the required observations as well. The importance of the latter cost item should not be overlooked, for there are many instrumentation schemes for which material and installation costs are low but the cost of obtaining readings is relatively high, and vice versa. For example, if strut loads are determined with the use of a hand-held mechanical strain gauge or extensometer, nearly all the cost is associated with making the observations. These instrument readings will generally be expensive because of difficulty in gaining access to the reference points on the struts, and because a good deal of time is usually required for each set of measurements even for an experienced observer. On the other hand, if strut loads are determined by means of remote-reading electrical strain gauges or load cells, the material and installation costs will be high but the cost of making measurements will be relatively low. On a project where frequent strut load observations are required, therefore, the overall cost of obtaining the data by means of expensive electrical devices that can be remotely read may be significantly less than the cost of using an inexpensive mechanical strain gauge.

Accuracy and sensitivity requirements for instrumentation systems are dictated by geotechnical or structural criteria relevant to the project that is to be instrumented. Usually, there is no difficulty in fulfilling these requirements for the instrumentation applications most frequently encountered in soft clay, because many of the instruments and measurement techniques commonly used have acceptable accuracy and sensitivity characteristics. The final selection of the type of instrumentation to be used will generally be based on factors other than accuracy and sensitivity.

A proper assessment of operational requirements and environmental conditions is essential if a proposed instrumentation project is to be successful. The selection of instruments must be based on important details such as robustness, watertightness, corrosion resistance, long-term stability, immunity to extreme temperature change or gradient, and the preferable type of output signal for electrical transducers. During the design of systems, key factors such as serviceability, need for redundancy, access, cable routing, and mechanical protection, must be carefully considered. Furthermore, interference to the contractor caused by pipes, casings, cables and other installations must be avoided wherever possible.

The final point to be considered in the selection of an instrumentation system is an obvious one, but nevertheless it is one that is often overlooked. The instrumentation system, the method of installation and the read-out facilities employed should be compatible with the technical qualifications and skills of the personnel who will install the system and carry out the observations.

10.3 BASIC INSTRUMENTATION SYSTEMS

This section is devoted to a brief review of well-known measurement principles that are commonly incorporated into instrumentation programmes for soft clays. The types of measurement considered are vertical displacements, horizontal displacements, loads, strains and stresses in structural elements, pore water pressures and earth pressures. Examples are given to illustrate the more commonly used measurement techniques, but these do not cover all the available methods, nor do they necessarily represent the best methods for all applications.

10.3.1 Measurement of vertical displacement

Of all the different types of observation that are made for control and performance monitoring purposes, settlement measurements are undoubtedly the easiest to accomplish. Surveying techniques are used for the majority of settlement measurements, but a number of special instruments are used as well. Four commonly employed techniques for monitoring settlement are shown in Fig. 10.3.

With a *'tell-tale' rod* installation (Fig. 10.3a), the settlement of a reference plate is measured relative to a central cased rod driven to great depth or to a

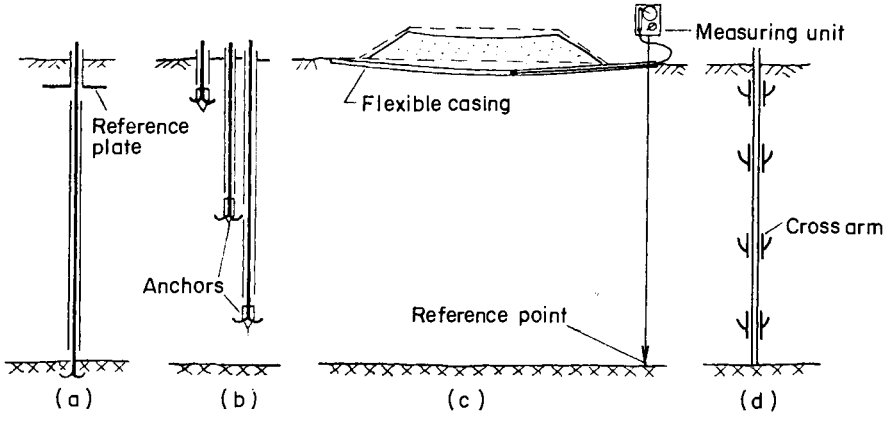

Fig. 10.3. Common techniques for monitoring settlement: (a) tell-tale rod, (b) Borros anchor, (c) full profile measurement, (d) borehole extensometer.

firm stratum. If the relative settlement is measured with a mechanical dial gauge, for example, the measurements can be very accurate, and this type of installation is often referred to as a *precision settlement gauge*. The central rod is often used as a bench-mark for other settlement observations.

The *Borros anchor* (Fig. 10.3b) is a special type of settlement reference point that is used frequently for near-surface and subsurface settlement observations by means of optical levelling.

Full-profile measurements (Fig. 10.3c) comprise continuous settlement profiling with the aid of special instruments pulled through a flexible casing. Probes containing inclinometers or sensitive pressure transducers that are pulled through a fluid-filled casing have been used for this type of measurement.

Borehole extensometers (Fig. 10.3d) are used for accurate measurement of distances between two or more points along a common axis. They consist of one or more reference points implanted in the ground with a reference point at the top of the installation. Rods and wires or electrical devices are used to determine the change in distance between the reference points.

10.3.2 Measurement of horizontal displacement

Accurate measurements of horizontal movements of structures and points on or below the ground surface are necessary for a complete evaluation of many geotechnical engineering projects. The majority of these measurements are made using surveying techniques or by means of special instruments. Three common methods of measuring horizontal displacements are shown in Fig. 10.4.

Horizontal displacements of points below the ground surface are commonly measured by means of borehole *inclinometers* that are used to survey special casing installed in the ground (Fig. 10.4a). The same procedures are used to measure the lateral displacement of sheeting, foundation piles and parts of structures below ground. The accuracy of modern instruments used for these

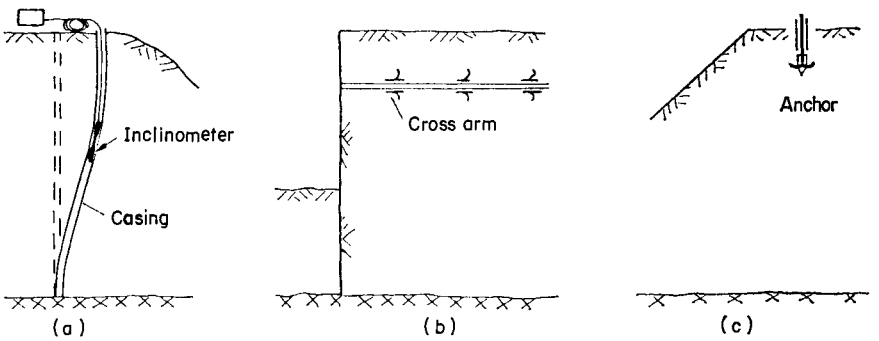

Fig. 10.4. Techniques for measurement of horizontal displacement: (a) inclinometer, (b) horizontal extensometer, (c) offset measurement.

measurements is very good. Measurements are time-consuming, and extensive computations are required.

Mechanical or electrical borehole extensometers are often installed behind a structure in order to monitor ground movements and lateral displacements of the structure (Fig. 10.4b). This type of installation, which is called a *horizontal extensometer,* has the advantage over conventional inclinometer measurements (described above) in that the observations can be made and interpreted much faster. Furthermore, the accuracy of some borehole extensometers is far superior to the accuracy that can be obtained with the use of inclinometers.

Lateral movements of reference points on the ground surface are normally monitored by surveying techniques such as chaining and triangulation or by offset measurements from a fixed theodolite line (Fig. 10.4c). These are called *offset measurements.*

10.3.3 Measurements of load and strain in structural elements

Forces and stresses in steel and concrete load-carrying members are frequently measured in connection with projects involving soft clay. Load cells, stress meters and strain gauges are the principal types of instrument used for this purpose. Five typical situations where observations of this kind occur are illustrated in Fig. 10.5.

Calibrated *hydraulic jacks* can be used to determine strut loads and forces in, for example, anchor tendons by measurement of the force required to unload or free a member by means of a hydraulic jack (Fig. 10.5a). This method is not suitable, however, in situations where frequent observations are required.

For many types of structure, loads and stresses can be determined reliably by means of mechanical or electrical *strain gauges* (Fig. 10.5b).

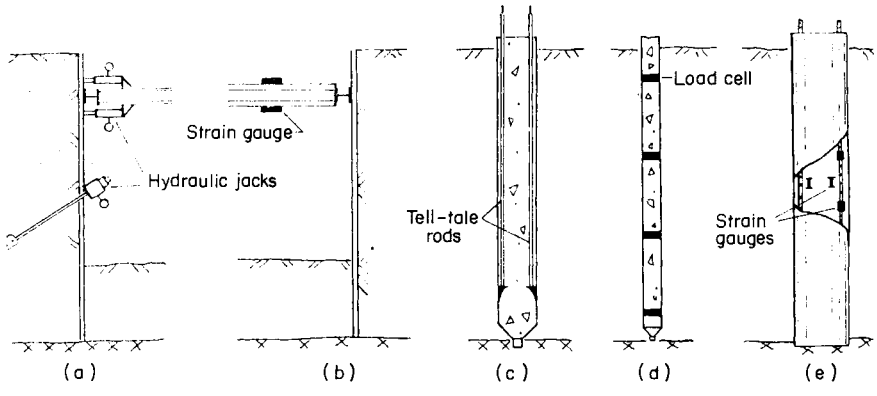

Fig. 10.5. Methods for monitoring load, stress and strain: (a) calibrated hydraulic jack, (b) strain gauge, (c) extensometer, (d) load cell, (e) embedment strain gauge.

In some cases, 'tell-tale' rods or *precision extensometers* are used as long-base strain gauges for deformation measurements. An example of this is the monitoring of the compression of a pile in order to determine the load distribution (Fig. 10.5c).

If parts of a structure can be separated to permit installation of calibrated *load cells,* then forces can be measured with a great deal of accuracy. Figure 10.5d illustrates a test pile which consists of a series of short pile sections connected together by load cells.

For concrete structures, it is sometimes necessary to supplement surface strain measurements with internal measurements by means of embedded strain gauges or stress meters and instrumented reinforcing steel (Fig. 10.5e).

10.3.4 Measurement of pore water pressure

Pore water pressure is a key geotechnical parameter for problems concerned with soft clay. Thus, it is fortunate that there is a relatively large number of instruments available for monitoring pore water pressure within a clay deposit or on the boundaries of a structure. Four types of measuring system commonly used are shown in Fig. 10.6.

The open *stand-pipe* (Fig. 10.6a) is the simplest and least expensive type of piezometer. It can be read directly and is accurate. It is generally installed in a borehole, but some variations can be pressed or driven into soft clay deposits. The major disadvantage is the relatively long time lag between a pore pressure change and its registration in the stand-pipe.

In *electric piezometers,* an electrical sensor responds to strain or deflection of an elastic element that deforms under pressure (Fig. 10.6b). The principal advantages of these instruments are remote reading capability, continuous output and rapid response time. The major drawback is the high cost per

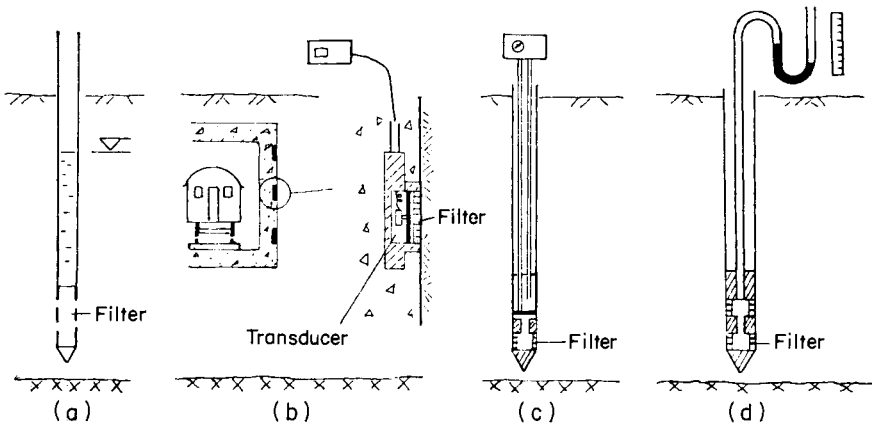

Fig. 10.6. Typical pore pressure measuring instruments: (a) open stand-pipe, (b) electric piezometer, (c) pneumatic piezometer, (d) closed hydraulic piezometer system.

unit. Their accuracy is generally acceptable. Some electric piezometers can be calibrated in-situ, and this is a great advantage for long-term measurements.

Pneumatic piezometers (Fig. 10.6c) are similar to electric piezometers except that air or a liquid is used to counterbalance and monitor the pore pressure acting on the piezometer. They can be remotely read, but they generally do not give a continuous output and cannot easily be incorporated into data acquisition systems. Pneumatic piezometers cost less than most electric piezometers.

With *closed hydraulic piezometer* systems (Fig. 10.6d) a U-tube manometer or other type of pressure gauge is attached to the piezometer in order to reduce the flow of pore water in the system and thereby improve the response time.

10.3.5 Measurement of earth pressure

Measurements of the magnitude and distribution of earth pressures on structures in soft clay can be achieved with confidence in some cases, but these are not of a routine nature. Three common instrumentation procedures for monitoring earth pressures are illustrated in Fig. 10.7.

Membrane-type pressure gauges (Fig. 10.7a) generally convert earth pressure into electric or pneumatic signals. Several types suitable for use in soft clay are obtainable from commercial sources. These instruments can be fitted to most structures and foundation elements for monitoring earth pressures.

Capsule-type pressure gauges generally consist of a flat fluid-filled body connected to a pressure sensing device (Fig. 10.7b). They can be made in a variety of sizes and shapes not possible with membrane-type gauges. These gauges are normally installed on structures to measure contact earth pressure, but because they can be made quite thin and can be pressed into soft clay with little disturbance, instruments of this kind are now being used on an experimental basis to measure in-situ stresses.

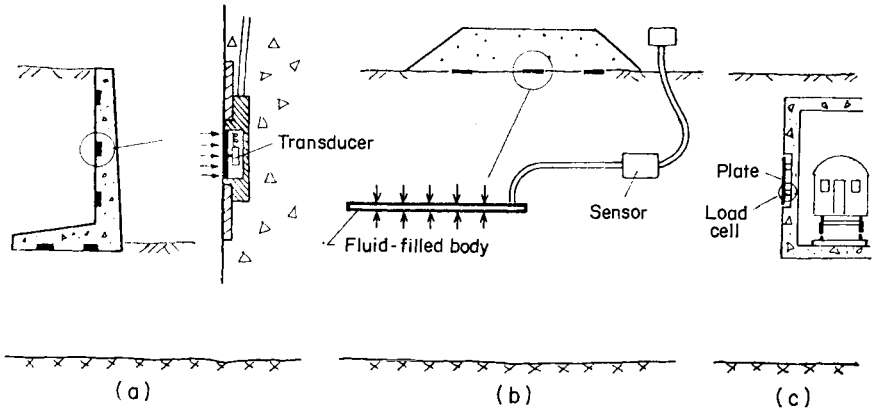

Fig. 10.7. Common techniques for measurement of earth pressure: (a) membrane-type pressure gauge, (b) capsule-type pressure gauge, (c) integrated pressure measurement.

Earth pressures are occasionally determined indirectly from measurements of the total force acting on a known area of a structure. Such *integrated pressure measurements* are generally effected by isolating a small section of the structure and supporting it on load cells in such a way that the total force on the section can be measured (Fig. 10.7c).

10.4 EXAMPLES OF INSTRUMENTATION APPLICATIONS FOR PROJECTS IN SOFT CLAY

In order to summarize present day instrumentation concepts and procedures, this section is devoted to hypothetical examples that illustrate the use of instrumentation for the most frequently encountered engineering projects in soft clay. Projects that generally require extensive amounts of instrumentation have been selected for presentation in order to illustrate as many different types of instrumentation applications as possible within the limited number of examples given. The eight examples included are concerned with strutted excavations, embankments, tunnels, excavated slopes, ground anchors, driven piles, cast-in-situ piles and slurry trench excavations. The examples are based on the Authors' experiences and on case studies reported in the literature. It is hoped that these examples will illustrate current instrumentation practice and provide guidance as to the merits as well as the limitations of instrumentation programmes.

A complete appreciation of the role that instrumentation plays in an engineering project is not possible unless the reasons for initiating the observational programme in the first place are fully understood. In each of the following examples, therefore, a brief description is given of the relevant geotechnical problems and the design or construction problems that are peculiar to the project, and the function of the instrumentation in relation to these problems is pointed out. The scopes of routine and specialized monitoring programmes are summarized, and an attempt is made to illustrate how instrumentation can be used to solve or provide a better understanding of the relevant geotechnical and practical problems.

10.4.1 Strutted excavations

10.4.1.1 Need for instrumentation

Depicted in Fig. 10.8 is a typical strutted sheet-piled excavation in soft clay where the sheet piles are not driven to bedrock or to a firm stratum. The design and construction of excavations of this type are based for the most part on empirical design procedures and past experience. Because the consequences of a failure may be catastrophic, they are frequently well instrumented.

Monitoring programmes for braced excavations will generally include:

(1) Routine instrumentation for the measurement of settlements, heave, strut loads, ground water level, and horizontal displacements.

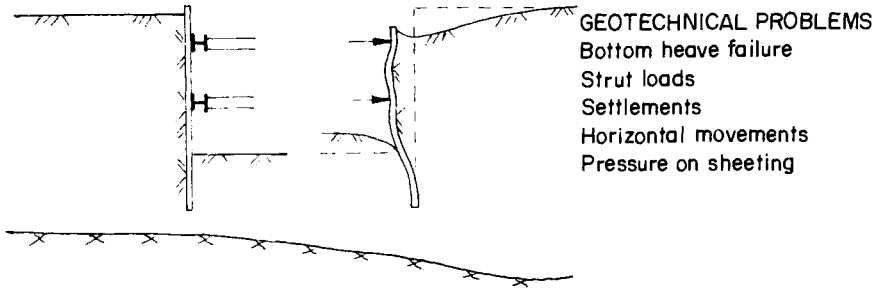

Fig. 10.8. Geotechnical problems with strutted excavations.

(2) Specialized instrumentation for the measurement of pore pressures in soil, earth pressures and water pressures on sheeting, stresses in sheeting and wales, and frost pressures and penetration.

10.4.1.2 Role of instrumentation

Instrumentation can be useful in evaluating the overall stability of an excavation and can provide assurance that the work is proceeding safely. Results of measurements will document the causes and extent of any damage to adjacent structures and utilities. Alterations to proposed excavating and bracing procedures can be initiated on the basis of the measurements in order to assure adequate safety or improved economy. Well instrumented and documented case histories provide the basis of improved knowledge and refinement in design procedures.

The main geotechnical problems (Fig. 10.8) and the suggested instrumentation applications (Fig. 10.9) for strutted excavations are:

(1) *The factor of safety against bottom heave failure may be low.* Instrumentation should be installed to permit bottom heave measurements, and

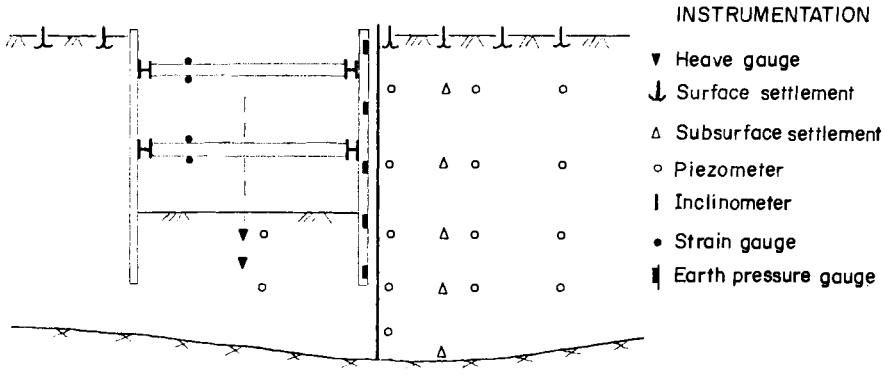

Fig. 10.9. Typical instrumentation for strutted excavations.

heave and settlement should be monitored frequently as the excavation level approaches the critical depth. Particular attention should be paid to settlement and heave rates. The rate of horizontal displacement of the lower part of the sheet piling can be a useful parameter for evaluating instability of the excavation.

(2) *There is a danger of overloading individual struts.* Strut loads should be monitored on as many struts as is practical or economically justified. Reserve struts should be available for immediate installation if measurements indicate allowable limits have been reached.

(3) *Ground movements due to deformation of sheeting and bottom heave may damage nearby structures or utilities.* Reference points should be established for monitoring the settlement of the ground surface and structures adjacent to the excavation. Deep settlement reference points should be established near important underground utilities. Horizontal displacements of the sheeting and on several profiles in the soil mass behind the sheeting should be monitored if there is danger of damage to structures or utilities.

(4) *Prolonged lowering of the ground water-table may cause serious consolidation settlements over a large area.* Pore pressures in permeable layers should be measured and, if economy permits, pore pressures with depth along several vertical sections behind the sheeting should also be monitored together with the elevations of the ground water-table near to and remote from the excavation.

(5) *The total load and distribution on the bracing system is unknown.* Sufficient struts on vertical sections should be monitored to enable the computation of the point of application of the resultant force, or the force transmitted to the wales from the sheeting at each strut level should be measured. The sheeting should be instrumented to permit the direct measurement of the magnitudes and distributions of earth pressures and water pressures against both sides of the sheeting.

Typical examples of instrumentation applications for strutted excavations are to be found in the publications by Peck (1969), Cunningham & Carpenter (1975) and Karlsrud & Myrvoll (1976).

10.4.2 Embankments

10.4.2.1 Need for instrumentation

Embankments are frequently built on layers of soft clay for highways, reservoirs and levees, or for full-scale loading tests carried out to investigate the engineering properties of the underlying soil. Inspite of a long and universal record of embankment construction throughout the history of civil engineering in soft clay, embankments that are designed with a theoretical factor of safety greater than unity fail embarrassingly often. On the other

hand, some test embankments that are designed to fail intentionally, never do. In recognition of this, it is not surprising that embankments on soft clay are frequently and extensively instrumented.

Monitoring programmes for embankments on soft clay will usually include:

(1) Routine instrumentation for the measurement of settlements of the embankment and the natural ground, pore pressures in the subsoil and horizontal spreading of the toe of the embankment.
(2) Specialized instrumentation for the measurement of settlements at different depths, horizontal displacements of the subsoil and fill, location of any failure plane, horizontal strains in the base of the embankment, in-situ stresses and permeabilities, and the surface loading caused by the embankment.

The problems which arise with embankments on soft clay are depicted in Fig. 10.10.

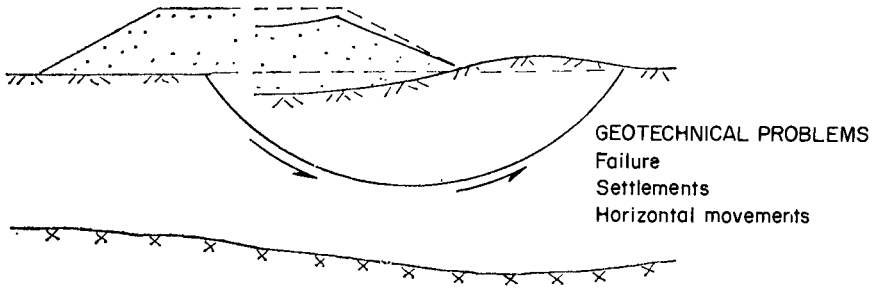

Fig. 10.10. Geotechnical problems associated with embankments.

10.4.2.2 Role of instrumentation

To be successful, an embankment must be designed and constructed such that neither the embankment material nor the underlying soil are overstressed to the point where the resulting deformations exceed the limits that are acceptable for the intended use of the embankment. Instrumentation is required, therefore, to monitor deformations as well as the factors that can be correlated to deformation and stability. Another need for instrumentation is to document that improved stability and a reduction in post construction settlements can be achieved by means of special construction procedures, such as sand drains, ground reinforcement, berms and preloading.

The factors which are important to the design and performance of embankments on soft clay, and the suggested instrumentation provisions (Fig. 10.11), are as follows:

(1) *Settlement data reflects the overall performance of an embankment.* Provision should be made for systematic observations of settlements

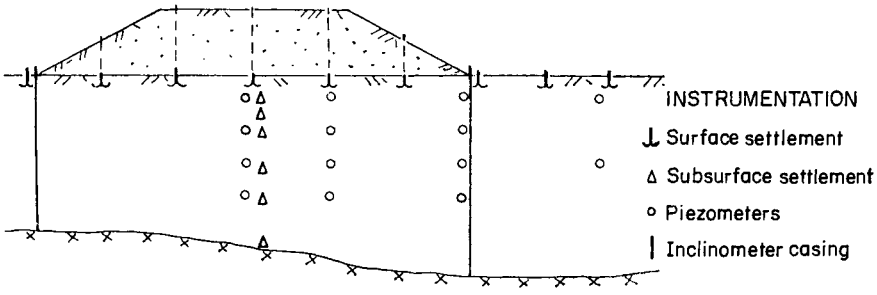

Fig. 10.11. Typical instrumentation for embankments.

of the ground surface adjacent to and beneath the embankment. Additional deep settlement points are useful, in particular if the subsoil is stratified. Both the magnitude and rate of settlement are important to an evaluation of the post-construction performance. In addition, the initial 'elastic' settlement caused by each increment of load as the fill is placed provides a basis for an appraisal of the stability during construction.

(2) *Pore pressures may provide the first indications of incipient failure.* A warning of impending failure of an embankment can often be obtained from a careful study of the measured settlements, deformations and pore pressures. A number of documented cases in soft clay show, however, that measured pore pressures may give a better first indication of failure conditions developing than can be derived from displacement data. If the factor of safety is low or questionable, therefore, and if the consequences of a failure are high, pore pressures should be monitored. In order to be certain that instruments are located in the zones of initial yielding, a relatively large number of piezometers may be required unless experience and proven methods of analysis can be used to locate the critical zones that require instrumentation. Pore pressure measurements are also essential to the evaluation of the progress of consolidation settlements and to aid in establishing the rate at which an embankment can be constructed.

(3) *Lateral yielding of the ground is an important evaluation and control parameter.* Horizontal strains at the ground surface must be measured, including the movement of the toe and the spreading of the embankment material. Inclinometer casing should be installed at several distances from the centre of the embankment to monitor horizontal displacements frequently during the placement of fill.

(4) *Location of the failure plane is important for a post-failure analysis.* Test embankments that are to be deliberately failed should be instrumented with mechanical or electrical devices that will permit location of the rupture surface.

Reference should be made to the papers by Eide & Holmberg (1972), Ladd (1972) and McKenna & Roy (1974) for descriptions of typical instrumented embankments on soft clays.

10.4.3 Tunnels

10.4.3.1 Need for instrumentation

Tunnel construction in soft ground has always been an important civil engineering activity, and these structures are becoming increasingly more important because of the need to locate utilities and transportation systems underground in congested metropolitan areas. Tunnels in soft clay are designed and built primarily on the basis of experience and empirical rules. Since they are generally built in congested areas, tunnel projects may cause serious damage to overlying structures and utilities. For this reason, monitoring programmes are commonly carried out on tunnel projects in order to verify that the work is proceeding safely, to provide information for improving future designs, or to obtain facts to be used in legal proceedings if damage claims arise.

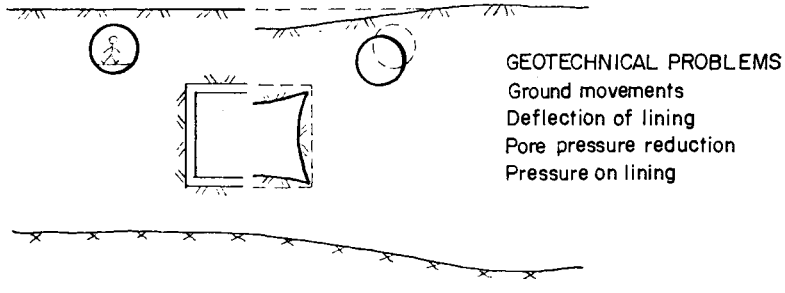

Fig. 10.12. Geotechnical problems associated with tunnels.

The problems that can arise with a tunnel in soft clay are illustrated in Fig. 10.12, and the instrumentation used for monitoring will usually include:

(1) Routine instrumentation for the measurement of settlements of the ground surface and the tilt of nearby structures, ground water levels, horizontal displacements, and deflections of the tunnel lining.

(2) Specialized instrumentation for the measurement of water pressure and earth pressure distributions on the lining, stresses or strains in the lining, and pore pressures in the soil.

10.4.3.2 Role of instrumentation

Instrumentation is needed in many cases to verify that a tunnel can actually be built in a safe manner using the prescribed construction procedures. This is generally investigated by the thorough instrumentation of a representative

section of the tunnel, called a *test section,* at the start of tunnel driving. Instrumentation is also needed to detect conditions, such as large soil movements or lining deflections, that may lead to excessive settlements and damage to overlying structures. Finally, instrumentation is needed to check that the performance of the completed tunnel is in accordance with the plans and specifications.

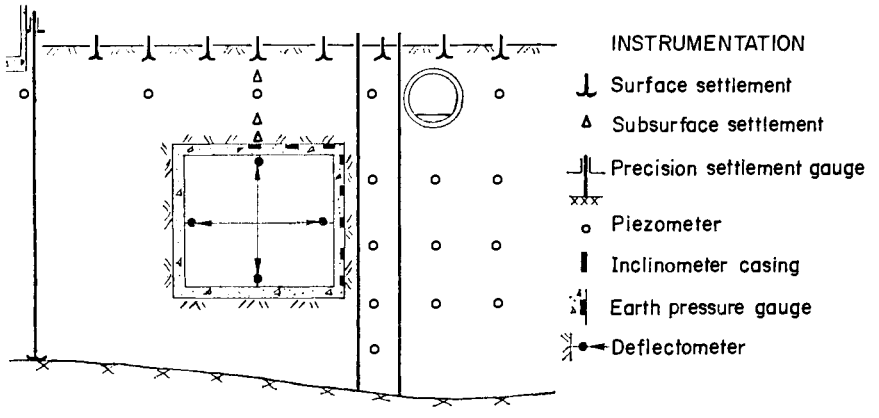

Fig. 10.13. Typical instrumentation for tunnels.

The main geotechnical problems and the suggested instrumentation applications (Fig. 10.13) for tunnels in soft clay are:

(1) *Tunnels cause settlements and settlements cause damage.* Reference points should be established at the ground surface above the centre-line of the tunnel and on some cross-sections, to permit settlements of buildings to be monitored along with the size and shape of the settlement trough as the tunnel advances. Deep settlement reference points should be established near important underground utilities and foundations.

(2) *The displacement field around a tunnel face is three-dimensional.* Inclinometer casing should be installed and horizontal displacements measured within the clay mass close to the tunnel. Combined with subsurface settlement data, a complete picture of the displacements caused by tunnelling will be obtained.

(3) *Lining deflections may aggravate the settlements and deformations.* Deflections of the tunnel lining need to be measured along several axes.

(4) *Prolonged lowering of the ground water-table may cause serious consolidation settlements over a large area.* The elevation of the ground water-table near to and remote from the tunnel should be monitored, together with pore pressures in the clay. Water pressures on the completed lining

should also be measured in order to evaluate the effectiveness of the seal provided by the liner.

(5) *The total load and the distribution of stresses in the lining are unknown.* Strain gauges should be installed on the lining to monitor the distribution of strains. The tangential forces transmitted through the lining can be measured by means of load cells mounted between lining elements, or alternately by means of specially instrumented and calibrated lining elements. The magnitudes and distributions of earth pressures and water pressures on the lining can in some cases be monitored by means of pressure transducers mounted on the lining.

Examples of instrumented tunnels in clay are given in the papers by Peck (1969), Rebull (1972) and Müller-Salzburg & Spaun (1977).

10.4.4 Excavated Slopes

10.4.4.1 Need for instrumentation

The construction of highways and canals, and the grading of building sites, often require that excavations with sloping sides (Fig. 10.14) be made in soft clay. Many of these are permanent slopes that must be stable indefinitely, while others are only temporary. The principal geotechnical design problem, therefore, is to provide a slope with the required short-term or long-term stability, and the role of instrumentation is to document whether or not the performance of the excavated slope is in accordance with the predicted behaviour. Instrumentation programmes for natural slopes are essentially the same as for post-construction man-made slopes.

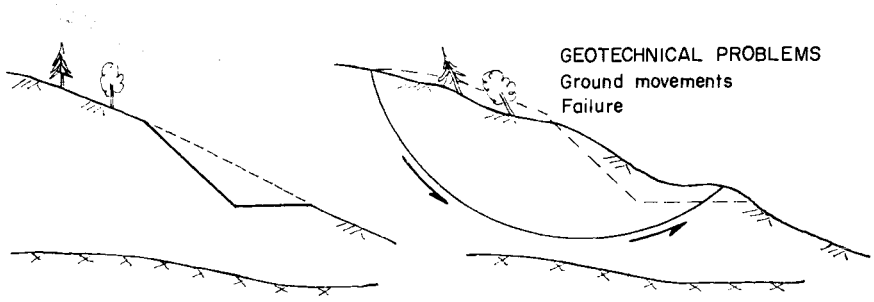

Fig. 10.14. Geotechnical problems associated with excavated slopes.

For excavated slopes, the instrumentation employed and the parameters measured are:

(1) Routine instrumentation for the measurement of surface strains, i.e. vertical and horizontal movements of points on the surface.

(2) Specialized instrumentation for the measurement of pore pressures and horizontal displacements within the soil mass.

10.4.4.2 Role of instrumentation

The instrumentation required for excavated slopes must provide information relative to the stability of the slope, both during excavation and on a short or long-term basis thereafter. Results of measurements can be used as a basis for the modification of the design slope angle, in order to assure adequate safety, or for reasons of economy. Instrumentation is also needed to document the effectiveness of ground reinforcement techniques and other specialized construction procedures for improving slope stability.

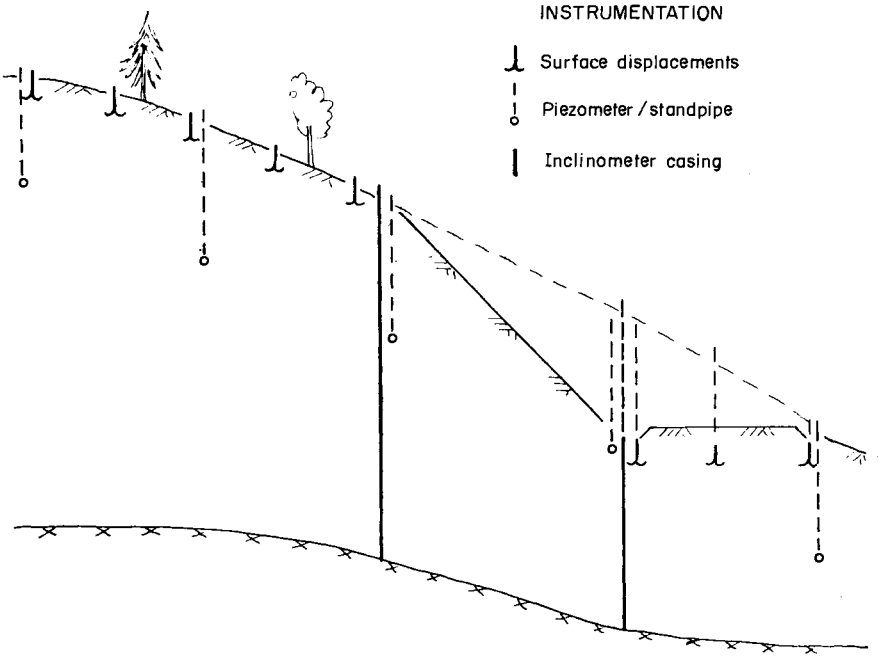

Fig. 10.15. Typical instrumentation for excavated slopes.

The observations which it is necessary to make on excavated slopes, and the methods to achieve these (Fig. 10.15), are:

(1) *A knowledge of the initial conditions is essential.* Observation wells and piezometers should be installed to monitor pore pressures and ground water levels in the area before excavation of the slope is started. A knowledge of the seasonal variations in water-table can be useful. To the degree that it is possible, instrumentation systems and reference points for the measurement of surface strains, settlements and horizontal displacements should be installed and initial measurements taken before excavation starts.

(2) *Observations during excavation provide clues to the performance of the completed slope.* Measured changes in pore pressures and observed displacements during excavation of the slope can be used in back computations to assess the validity of the soil parameters used in the original design computations. On the basis of these computations, new values can be estimated and used to predict the performance of the final slope.

(3) *Systematic monitoring is necessary to evaluate the long-term performance of a slope.* The development of shear zones or failure planes can generally be seen from the data obtained from the inclinometer surveys used to measure horizontal displacements of casings installed in the ground. Observed creep rates and pore pressure data may indicate instability and local yielding in the ground.

Details of typical instrumented cut slopes are contained in the publications by Wilson & Hilts (1972), Tice & Sams (1974) and Blondeau & Queyroi (1976).

10.4.5 Ground anchors

10.4.5.1 Need for instrumentation

Ground anchors (or tie-backs) are being applied to an increasing number of construction projects in soft clay, as for example in anchored excavations (Fig. 10.16). Anchors have the advantage over traditional bracing systems, such as struts and raker beams, in that the supporting elements are external and do not interfere with construction activities within the excavation area. In principle, an anchor consists of parallel wires, a spun cable or an individual steel rod anchored at one end in a cement-grout body embedded in the soil or in bedrock. The design and construction of anchors is based primarily on empirical procedures and past experience, and since the consequences of an anchor failure may be catastrophic, projects of this kind are seldom undertaken without a monitoring programme to check anchor performance.

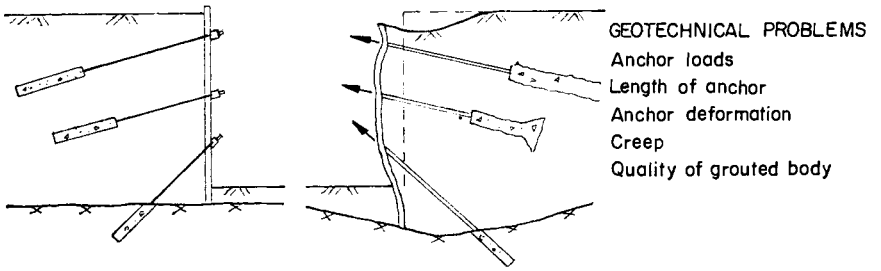

Fig. 10.16. Geotechnical problems associated with ground anchors.

The instrumentation for anchors normally consists of:

(1) Routine instrumentation for the measurement of the total load in the anchor, and the elastic strain and total elongation of the anchor assembly.
(2) Specialized instrumentation for the measurement of the distribution of load and strains along the length of the anchor, and the creep that occurs in the assembly.

10.4.5.2 Role of instrumentation

The maximum load carrying capacity of conventional supports, such as struts, can be determined accurately from the material properties and dimensions of each support. The ultimate capacity of ground anchors, on the other hand, does not generally depend on the properties of the tendons, but on the properties of the soil and on the size and shape of the embedded (grouted) body of the anchor. Consequently, computations of allowable loads for anchors are at best only rough approximations. Ground anchors should not be used on a project unless their holding power is verified by full-scale tests or unless the design is so overly conservative that a failure cannot possibly occur. Even this may not prevent a failure if the installation work performed by the contractor is inferior.

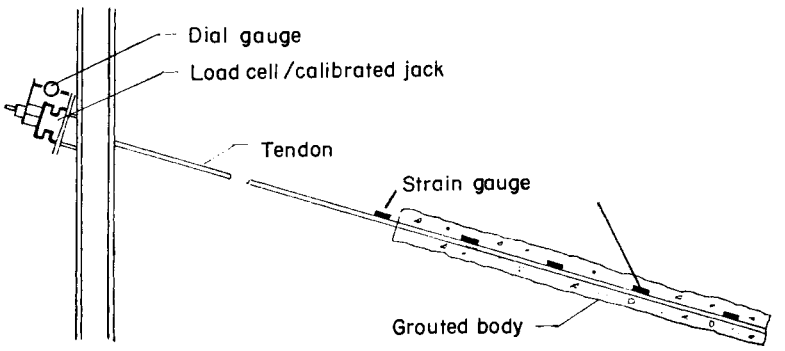

Fig. 10.17. Typical instrumentation for ground anchors.

The important factors to bear in mind for ground anchor design, and the instrumentation procedure necessary to control these (Fig. 10.17), are:

(1) *Full-scale tests are needed to evaluate the necessary embedded anchor length.* Test anchors must be constructed with embedded grouted lengths less than, equal to, and greater than the design length. These should be loaded to failure, and load and displacement monitored throughout each test. Each anchor should be pulled out if possible to

examine the size, shape and quality of the grouted body. As a supplement, each anchor can be instrumented with strain gauges or extensometers to investigate the distribution of load along the anchor.

(2) *All anchors should be test loaded.* In order to detect a defective anchor or to verify that the carrying capacity is within acceptable limits, each anchor should be test loaded to a value in excess of the design working load. The movement of the anchor head corresponding to the maximum test load should be recorded. Loads developed in anchors during construction should be measured periodically. On critical projects, load cells should be installed on a number of anchors to permit continuous monitoring of forces.

Instrumented ground anchors are described in the papers by Oosterbaan & Gifford (1972) and Ostermayer (1975).

10.4.6 Driven piles

10.4.6.1 Need for instrumentation

Pile foundations are often needed to provide adequate support for structures on soft clay. Driven piles, as either end-bearing piles or friction piles, are commonly used for these foundations. As is the case with most piles, the design of driven piles involves assumptions and uncertainties that need to be checked by means of instrumented full-scale tests. Furthermore, certain types of driven pile cause large displacements and pore pressure changes in the surrounding soil; these may in turn have a detrimental effect on neighbouring piles or on the stability of the site as a whole. An instrumentation programme is needed, therefore, to monitor the effects of pile driving as well as to establish the load-deformation characteristics of the piles.

Monitoring programmes for driven piles in soft clay will generally include:

(1) Routine instrumentation for the measurement of the load-settlement characteristics of the pile and the heave and horizontal movements of the ground surface during driving.

(2) Specialized instrumentation for the measurement of pore pressures, subsurface horizontal movements of the soil and the piles already driven, the distribution of vertical load along the length of the piles, bending stresses in laterally loaded piles, loads caused by negative skin friction, and the distribution of load in pile groups.

10.4.6.2 Role of instrumentation

Pile foundations must be able to carry the design loads safely and without excessive settlement. Full-scale tests on driven piles are necessary, therefore, to check geotechnical design procedures and driving criteria, or to document whether the load-settlement characteristics for a driven pile fall within

acceptable performance limits. Field tests may be required to establish the length of friction piles or to monitor the long-term loading caused by surface settlements and negative skin friction. Instrumentation is necessary to monitor pile driving operations and to detect factors, such as badly deformed piles or large bending moments, which may lower the bearing capacity of a pile.

The common geotechnical problems encountered with driven piles are depicted in Fig. 10.18, and the instrumentation required to provide design and performance information on these problems is shown in Fig. 10.19. The problems and the suggested instrumentation applications are:

(1) *A failure at the site may be triggered by the driving of displacement piles.* If topography and site conditions suggest that a slide or failure may be initiated by the driving of displacement piles, settlement reference points should be established and inclinometer casing and piezometers installed to monitor ground movements and pore pressures. Some types of pile can be fitted with inclinometer casing before driving and surveyed later to determine the displacements caused by driving neighbouring piles.

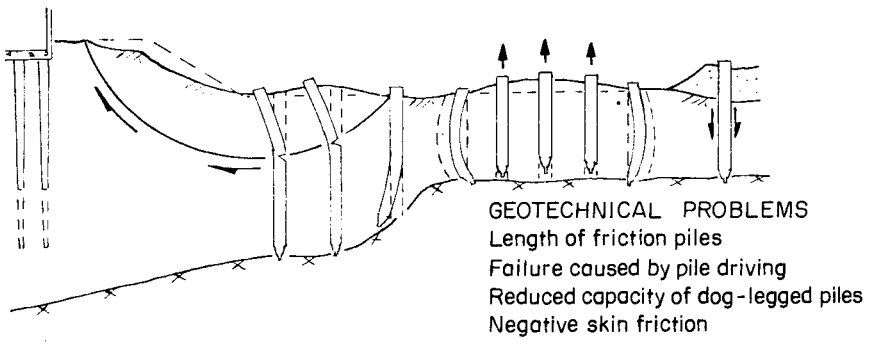

Fig. 10.18. Geotechnical problems associated with driven piles.

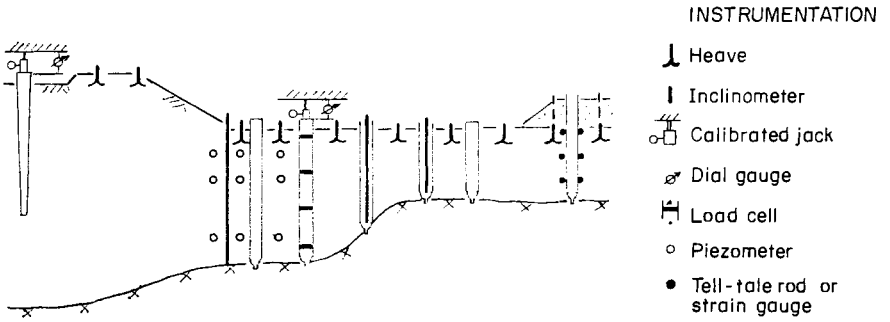

Fig. 10.19. Typical instrumentation for driven piles.

(2) *The design length of friction piles may be uncertain.* Full-scale load tests should be carried out. The evaluation of the results is simplified if a load cell can be mounted at the tip of the pile in order to separate the total load into point resistance and skin friction.

(3) *End-bearing piles driven to an irregular bedrock surface may have reduced capacities.* A dog-legged pile, caused by the deflection of the pile from a sloping bedrock surface, may have a reduced end bearing. Some building codes specify bearing capacity of piles as a function of the radius of curvature of the pile, and this curvature can be confirmed by means of inclinometer measurements. Alternately, the pile can be test loaded to verify its bearing capacity.

(4) *Lateral yielding of the soil may induce large bending stresses in the pile.* Non-uniform surface loading may cause substantial horizontal ground movements in soft clay, thereby causing displacements and undesirable bending stresses in piles. When these conditions are anticipated, piles can be instrumented with inclinometer casing or strain gauges to permit the determination of lateral movements and stresses.

(5) *Negative skin friction may cause overload of end-bearing piles or increased settlements of friction piles.* If large surface settlements are expected, piles can be instrumented to measure the long-term build-up of load. Precise mechanical extensometers to measure incremental strains along the piles are advisable because of long-term stability requirements. Alternately, piles can be equipped with load cells or strain gauges at various levels.

Some instrumentation applications to pile foundations in soft clay are described in the papers by Johannessen & Bjerrum (1965) and Fellenius & Haagen (1968).

10.4.7 Cast-in-situ piles

10.4.7.1 Need for instrumentation

Cast-in-situ point-bearing piles are commonly used for foundations in soft clay in order to avoid construction problems associated with ground movements caused by the driving of displacement piles. These piles are made by drilling or excavating a cylindrical hole in the ground and filling this with concrete. A temporary outer casing is generally required to support the sides of the hole in soft clay. Water or drilling mud may be used in advancing the borehole, in which case, the fluid in the hole is displaced by tremie concrete as the casing is withdrawn. Although assumptions must be made concerning design parameters for side shear and end bearing for cast-in-situ piles, the principal problems encountered with the use of this type of pile in soft clay are not related to design but to construction technique and control. Instrumentation is required, therefore, primarily for control purposes.

The principal instrumentation applications for cast-in-situ piles in soft clay are:

(1) Routine instrumentation for the measurement of the load-deformation characteristics of the completed pile.
(2) Specialized instrumentation for the measurement of the integrity of the pile and the quality of the concrete, the distribution of the vertical load in the pile, lateral movements, and the profiling of belled piles if visual inspection of the bells is not possible.

10.4.7.2 Role of Instrumentation

The danger of an undetected flaw in a pile is without doubt the most worrisome aspect of using cast-in-situ piles in soft clay (Fig. 10.20). This is particularly so for those cases where the temporary casing is extracted during concreting, or when the hole is filled with a stabilizing fluid and concrete is placed by the tremie method. In these situations, the chances of 'necking' in the pile or of a pocket of clay or drilling mud being trapped between the soil and the concrete are high. This is particularly true if the contractor is inexperienced or careless. The main objective of instrumentation, therefore, is to provide information that can be used to evaluate the soundness of the completed pile.

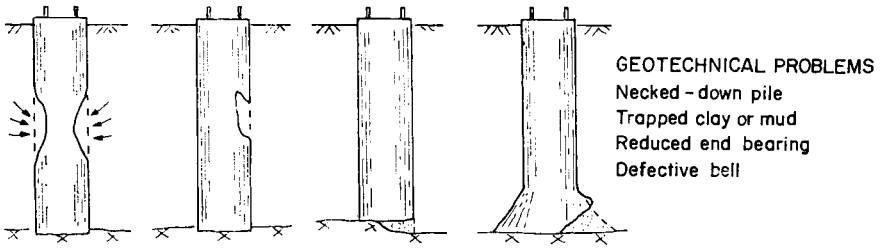

Fig. 10.20. Geotechnical problems associated with cast-in-situ piles.

The functions of instrumentation in cast-in-situ piles (Fig. 10.21) are:

(1) *Load tests demonstrate the pile capacity and may indicate a defective pile.* Conventional load tests will prove the capacity of the pile. Strain gauges, hydraulic load capsules, pressure cells at the base, and precision extensometers or other special instruments to monitor load transfer and compressibility, will provide information useful to the interpretation of the pile behaviour. An unexpectedly low capacity may be because of a serious defect in the pile, such as a large clay-filled void or a reduced cross-section caused by necking. Large observed settlements of a pile designed to carry a significant load in end bearing may be a

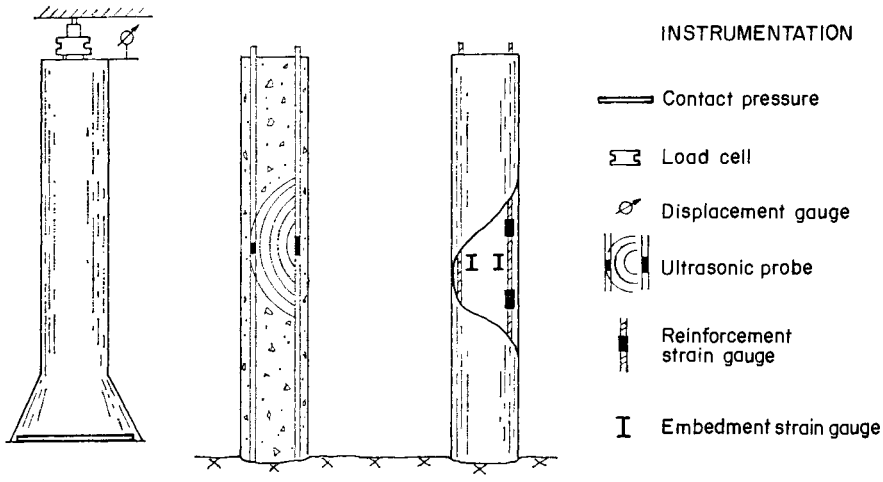

Fig. 10.21. Typical instrumentation for cast-in-situ piles.

result of remoulded clay and mud trapped beneath the pile because the bottom of the hole was not properly cleaned before the concrete was poured.

(2) *Non-destructive methods can be used to investigate the integrity of the concrete.* Continuous diamond core drilling is sometimes used to obtain concrete samples to assess the quality and to detect flaws. With long piles, however, this method is expensive and time consuming, and it can be complicated and misleading because of the alignment problems. An alternative procedure to detect flaws is to survey the pile with an ultrasonic borehole probe or similar type of device that can detect variations in the quality of the concrete. Such devices can be operated in a single hole, or cross-hole techniques can be used depending on the type of instrument and size of pile. The casings required for the survey can be cast integral with the pile during concreting.

Examples of the instrumentation of cast-in-place piles can be found in the papers by Frischmann & Fleming (1962), Whitaker (1963) and Paquet (1969).

10.4.8 Slurry trench excavations

10.4.8.1 Need for instrumentation

A slurry trench excavation is supported by an internal fluid instead of a conventional structural bracing system such as struts and sheeting. Otherwise, the design, control and performance monitoring problems, and also the consequences of a failure, are similar to those encountered with braced excavations; the instrumentation requirements, therefore, are analogous in many ways. There is, however, one fundamental difference between a conven-

tional braced excavation and a slurry-filled trench that emphasises the need for instrumentation of the latter. In a braced excavation, an experienced observer can see such things as unusually large deformations of the sheeting or signs of overstressing in the bracing system, that may give a warning of unexpected performance or even impending collapse; in many instances, serious difficulties have been avoided by the initiation of remedial action based on simple visual observations alone. On the other hand, it is not possible to see anything in a slurry-filled trench, and unexpected events and even a failure could occur suddenly and without warning unless a suitable instrumentation programme for monitoring the performance of the excavation has been provided for.

Monitoring programmes for slurry trench excavations in soft clay will generally include:

(1) Routine instrumentation for the measurement of settlements, horizontal displacements, levels of slurry and ground water-table, and the physical properties of the slurry.

(2) Specialized instrumentation for the measurement of the profile of the trench (width), the creep of the sides, the vertical alignment of the excavation, the pore pressures in the soil, the loading and deformations when the trench is concreted, the stresses and deformations in the completed wall, and the distribution of the water and earth pressures on the completed wall.

10.4.8.2 Role of instrumentation

Although contractors have extensive experience with the use of slurry to stabilize deep excavations in many types of soil, this contruction procedure has not been generally applied to soft clays. For this reason, a considerable effort has recently been devoted to the instrumentation of slurry trench excavations in soft clays in order to obtain answers to even the most elementary design assumptions. On slurry trench projects, it may not always be feasible or possible to integrate all aspects of planned instrumentation and monitoring programmes into the contractor's work routine. A full-scale instrumented test panel at the site may be necessary, therefore, in order to verify critical design assumptions before the main contract work begins.

The common construction problems with slurry trenches are depicted in Fig. 10.22, while typical instrumentation used is shown in Fig. 10.23. The important factors to remember are:

(1) *Ground movements provide simple and reliable indications of stability.* Settlements of the ground surface, guide walls and adjacent structures should be monitored. Settlements may be quite small, and settlement rate is a key factor in evaluating overall stability. At least one precision settlement gauge that can be read quickly and frequently, therefore,

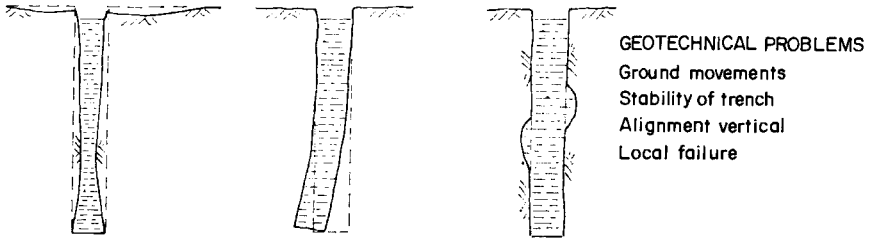

Fig. 10.22. Geotechnical problems associated with slurry trench walls.

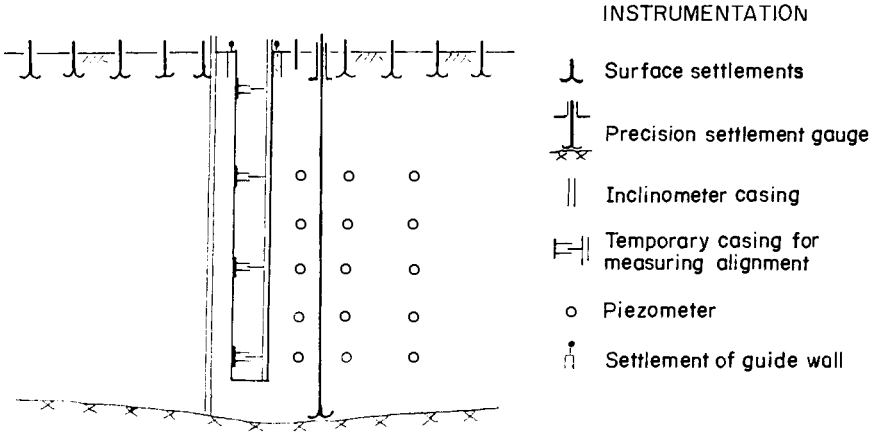

Fig. 10.23. Typical instrumentation for the construction of slurry trenches.

should be installed close to the excavation. At least one inclinometer casing should also be installed close to the edge of the excavation and horizontal displacements monitored frequently.

(2) *The position of the ground water-table relative to the level of the slurry is important.* The elevation of the top of the slurry inside the excavation as well as the elevation of the ground water-table near to and remote from the excavation should be monitored. Unexpected loss of slurry should be accounted for.

(3) *High density slurry is expensive and can complicate the concreting operations.* Some form of observational programme is required to establish the acceptable minimum density of the slurry to be used on the project. This fundamental parameter can be evaluated, of course, on the basis of a series of full-scale tests with slurries of different densities, but this is not always practical or economical. An alternative procedure is to excavate one panel using a slurry with a high initial density and to subsequently dilute the slurry in stages until the observed deformation or rate of deformation becomes excessive or a

collapse occurs. Tests of this kind require very accurate and frequent measurements of settlements and horizontal displacements in order to correlate the creep rate with density. Measurements of pore pressure versus time are also valuable in this respect. To obtain adequate creep data for the inward movement of the walls, it is advisable to install extensometers inside the trench at a number of levels.

(4) *End effects influence the maximum length of panel that can be excavated.* The limiting length/width ratio for which a single panel can be excavated safely can be established on the basis of full-scale test panels of varying length. Comprehensive measurements of the displacement field are needed to evaluate the restraint provided by the soil at the ends of a trench.

(5) *The configuration of the completed excavation may deviate from the designed shape.* As it is not possible to examine visually a completed excavation, instrumentation must be relied upon entirely for a final 'inspection'. The width of the excavated panel at any point can be determined with remote reading caliper tools lowered into the trench. The same instrument can also be used to search for local failures or overbreak, particularly in fissured clay where blocks of material may fall out during or subsequent to excavation. Errors in vertical alignment may have serious consequences on certain applications. Verticality can be measured using conventional inclinometer surveying techniques inside flexible casing lowered temporarily into the trench and pressed against one side (by small pneumatic jacks, for example).

(6) *The length of time for which an excavated panel may be kept open is important.* As a rule, a contractor will complete a section of slurry trench wall as quickly as possible in order to minimize the time the excavation is kept open. There are occasions, however, when an excavated panel must be kept open for longer periods of time, and the effect of time on overall stability becomes an important factor to consider. One way to evaluate this factor is to monitor settlements and lateral displacements very carefully with time, as described above, and to make conclusions based on the magnitudes of the observed deformations and whether the rate of deformation is increasing or decreasing with time. Trends in measured pore pressure in the surrounding soil can also serve as an indication of the rate of deterioration of the excavation.

(7) *The local stability of the excavation may be uncertain during concreting.* The filling of a deep slurry trench excavation with fresh concrete causes a significant increase in loading on the bottom and sides of the trench with respect to the slurry that is displaced. As a result, large local deformations may occur in very soft clay, and there may even be a chance of a bearing capacity type failure of the bottom or sides of

the excavation during concreting. Lateral deformations, settlements and pore pressures (if possible), therefore, should be monitored closely during concreting of the first panel.

(8) *Cast-in-place diaphragm walls can be instrumented for performance measurements.* Embedment strain gauges or similar instruments can be attached to the reinforcing steel cage before it is lowered into the trench. If the completed wall is to be used as an earth retaining structure, inclinometer casings for measurements of horizontal displacements can be installed prior to concreting. Furthermore, special procedures have been developed for the installation of transducers flush with the surface of the final wall for subsequent monitoring of earth pressures and pore pressures on the completed wall (Fig. 10.24).

Uff (1970) and DiBiagio & Myrvoll (1972) have described projects involving the instrumentation of slurry trench excavations.

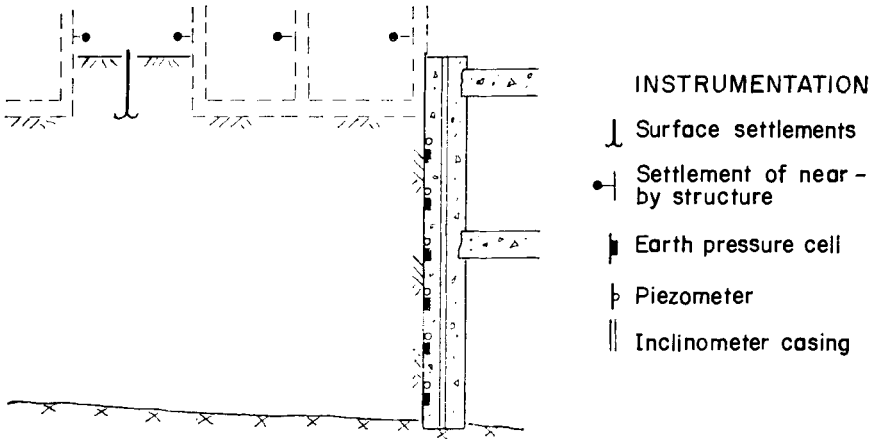

Fig. 10.24. Typical instrumentation for the post-construction performance of a diaphragm wall.

10.5 SUMMARY OF PRESENT INSTRUMENTATION CAPABILITY FOR SOFT CLAY

In the eight examples given in Section 10.4 to illustrate contemporary instrumentation concepts and procedures for projects involving soft clay, it is readily seen that a number of fundamental measurements are common to many of the cases presented.

The present state-of-the-art concerning the use of field instrumentation in soft clay engineering can be summarized thus:

(1) Ground settlements are relatively easily measured.
(2) Bottom heave measurements are difficult.
(3) Horizontal displacements can be measured.
(4) Ground water-table and pore pressure measurements are routine.
(5) Strut load measurements are reliable.
(6) Strains in structures can be measured with various degrees of confidence.
(7) Earth pressure distributions on structures can be measured.

It is fairly easy to measure ground settlements. Surface settlements are generally measured by optical levelling. Tell-tale rod installations can be used for optical levelling or mechanical measurements of deep settlements. Subsurface settlements can also be determined using extensometers based on a variety of mechanical, magnetic, inductive and hydraulic sensors that detect the positions of devices implanted in the ground, generally in a borehole.

Difficulty is experienced with bottom heave measurements. In principle, the heave of the bottom of an excavation can be determined by means of the same borehole techniques used to measure deep settlements. In practice, however, heave data is difficult to obtain, primarily because it is almost impossible to prevent damage to the installations during excavation of the soil surrounding or covering the burried instrumentation. In some instances, the measurements can be quite misleading because the heave instrumentation is unknowingly accidentally displaced by excavating machinery.

Horizontal displacements can be measured. Borehole inclinometers that are used to survey special casings mounted on retaining walls, piles and structures, or installed in boreholes, are now almost universally used for monitoring lateral deformations of the ground and horizontal displacements of structures. Numerous commercial instruments are available that have the resolution and repeatability needed for these observations, but measurements are time-consuming and require a considerable amount of computation. In some cases, fixed inclinometer installations are preferred because the angular displacement of a point in the ground or on a structure can be continuously monitored.

Ground water-table and pore pressure measurements are routine matters. Open stand-pipes can be used to monitor variations in ground water levels. Inflow of surface water must be prevented, otherwise serious errors in the interpretation of the data may occur. The measurement of pore water pressures in the ground is now a relatively straightforward task, since there are available many reliable commercial piezometers of varying cost and complexity, ranging from simple hydraulic stand-pipes to sophisticated electronic sensors that can be recalibrated in the ground. Although pore pressure can be measured with success, we do not always know where to measure it or at how many points, and this is sometimes the most difficult aspect of the problem. The different numbers and locations of the piezometers installed in 1957 and 1971 respectively behind the two open cuts illustrated

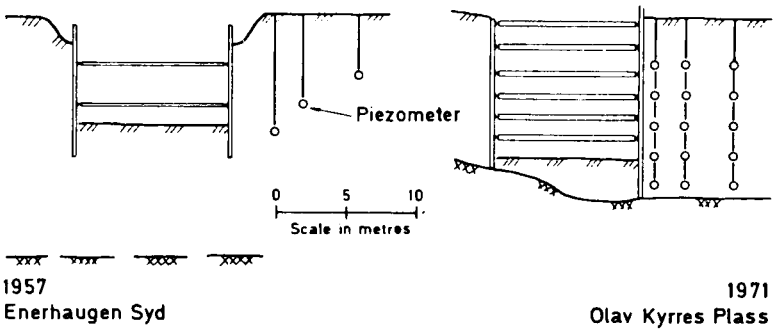

Fig. 10.25. Piezometer installations in 1952 and 1971 that illustrate changes in scope of instrumentation (DiBiagio, 1975).

in Fig. 10.25 (DiBiagio, 1975) illustrates the general trend that has evolved as a result of practical experience and a better appreciation of the complex role of pore pressure.

Strut loads, anchor loads and similar measurements can be determined with a high degree of confidence using several types of strain gauge or calibrated load cell, or by determining the force required to free a loaded strut or anchor by means of an auxiliary calibrated jack. If strain gauges are used, it is generally advisable to install more than one gauge per strut.

Surface strains in most structures can be measured with sufficient accuracy using mechanical or electrical strain gauges of various types. Although most steel structures present no serious problem, the interpretation of data may be difficult for some structures; for example, sheet-pile walls, because of high local stresses developed during pile driving. Strain gauging of concrete structures may be troublesome, even though the same general techniques used for steel structures can be used to monitor strains on the surfaces of concrete members. The conversion from strain to stress or load is much more uncertain for concrete, and the stresses and strains developed in concrete during hardening and curing often complicate the design of instrumentation and the interpretation of results.

Internal strains in concrete are without doubt more difficult to measure than surface strains. Embedment strain gauges, stress meters and instrumented reinforcement are generally used for this purpose. Cast-in-situ concrete piles are relatively easy to instrument for strain measurements, but strain-gauged precast concrete piles that are driven into the ground are difficult to deal with because the strain readings may be greatly influenced by cracks that develop in the concrete during driving.

Earth pressure distributions on structures can be measured. These specialized measurements can be made with considerable success in soft clay by means of pressure transducers, provided that the instruments satisfy basic criteria and are installed properly. The cost is high, however, because of the large number of sensors required to give a reliable picture of the pressure

distribution on most structures. Earth pressure measurements on a plane surface do not present serious difficulty, but observations on an irregular surface often fall into the 'difficult-to-interpret' category. Observations of earth pressures on sheet-pile walls fall into the latter group. The instrumentation problem presented by a sheet-pile wall results from the fact that, because of the corrugations in the wall, transducers for monitoring pressure must be placed on either the inside or outside of the corrugations (Fig. 10.26). In Fig. 10.26, actual sets of measurements are shown for two pressure gauges mounted on the same pile at the same depth in a soft clay. Typically, p_1 and p_2 are equal immediately after pile driving (as shown in the figure) but, because of consolidation, arching or movements of the sheeting, they seldom remain equal.

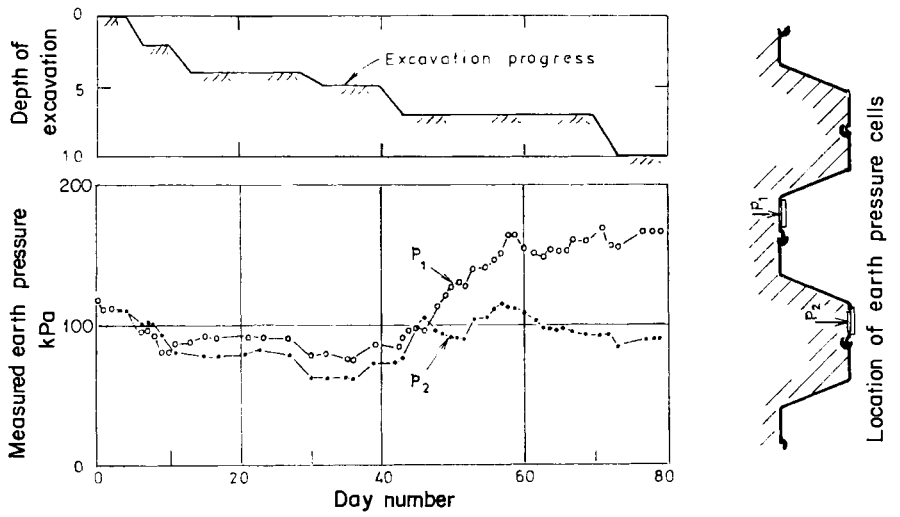

Fig. 10.26. Variation in earth pressure measurements on a sheet-pile wall (DiBiagio, 1975)

Because of uncertainties in the design and construction of many geotechnical engineering projects in soft clay, field instrumentation has been used significantly as an aid in the solution of these geotechnical problems for many years. Neither is the reliance on instrumentation likely to lessen in the future, because there is little chance that the theories, design procedures and construction practices will ever become so reliable and foolproof that field instrumentation will no longer be needed. Instrumentation is, and always will be, one of the most important implements in the geotechnical tool box.

With the advent of large digital computers together with the development of numerical techniques like the finite element method, there was an initial feeling that a de-emphasis of field instrumentation might occur. In reality, the opposite has happened, and instrumentation is today far more important

as a supplement to analytical methods than it has been in the past. With field instrumentation, verification is possible of these more advanced analytical predictions, and therein lies the promise of real improvements in the state-of-the-art in geotechnical engineering.

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Chapter 11

SEISMIC PROBLEMS OF SOFT CLAY DEPOSITS

by

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Mr Iwazaki has been particularly active in the use of observational techniques in geotechnical engineering and earthquake engineering. As a Special Technical Member of the Earthquake Disaster Prevention Committee of Osaka, he has worked for some time on micro-zonation in the Osaka area from both seismic and geotechnical viewpoints.

Chapter 11

SEISMIC PROBLEMS OF SOFT CLAY DEPOSITS

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11.1 INTRODUCTION

Soil dynamics has developed from research work on seismic exploration, machine foundations, highway design and earthquake problems. The dynamic stress-strain characteristics of soils are quite different from those which are found to apply under static loading. For the purposes of design for dynamic loading, it is necessary to ascertain the dynamic stress-strain characteristics of the foundation soil. The two most important soil parameters which describe their dynamic behaviour are the shear modulus, G, and the damping ratio, D, and it is necessary to determine values of these by means of laboratory or field tests. Constitutive relationships have been developed to describe the generalized behaviour of soils under dynamic loading, the behaviour for a particular soil being controlled by its shear modulus and damping ratio.

One of the first international meetings dealing with soil dynamics was the *International Symposium on Wave Propagation and Dynamic Properties of Earth Materials* held at the University of New Mexico in 1967, during which a wide range of topics was presented; some of the papers dealt with the seismic behaviour of soft clays. During the *Seventh International Conference on Soil Mechanics and Foundation Engineering* in Mexico City in 1969, a *Specialty Session on Soil Dynamics* was organised, and state-of-the-art papers on the effect of local soil conditions upon earthquake damage were prepared by Ohsaki (1969) and Seed (1969). Ohsaki reviewed the structural damage from past earthquakes in relation to soil conditions, and he pointed out the association of greater damage with soft ground and the effects of interaction between structure and ground. Seed presented a general review of structural damage intensity as a function of soil conditions and tried to interpret the damage potential in terms of response spectra. He introduced some methods for obtaining the seismic response of soil layers that account for the strain-dependent characteristics of soft soil, and he discussed soil settlements, liquefaction and slope instability.

Ishihara (1971) presented a general report on soil dynamics with special emphasis on the importance of strain levels and the various factors which affect the dynamic properties of soils. For the *International Symposium on Microzonation,* held in Seattle in 1972, several papers were produced illustrating the significance of soft soil in seismic zoning. In the *Specialty Session on Soil Dynamics and Seismic Effects on Foundations* at the *Eighth International Conference on Soil Mechanics and Foundation Engineering* in Moscow, Prakash (1973) presented general topics on the dynamic behaviour of soils. Finally, during the *Ninth International Conference on Soil Mechanics and Foundation Engineering* in Tokyo, a Main Session was devoted to soil dynamics, and an excellent state-of-the-art report was delivered by Yoshimi et al (1977).

In the first part of this chapter, the dynamic stress-strain relations for clayey soils are dealt with. In particular, the simple and practical expressions developed recently by the Senior Author for the dynamic behaviour of clays are given. In the second part, the seismic behaviour of soft clay ground is discussed with emphasis on the relation between input motion and soft ground response; this is controlled by the amplification due to multiple reflection and by the damping effect of the soft soil layers. Damage to residential structures, structural foundations and earth structures are briefly reviewed, and the recent methods of analysis are summarized.

11.2 DYNAMIC STRESS-STRAIN RELATIONSHIPS FOR CLAYEY SOILS

11.2.1 Stress-strain models

Many attempts have been made so far to establish analytical expressions for the nonlinear stress-strain characteristics of soils under cyclic loading conditions with the intention of modelling nonlinear behaviour in the dynamic response of soil masses. Typical formulations which have been proposed are the bilinear model, the Ramberg-Osgood model and the Hardin-Drnevich model.

The equations proposed by Ramberg & Osgood (1943) were initially applied to structural problems and later to soils (Streeter et al, 1974; Anderson, 1974; Idriss et al, 1976). On the other hand, Hardin & Drnevich (1972a, 1972b) developed the basic relationships for the variation of shear modulus and damping with shear strain amplitude, based on a hyperbolic stress-strain law proposed by Kondner (1963). A comparison of these two models was made by Richart (1975) in the tenth Terzaghi Lecture; he showed that both the Hardin-Drnevich and the Ramberg-Osgood type stress-strain model could adequately describe experimental data. For more detailed information the reader is referred to Richart's paper.

In the first part of this section, the Hardin-Drnevich model is presented. It is a rather simple model for describing soil behaviour under repeated loading. Subsequently, three particular aspects of soil behaviour which are

important for the analysis of all soil dynamic problems are discussed, namely: (1) stress-strain relationships in terms of effective confining pressure, (2) shear modulus, and (3) damping ratio of clayey soils.

11.2.2 Hardin-Drnevich model

The Hardin-Drnevich model employs a hyperbolic shear stress-shear strain relationship as a 'backbone' or 'skeleton' curve for soil, as shown in Fig. 11.1. This is given by:

$$\frac{\tau}{\gamma} = \frac{G_{oi}\,\tau_{fi}}{\tau_{fi} + G_{oi}\,\gamma} \tag{11.1}$$

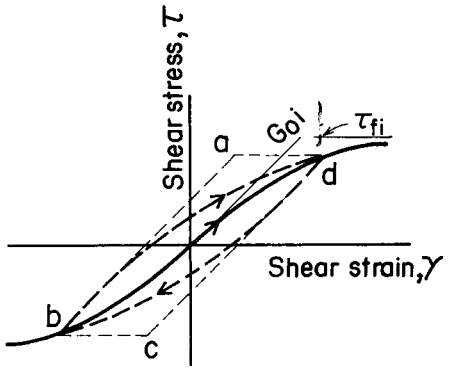

Fig. 11.1. Backbone curve and hysteresis loop for a dynamic shear stress-strain relationship

where τ is the shear stress, γ is the shear strain, G_{oi} is the initial or maximum shear modulus, and τ_{fi} is the shear stress at failure. Since τ/γ in Eq. 11.1 represents the secant shear modulus, G, for any stress or strain level, the shear modulus ratio, G/G_{oi} is then:

$$\frac{G}{G_{oi}} = \frac{1}{1 + \left(\frac{G_{oi}}{\tau_{fi}}\right)\gamma} = \frac{1}{1 + \frac{\gamma}{\gamma_r}} \tag{11.2}$$

where γ_r is called a *reference strain*. In this model, therefore, the influence of void ratio and mean principal effective stress can be expressed through their effect on the parameter γ_r.

The damping ratio of a soil can also be evaluated from the area of the parallelogram *abcd* and the area of the hysteresis loop in Fig. 11.1 by a correction factor, D_f. This correction factor is termed *damping ratio* at very large strains. The simple relationship between the shear modulus, G, and the damping ratio, D, is then:

$$D = D_f \left(1 - \frac{G}{G_{oi}}\right) \tag{11.3}$$

By substitution of Eq. 11.2 into Eq. 11.3, the relationship which describes the damping ratio as a function of strain level is obtained as:

$$\frac{D}{D_f} = \frac{\gamma/\gamma_r}{1+\gamma/\gamma_r} \tag{11.4}$$

The Hardin-Drnevich model, therefore, can fully describe the dynamic stress-strain relationships of soils if the three parameters G_{oi} and τ_{fi} (or γ_r) for the 'skeleton' curve and D_f for the hysteresis loop are given. Comparisons of the model with experimental data, however, are not always satisfactory, and a refinement in the evaluation of test data can be obtained by distorting the strain scale to produce a *hyperbolic strain*, γ_h, defined by:

$$\gamma_h = \frac{\gamma}{\gamma_r}\left\{1 + a \exp\left[-b\left(\frac{\gamma}{\gamma_r}\right)\right]\right\} \tag{11.5}$$

where a and b are empirical quantities which depend on such factors as number of cycles and frequency of load repetitions.

The shear strain and reference strain in Eq. 11.2 can be divided by the square root of the mean principal effective stress, σ_o', to give:

$$\frac{G}{G_{oi}} = \frac{1}{1+\left(\frac{\sqrt{\sigma_o'}}{\gamma_r}\right)\left(\frac{\gamma}{\sqrt{\sigma_o'}}\right)} \tag{11.6}$$

The relationship between G/G_{oi} and $\gamma/\sqrt{\sigma_o'}$ is shown schematically in Fig. 11.2. It should be noted that the value of $\gamma_r/\sqrt{\sigma_o'}$ corresponds to the value of $G/G_{oi}=0.5$ on the ordinate. For sand, Shibata & Soelarno (1975) obtained:

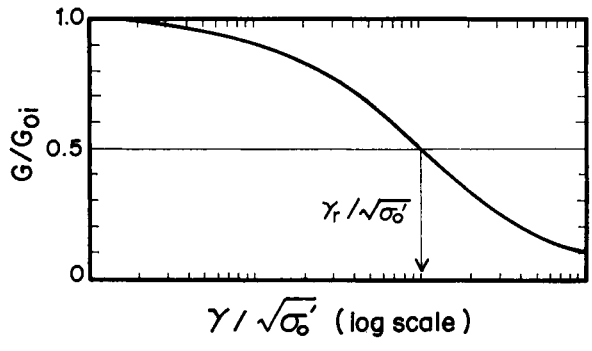

Fig. 11.2. Relationship between normalized shear modulus, G/G_{oi}, and the ratio $\gamma/\sqrt{\sigma'_o}$.

$$\gamma_r/\sqrt{\sigma'_o} = (1.0 \pm 0.3) \times 10^{-4}\ (\text{kPa})^{-\frac{1}{2}}$$

Hardin & Drnevich (1972a) obtained relationships in the laboratory between G/G_{oi} and γ for seven different cohesive soils consolidated under various mean effective principal stresses, σ'_o, ranging from 10 to 700 kPa. Some of the results for Lich Creek silt are reproduced in Fig. 11.3a, where it can be seen that the value of G/G_{oi} is hardly affected by the confining pressure. If the data are plotted against $\gamma/\sqrt{\sigma'_o}$ instead of γ, the relationship becomes

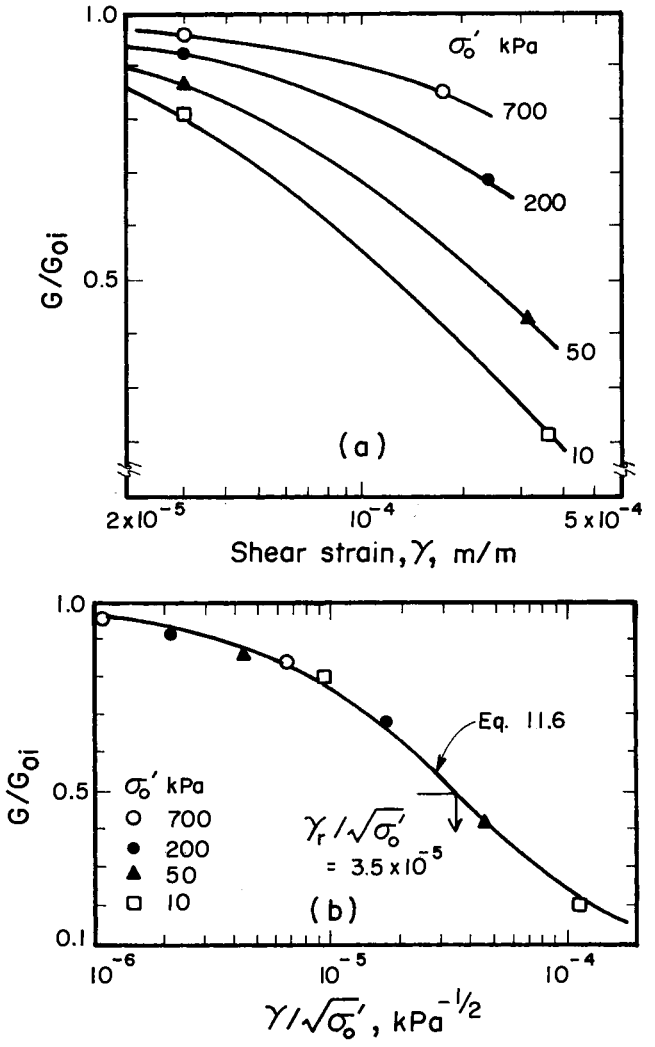

Fig. 11.3. Laboratory measurements of normalized shear modulus against (a) shear strain amplitude, and (b) ratio $\gamma/\sqrt{\sigma'_o}$ (data from Hardin & Drnevich, 1972a).

unique, as demonstrated in Fig. 11.3b. From this figure, the ratio $\gamma_r/\sqrt{\sigma'_0}$ for Lick Creek silt is obtained as 3.5×10^{-5} (kPa)$^{-\frac{1}{2}}$.

The values of $\gamma_r/\sqrt{\sigma'_0}$ for various cohesive soils have been estimated by the Authors from data reported by several investigators. These values, however, have been found to spread over a much wider range than those for sands. The results are summarized in Fig. 11.4 and in Table 11.1. The limited amount of data in the figure indicates an increase in $\gamma_r/\sqrt{\sigma'_0}$ with increasing void ratio. More data needs to be compiled and compared in order to arrive at some definite conclusions.

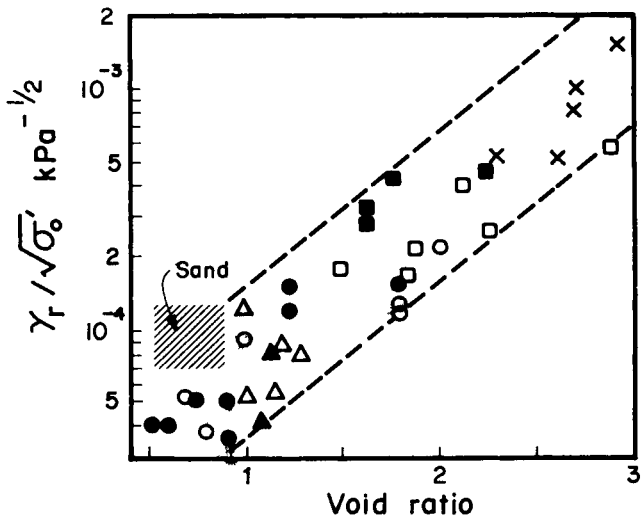

Fig. 11.4. Relationship between the ratio $\gamma_r/\sqrt{\sigma'_0}$ and void ratio for clay soils as measured by the investigators listed in Table 11.1.

Table 11.1 Definitions of symbols used in Fig. 11.4

Symbol	Soil tested	Reference
●	Undisturbed soils, I_p=7–52	Hardin & Drnevich (1972a)
×	Undisturbed soils, I_p=23–45	Hara (1973)
○	Artificial and undisturbed soils, I_p=19–64	Anderson (1974)
□	Undisturbed soils, I_p=22–59	Ishihara et al (1975, 1976)
△	Undisturbed soils, I_p=12–31	Iwasaki et al (1977)
■	Undisturbed soils, I_p=22–38	Suzuki & Sugimoto (1977)
▲	Undisturbed soils, I_p=24 and 37	Tanaka et al (1977)
▨	Sandy soils	Shibata & Soelarno (1975)

11.2.3 New stress-strain models

Test results of cyclic loading on soft clays indicate a reduction in soil stiffness with number of cycles. This effect is associated with an increase in pore pressure, i.e. a decrease in effective confining pressure, and also with a deterioration of the clay structure due to remoulding (Idriss et al, 1976). In this section, a model for the cyclic loading of clays is developed in terms of shear stress, shear strain and effective confining pressure (mean principal effective stress). In the last part of the section, the soil gradation model proposed by Idriss et al (1976) is reviewed.

11.2.3.1 Degradation of backbone curve

In a cyclic loading test with constant strain amplitude (Fig. 11.5a), the effective confining pressure, σ'_o, gradually decreases with increasing number of cycles from an initial value σ'_o to a value σ'_{oN} after N cycles, as shown

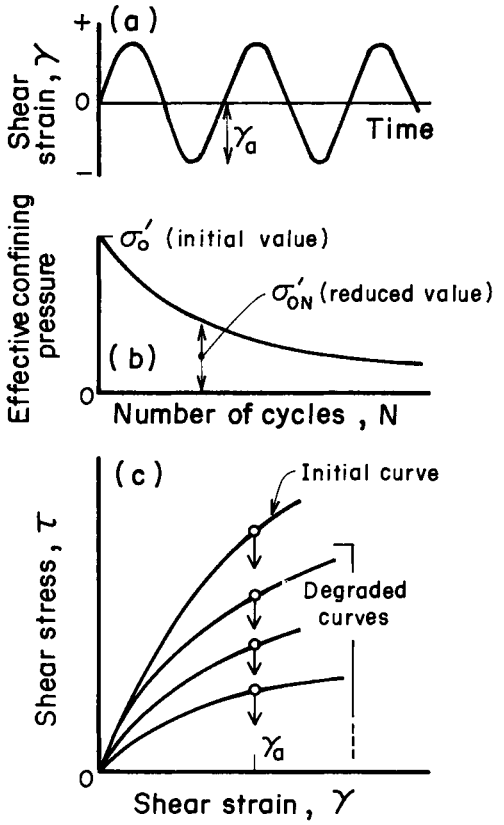

Fig. 11.5. Degradation of a stress-strain curve under cyclic loading: (a) cyclic loading at constant strain amplitude, (b) decrease in effective confining pressure with number of cycles, (c) degradation of backbone curve.

schematically in Fig. 11.5b. As a result of this, the backbone curve *degrades* and assumes a position lower than the initial curve (Fig. 11.5c). This figure also illustrates the stress changes during a test with a constant strain amplitude; the shear stress required to produce a certain strain amplitude γ_a will decrease with increasing number of cycles, as indicated by the arrows.

In Section 11.2.1, the initial backbone curve was described by Eq. 11.1. The degraded curves may be expressed by:

$$\frac{\tau}{\gamma} = \frac{G_o\,\tau_f}{\tau_f + G_o\,\gamma} \tag{11.7a}$$

which is identical to Eq. 11.1 if the two variables G_o and τ_f at $\sigma'_{oN} = \sigma'_o$ are denoted by G_{oi} and τ_{fi} respectively. Invertion of both sides of Eq. 11.7a gives the equation for the degraded curves as:

$$\frac{\gamma}{\tau} = \frac{1}{G_o} + \frac{\gamma}{\tau_f} \tag{11.7b}$$

In the following section, changes in G_o and τ_f with respect to the mean principal effective stress (or confining pressure) will be considered.

11.2.3.2 Shear modulus and shear strength based on effective stress

Hardin & Black (1969) measured the initial shear modulus, G_{oi}, of various undisturbed cohesive soils for normally consolidated and overconsolidated conditions, as shown in Fig. 11.6. In this figure, the values of G_{oi} are normalized with respect to a void ratio function, $F(e)$, in order to consider the effect of σ'_o independently of void ratio. The dashed lines in the figure define approximately the overconsolidated range, and the solid lines define the normally consolidated range. It can be seen that, in the normally consolidated range, the value of G_{oi} varies with $\sqrt{\sigma'_o}$. In the overconsolidated range,

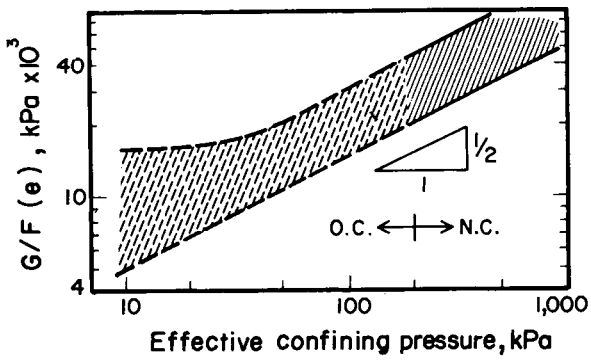

Fig. 11.6. Effect of overconsolidation ratio on the initial shear modulus of clay (Hardin & Black, 1969).

however, G_{oi} varies with different powers of σ'_o which range approximately between 0 and 0.5.

On the basis of Fig. 11.6, Hardin & Black (1969) proposed;

$$G_o = A\,F(e)\,\sqrt{\sigma'_o}\left(\frac{\sigma'_c}{\sigma'_o}\right)^m \qquad (11.8)$$

$$F(e) = \frac{(2.97-e)^2}{1+e}$$

where A is an empirical quantity (e.g. A=1500 to 3300 (kPa)$^{\frac{1}{2}}$ in the case shown in Fig. 11.6), and the exponent m varies between 0 and 0.5 depending on the plasticity index of the soil. The quantity σ'_c is the preconsolidation mean principal stress and σ'_c/σ'_o is the overconsolidation ratio.

Under undrained cyclic loading conditions (i.e. void ratio, e=constant), for which σ'_o and σ'_{oN} denote respectively the initial and reduced values of the confining pressure, σ'_c and σ'_o in Eq. 11.8 correspond to the quantities σ'_o and σ'_{oN}, and Eq. 11.8 becomes:

$$\frac{G_o}{\sqrt{\sigma'_{oN}}} = a\left(\frac{\sigma'_o}{\sigma'_{oN}}\right)^m \qquad (11.9)$$

For the normally consolidated case, $\sigma'_{oN}=\sigma'_o$ and $G_o=G_{oi}$, and Eq. 11.8 yields:

$$G_{oi} = A\,F(e)\,\sqrt{\sigma'_o} = a\sqrt{\sigma'_o} \qquad (11.10)$$

If it is assumed that m=0 for clays of low plasticity, Eq. 11.9 may further be modified to:

$$a = \frac{G_o}{\sqrt{\sigma'_{oN}}} = \frac{G_{oi}}{\sqrt{\sigma'_o}} \qquad (11.11)$$

In order to understand the role of the shear strength, τ_f, in Eq. 11.7, it is necessary to examine how the ratio τ_f/σ'_o is related to the overconsolidation ratio, σ'_o/σ'_{oN}. It is assumed here that the relationship between these is a function of the plasticity of the soil, such that:

$$\frac{\tau_f}{\sigma'_o} = \beta\left(\frac{\sigma'_{oN}}{\sigma'_o}\right)^{\bar{n}} \qquad (11.12)$$

where $\beta=\tau_{fi}/\sigma'_o$, and the exponent $\bar{n}$ varies between 0 and 1.0 depending on the plasticity index of the soil. On the assumption that $\bar{n}$=1 for clays of low plasticity, β is given by:

$$\beta = \frac{\tau_f}{\sigma'_{oN}} = \frac{\tau_{fi}}{\sigma'_o} \qquad (11.13)$$

11.2.3.3 Relationship between shear stress, shear strain and effective confining pressure

If both sides of Eq. 11.7b are multiplied by σ'_{oN} and rearranged, then:

$$\frac{\sigma'_{oN}}{\tau} = \left(\frac{\sqrt{\sigma'_{oN}}}{G_o}\right)\left(\frac{\sqrt{\sigma'_{oN}}}{\gamma}\right) + \frac{\sigma'_{oN}}{\tau_f} \tag{11.14}$$

Substitution for α and β from Eqs. 11.11 & 11.13 gives:

$$\frac{\sigma'_{oN}}{\tau} = \frac{1}{\alpha}\frac{\sqrt{\sigma'_{oN}}}{\gamma} + \frac{1}{\beta} \tag{11.15}$$

Equation 11.15 indicates the relationship between τ, γ and σ'_o and may be compared with Eq. 11.1 or Eqs. 11.7a & 11.7b. The correspondence between the terms in these equations is as follows:

Eq. 11.1 or Eq. 11.7	*Eq. 11.15*
τ	τ/σ'_{oN}
γ	$\gamma/\sqrt{\sigma'_{oN}}$
G_{oi} or G_o	α
τ_{fi} or τ_f	β

The validity of the simple relationship expressed in Eq. 11.15 will now be discussed by applying it to experimental results. Taylor & Bacchus (1969) reported results of strain-controlled cyclic triaxial tests on clay with pore pressure measurement. Figure 11.7a, which is reproduced from the results of one of these tests, shows the shear stress-strain relationships for normally consolidated samples subjected to 30 cycles. It can be seen from this figure that the stress-strain relationship is not uniquely determined, but the shear stress decreases with increasing number of cycles.

The data from Fig. 11.7a are replotted in Fig. 11.7b as σ'_{oN}/τ versus $\sqrt{\sigma'_{oN}}/\gamma$ to verify the proposed Eq. 11.15. Although the data are scattered, they indicate a linear relationship between σ'_{oN}/γ and $\sqrt{\sigma'_{oN}}/\gamma$, as postulated by Eq. 11.15.

11.2.3.4 Soil gradation model proposed by Idriss et al (1976)

A new stress-strain model was developed by Idriss et al (1976) in order to account for the reduction in shear modulus in response analysis. The development of this model was based on the results of a series of cyclic tests on soft San Francisco Bay Mud. For their backbone curve, they employed the Ramberg-Osgood equation:

$$\gamma = \gamma_y\left[\frac{\tau}{G_{oi}\,\gamma_y} + \zeta\left(\frac{\tau}{G_{oi}\,\gamma_y}\right)^R\right] \tag{11.16}$$

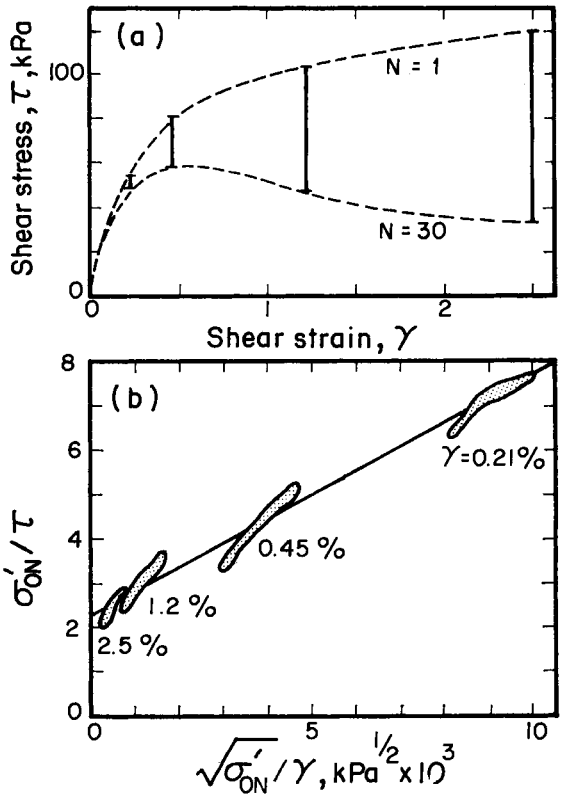

Fig. 11.7. Experimental verification of Eq. 11.15: (a) shear stress-strain curves for first and thirtieth cycle of loading on clay (data from Taylor & Bacchus, 1960), (b) linear relationship between σ'_{oN}/τ and $\sqrt{\sigma'_{oN}}/\gamma$.

where γ_y, ζ and R are soil parameters. For subsequent cycles, this equation was modified to:

$$\gamma = \gamma_y \left[\frac{\tau}{I_D \, G_{oi} \, \gamma_y} + \zeta \left(\frac{\tau}{I_D \, G_{oi} \, \gamma_y} \right)^R \right] \tag{11.17}$$

where I_D is the value of the degradation index at each strain level.

In order to verify the proposed model, the cyclic tests on San Francisco Bay Mud were carried out for several hundred cycles with various values of cyclic strain. Figure 11.8 shows a comparison between the measured shear stresses and those predicted by the proposed model. In this test, a cyclic shear strain, γ, of $\pm$ 3% was applied during the first 100 cycles, 1.5% for the next 100 cycles, etc, until 600 cycles had been completed (see Fig. 11.8a). The cyclic shear stress, τ, was measured at different numbers of cycles, and the results are shown in Fig. 11.8b for comparison with the values predicted by

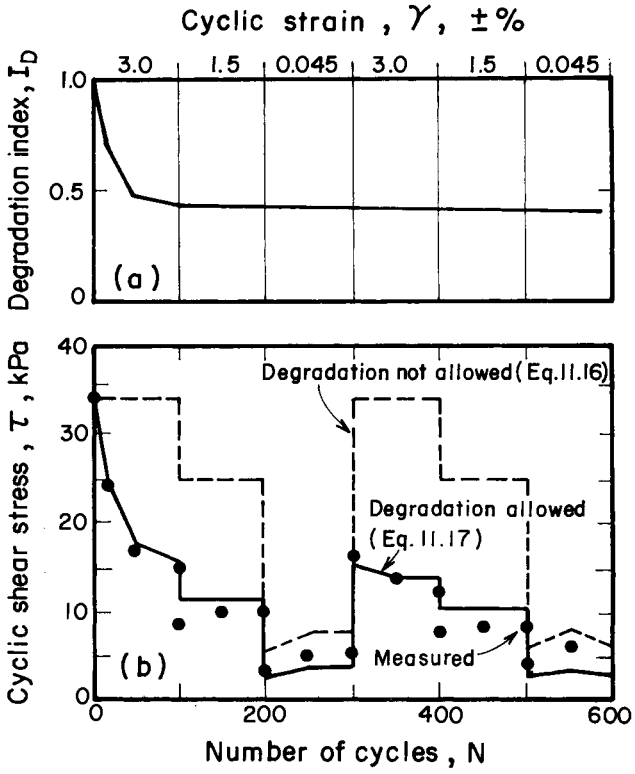

Fig. 11.8. Comparison of measured and predicted shear stresses during cyclic triaxial tests on San Francisco Bay Mud: (a) variation of degradation index with shear strain and number of cycles, (b) vertification of Eq. 11.17 (Idriss et al, 1976).

means of Eq. 11.17 using the parameters $\gamma_y = 0.03\%$, $G_{oi}\,\gamma_y = 7$ kPa, $\zeta = 0.2$ and $R=4$. The effect of degradation is very pronounced in this particular test, with I_D dropping to a value of 0.4 after 600 cycles. The agreement between measured and predicted shear stresses is very good.

Having confirmed the validity of their soil gradation model for transient cyclic loading, Idriss et al (1976) developed a computer program for nonlinear ground response analysis based on the new model.

11.2.4 Initial shear modulus and damping ratio

11.2.4.1 Initial shear modulus

The initial shear modulus of clayey soils, G_{oi} (or $\alpha = G_{oi}/\sqrt{\sigma'_o}$), together with the quantity β, is the key parameter which determines the stress-strain relationship expressed by Eq. 11.15. Several empirical equations have been proposed to express G_{oi} as a function of void ratio and mean principal effective stress, σ'_o, for normally consolidated clays. These are:

(1) Hardin & Black (1968):

$$G_{oi} = A\,F(e)\sqrt{\sigma'_o}\ \text{(kPa)} \tag{11.18a}$$

where A is approximately 3300 (kPa)$^{\frac{1}{2}}$ and $F(e)$ is given by Eq. 11.8.

(2) Humphries & Wahls (1968):

$$G_{oi} = 35{,}000\text{–}13{,}000\ e\ \text{(kPa)} \tag{11.18b}$$

valid for bentonite under a pressure of $\sigma'_o < 50$ kPa

(3) Marcuson & Wahls (1972):

$$G_{oi} = 450\,\frac{(4.4-e)^2}{1+e}\sqrt{\sigma'_o}\ \text{(kPa)} \tag{11.18c}$$

valid for bentonite

(4) Shibata et al (1970):

$$G_{oi} = 42{,}000\,(0.67-n^*)\sqrt{\sigma'_o}\ \text{(kPa)} \tag{11.18d}$$

where n^* is the porosity.

These equations, with the exception of Eq. 11.18b, indicate that the value of $\alpha = G_{oi}/\sqrt{\sigma'_o}$ may be expressed as a function of void ratio, e, or porosity, n^*. Relationships between α and e (or n^*), obtained from in-situ and laboratory tests, are shown in Figs. 11.9 & 11.10 respectively.

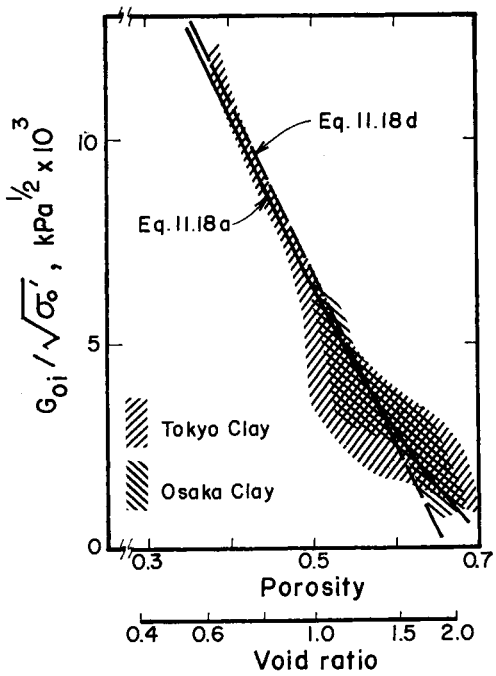

Fig. 11.9. Values of $G_{oi}/\sqrt{\sigma'_o}$ measured in-situ compared with predictions by Eq. 11.18a (Hardin & Black, 1968) and Eq. 11.18d (Shibata et al, 1970).

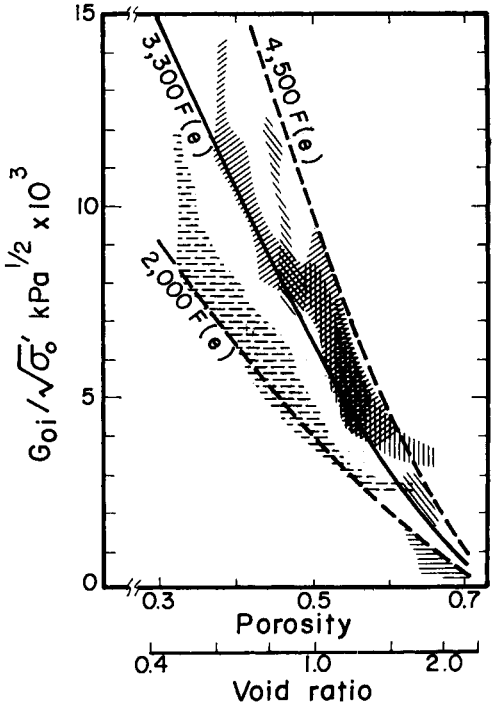

Fig. 11.10. Values of $G_{oi}/\sqrt{\sigma'_o}$ for clays in Table 11.2, measured by resonant column tests, compared with prediction by Eq. 11.18a (with A=2000, 3300 & 4500 $F(e)$ kPa$^{\frac{1}{2}}$).

Table 11.2. Definitions of symbols used in Fig. 11.10

Symbol	Soil tested	Reference
	Kaolinite and Boston Blue Clay	Hardin & Black (1968)
	Kaolinite and bentonite	Humphries & Wahls (1968) Marcuson & Wahls (1972)
	Various undisturbed cohesive soils	Hardin & Black (1969)
	Kaolinite	Afifi & Richart (1973)
	Sensitive clay	Yong et al (1977)

Figure 11.9 shows the results of in-situ measurements of shear wave velocity as well as the curves predicted by Eqs. 11.18a & 11.18d. Both Tokyo and Osaka Clays are of alluvial age and are considered to be in a normally consolidated state. The values of G_{oi} were calculated from the measured shear wave velocities and the unit weights of the clays, while the effective stresses, σ'_o, were estimated from the effective overburden pressures in the soil layer. From the comparison of test results and predicted values illustrated in Fig. 11.9, it appears that Eqs. 11.18a & 11.18d provide reasonable estimates for average in-situ α-values.

In Fig. 11.10 are summarized some published results of resonant column tests on various clays; this figure should be read in conjunction with Table 11.2. The three curves in the figure were calculated by Eq. 11.18a for values of A=2000, 3300 & 4500 (kPa)$^{\frac{1}{2}}$. The two dashed lines in this figure define approximately the upper and lower limits of the measured α-values; the lower limit of α, however, will increase with time during secondary consolidation, as shown by Afifi & Richart (1973). Equation 11.18a proposed by Hardin & Black (1968), therefore, defines the relationship between α and void ratio very well.

11.2.4.2 Initial shear modulus and undrained shear strength

Section 11.2.3.1 dealt with ways in which the initial shear modulus can be expressed as a function of void ratio (or porosity) and mean principal effective stress. The relationship between initial shear modulus, G_{oi}, and undrained shear strength, s_u, however, has also been studied by several investigators. Figure 11.11, for example, shows a relationship between G_{oi} obtained from in-situ measurements of shear wave velocity and s_u for a clay determined from

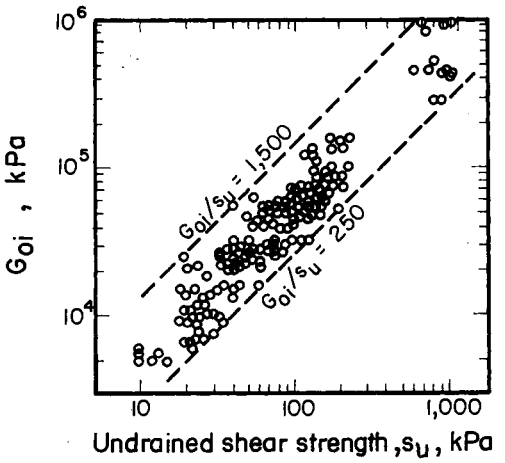

Fig. 11.11. Relationship between shear modulus and undrained shear strength of clay (data from Hara et al, 1974).

laboratory tests (Hara et al, 1974). Since the relationships plotted on a log-log scale correspond to lines with an almost 45° slope, the ratio G_{oi}/s_u may be assumed to be of constant value. This value falls within the range 250 to 1500 for the cases shown in Fig. 11.11.

In the following, some further development of Eq. 11.18a is presented by the incorporation of the ratio G_{oi}/s_u. It is well-known that, for clayey soils, s_u is governed primarily by the consolidation pressure, σ'_o, and a ratio K′ may then be defined as:

$$K' = s_u/\sigma'_o$$

The ratio K′ may be assumed to be about 0.3, even though K′ varies with the plasticity index and the method of testing. On the basis of Eq. 11.18a, G_{oi}/s_u may then be expressed as:

$$\frac{G_{oi}}{s_u} = \frac{A\,F(e)\,\sqrt{\sigma'_o}}{K'\,\sigma'_o} = \frac{A\,F(e)}{K'\,\sqrt{\sigma'_o}} \tag{11.19a}$$

which gives:

$$F(e) = \left(\frac{K'}{A}\right)\left(\frac{G_{oi}}{s_u}\right)\sqrt{\sigma'_o} = \frac{(2.97-e)^2}{1+e} \tag{11.19b}$$

On the assumption that the values of the parameters in Eq. 11.19b are $A=3300\,(\text{kPa})^{\frac{1}{2}}$, $K'=0.3$ and $G_{oi}/s_u=200, 500, 1000, 1500$ & 2000, relationships between void ratio, e, and consolidation pressure, σ'_o, have been calculated to be as shown in Fig. 11.12. The compression index, C_c, for each of these

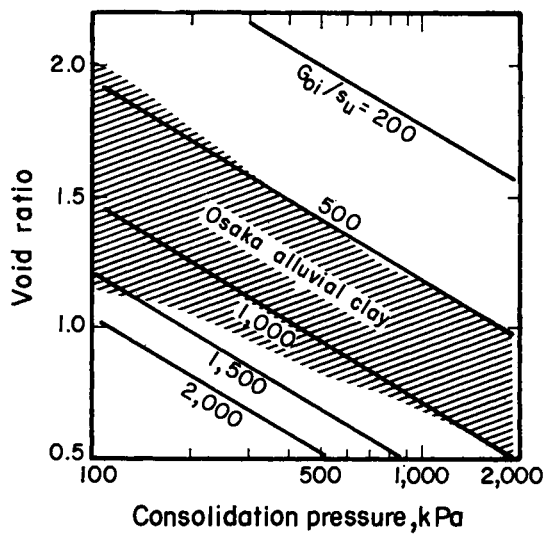

Fig. 11.12. Theoretical relationships between void ratio and consolidation pressure (Eq. 11.19b) and data on Osaka alluvial clay.

e-log σ_o' lines is 0.75, which implies that any clay with $C_c=0.75$ will have a constant G_{oi}/s_u ratio. It should be noted that the smaller G_{oi}/s_u ratios are associated with higher void ratios; this is in agreement with the experimental results that: (1) clays of high plasticity have larger void ratios than lower plasticity clays under the same consolidation pressure, and (2) higher plasticity clays have lower values of G_{oi}/s_u (Wilson & Dietrich, 1960). It can be seen from Fig. 11.12 that the ratio G_{oi}/s_u for Osaka alluvial clays varies between about 500 and 1500.

11.2.4.3 Damping ratio

The damping ratio, D, in the Hardin-Drnevich model (Section 11.2.2) is expressed by Eq. 11.3 as:

$$D = D_f\left(1-\frac{G}{G_{oi}}\right)$$

This equation indicates that, if the values of D_f and G_{oi} are constant, the relationship between D and G for various strain levels should be linear. Figure 11.13 shows some relationships between damping ratio and shear modulus which have been obtained by different experimental methods on various types of clay by Idriss (1966) (free vibration test), Thiers & Seed (1968) (simple shear test) and Donovan (1969) (triaxial compression test); the shear moduli have been normalized with respect to s_u. The results are scattered, but they show the tendency towards a linear relationship between D and G, as postulated by Eq. 11.3.

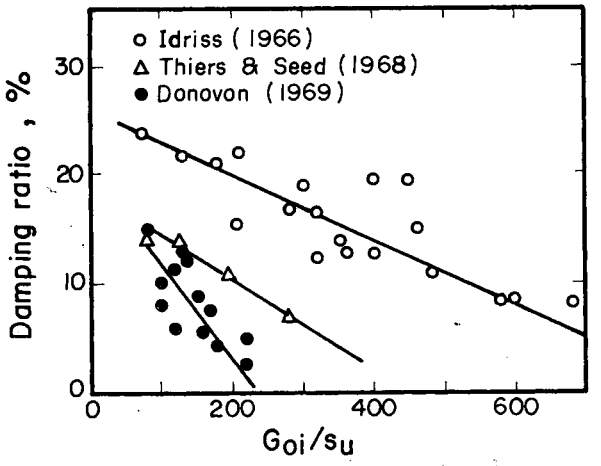

Fig. 11.13. Measured relationships between damping ratio and G_{oi}/s_u.

Substitution of Eq. 11.6 into Eq. 11.3 yields:

$$D = \frac{D_f}{1 + \left(\frac{\gamma_r}{\sqrt{\sigma_o'}}\right)\left(\frac{\sqrt{\sigma_o'}}{\gamma}\right)} \tag{11.20}$$

In this equation, D_f may be assumed to be about 0.15 to 0.20 for clayey soils according to the values of damping ratio which correspond to strain levels of more than 10^{-2} (Seed & Idriss, 1970). The ratio $\gamma_r/\sqrt{\sigma_o'}$ may assume a value within the rather wide range of 0.3 to 2.0×10^{-4} $(\text{kPa})^{-\frac{1}{2}}$, as shown in Fig. 11.14.

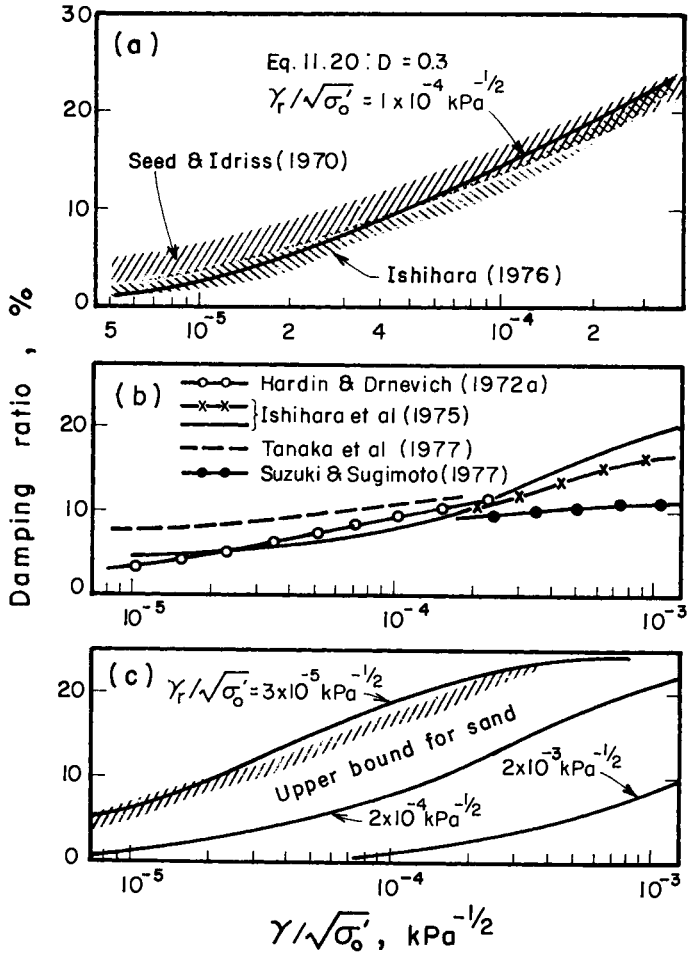

Fig. 11.14. Measured damping ratios and predictions by Eq. 11.20: (a) for sandy soils, (b) for clay soils, (c) theoretical curves by Eq. 11.20.

Variations of the damping ratio, D, with the ratio $\gamma_r/\sqrt{\sigma_0'}$ are shown in Fig. 11.14. Data obtained by Ishihara (1976) and that collected by Seed & Idriss (1970) for sandy soils are plotted on Fig. 11.14a together with the relationship determined from Eq. 11.20 with $D_f=0.3$ and $\gamma_r\sqrt{\sigma_0'}=1.0\times10^{-4}$ $(\text{kPa})^{-\frac{1}{2}}$ for sand (see Fig. 11.4); the computed curve is in good agreement with the experimental results. The data from the five published investigations on clays are plotted on Fig. 11.14b; they fall in a narrow band over a wide range of $\gamma_r/\sqrt{\sigma_0'}$, a fact which is surprising since the value of $\gamma_r/\sqrt{\sigma_0'}$, unlike that for sands, varies widely. The theoretical relationships of D vs $\gamma_r/\sqrt{\sigma_0'}$, as calculated from Eq. 11.20, have been plotted in Fig. 11.14c for $D_f=0.25$ and $\gamma_r/\sqrt{\sigma_0'}=3\times10^{-5}$, 2×10^{-4} & 2×10^{-3} $(\text{kPa})^{-\frac{1}{2}}$, which covers the range of values for clays. It can be seen that, unlike the experimental data, the theoretical damping ratios vary greatly, and it must be concluded that Eq. 11.20 does not predict well the damping in clay soils.

11.3 SEISMIC BEHAVIOUR OF SOFT CLAY DEPOSITS

11.3.1 General review

Past experience has shown that earthquake motions on the surface of soft deposits are several times stronger than those on hard ground. In the San Francisco earthquake of 18 April 1906 (magnitude $M=8.3$), severe damage to the downtown area, and particularly to structures on soft ground, was reported. On September 1923, the Kwanto earthquake ($M=7.9$) destroyed the Tokyo area (epicentral distance about 90 km), and an investigation into the failures of wooden houses revealed that the severity of the damage increased with the thickness of the soft alluvial soil layer. The Nankaido earthquake of 21 December 1941 ($M=8.1$) caused damage in the reclaimed soft clay area of Kojima Bay, Okayama, which was located as far as 250 km away from the epicentre. The Fukui earthquake of 28 June 1948 ($M=7.3$) produced severe damage in the Fukui area (epicentral distance less than 20 km). These are good examples of the high damage susceptibility of soft ground to earthquakes, and of the way in which the distribution of damage is closely influenced by the geological setting.

Much research has been conducted to correlate earthquake damage with ground characteristics. Sassa et al (1949) presented some relationships between the damage to wooden structures and the elastic wave velocity, determined by the in-situ refraction method at several sites which suffered damage during different earthquakes. Kanai et al (1954) proposed relating measurements of microtremors, which represent a significant characteristic of the ground, to earthquake damage, and Kanai et al (1965) showed how soil deposits could be classified into several types based on the frequency-period relationship obtained from microtremor observations. The Tokachioki earthquake of 16 May 1968 ($M=7.9$) resulted in extensive damage to embankments

and structures on soft soils in the northern part of Japan, and this resulted in intensive research work related to soft ground under earthquake conditions.

Attempts to obtain records of strong ground motions were begun by the US Coast & Geodetic Survey in 1932 and by the Japanese Strong Acceleration Committee in 1952. Hisada et al (1965) compared the response spectra of strong ground motion records obtained with various soil conditions and found that, for soft soil conditions, the period that corresponds to the peak acceleration response is increased by a few seconds.

Idriss & Seed (1968) introduced strain-dependent dynamic stress-strain characteristics into earthquake response analysis of multi-layer soil systems and tried to simulate earthquake motions at several sites in the USA and Japan. Seed et al (1970) presented an interpretation of the variation in structural damage due to different soil conditions by using the differences in response spectra as a measure of the interaction effect between soil and structure.

Several cases have been reported recently in which records of the seismic motion of soft soil deposits were obtained not only on the ground surface but also at various depths in the soil. Sugimura (1970), Yokoyama et al (1975), Ohta et al (1975) and Ishihara et al (1975) compared the results of such observations with motions computed by modelling the subsurface soils as either elastic, bilinear, visco-elastic or strain-dependent materials. Hakuno et al (1975) reported on the results of some simulations and discussed the time duration and maximum acceleration of the earthquake motion at the surface of a soft soil layer.

Most theoretical treatments of soil behaviour under seismic loading assume that shear waves travel upwards from the base rock. Asada (1975), however, reported on the possibility that surface waves might also contribute to damage. Tsuchida & Kurata (1975) reported on observations at Tokyo airport, where a thick alluvial clay layer is present, and discussed the relative movement which occurred simultaneously at several points on the ground surface.

11.3.2 Earthquake response of soft clay ground

Records of surface displacements on soft alluvium, hard diluvium and rock, as caused by a small local earthquake, are shown in Fig. 11.15 (Okano & Yamazaki, 1975). It can be seen that the earthquake motions on the soft ground tended to have larger amplitudes and longer durations than those on diluvium or rock. There are cases, however, where earthquake motions of soft soil layers do not produce larger amplitudes than those on hard ground.

11.3.2.1 Change of amplitude due to wave transmission between two elastic media

The transmission of a wave from one medium to another results in both transmitted and reflected waves, as illustrated in Fig. 11.16a. If the incident, reflected and transmitted amplitudes are denoted by A_i, A_r and A_t respec-

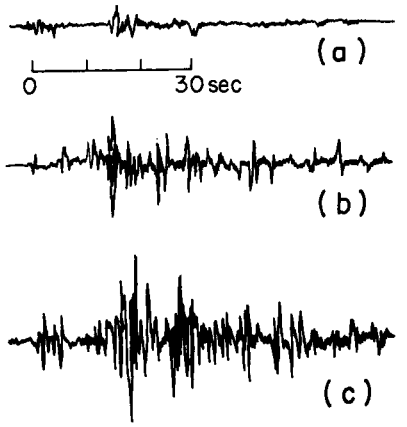

Fig. 11.15. Seismograph records for a small earthquake showing surface displacements on (a) rock, (b) hard diluvium, (c) soft alluvial clay (Okano & Yamazaki, 1975).

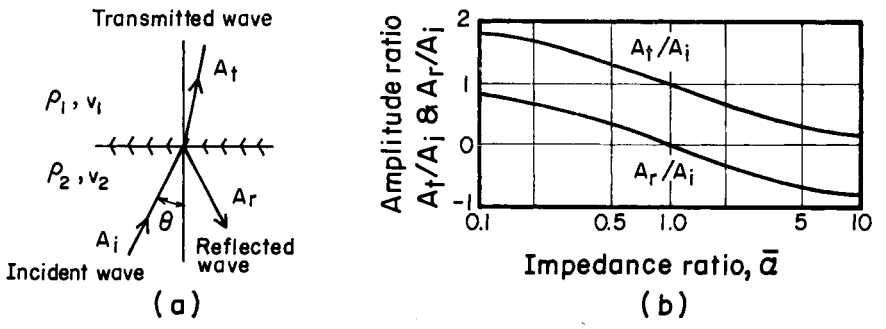

Fig. 11.16. Change in amplitude on transmission of wave from one medium to another: (a) wave amplitudes, (b) amplitude ratios.

tively, and the incident angle by θ, then in the simple case of $\theta=0$, the relationships among these amplitudes can be expressed as:

$$A_r = \frac{1-\bar{a}}{1+\bar{a}} A_i$$

$$A_t = \frac{2}{1+\bar{a}} A_i \tag{11.21}$$

$$\bar{a} = \frac{\rho_1 v_1}{\rho_2 v_2} \tag{11.22}$$

where $\bar{a}$ is the wave impedance ratio, ρ is the density and v is the wave velocity.

In general, the wave impedance of a soft layer is smaller than that of a hard layer, and the impedance ratio is always less than unity if the medium from which the wave leaves is harder than the medium it enters. In Fig. 11.16b, the amplitude ratio (amplification factor) between transmitted and incident wave, A_t/A_i, varies between 1.0 and 2.0, and this demonstrates that the existence of the soft layer can increase the amplitude of the transmitted wave by as much as twice that of the incident wave from the harder layer.

11.3.2.2 Amplification by multiple reflection

It has often been observed that the amplitudes of ground motion at the surface of a soft layer are several times larger than those on rock outcrops. Kanai et al (1953) identified this phenomenon of 'excessive amplification' as being caused by the multiple reflection of waves in the soft surface stratum. If a horizontally polarized shear wave travels upwards in a two-layer system, the wave transmitted into the soft surface layer is reflected at the ground surface and travels back to the lower boundary of the soft layer, where it will be reflected again. These multiple reflections between the two boundaries of the soft layer continue until the wave energy is either dissipated internally in the layer or transmitted partly outside the system.

If the incident wave motion is assumed to be harmonic and of sinusoidal form, the amplitude of motion at distance z' from the bottom of the soft layer is given by:

$$A_{(z')} = \frac{2\,A_i \cos\left[\frac{\pi}{2}\frac{T_g}{T_i}\left(1-\frac{z'}{h}\right)\right]}{\sqrt{\cos^2\left(\frac{\pi}{2}\frac{T_g}{T_i}\right) + \bar{\alpha}^2 \sin^2\left(\frac{\pi}{2}\frac{T_g}{T_i}\right)}} \tag{11.23}$$

where A_i is the amplitude of the incident wave, h is the thickness of the surface layer, T_i is the period of the incident wave, and T_g is the natural period of the surface layer ($T_g=4h/v_1$). Figure 11.17 shows Eq. 11.23 plotted as amplification factor, A/A_i, vs period ratio, T_g/T_i.

If the incident sine wave has a period of $T_g=T_i$, the amplitude at the surface ($z'=h$) assumes the maximum value (i.e. $A_{(z'=h)} = (2/\bar{\alpha})A_i$), and this condition represents the resonant state. It should be realized that the maximum amplification, which is $2/\bar{\alpha}$, is always larger than 2.0 for a soft surface layer with $\bar{\alpha}$ less than unity.

If the sinusoidal motion is not continuous but consists of a limited number of sine waves, the amplitude at the surface is less than $2/\bar{\alpha}$ and is given by (Sezewa & Kanai, 1932):

$$A_{(z'=h)} = \frac{2}{\bar{\alpha}}\left[1-\left(\frac{1-\bar{\alpha}}{1+\bar{\alpha}}\right)^n\right] \tag{11.24}$$

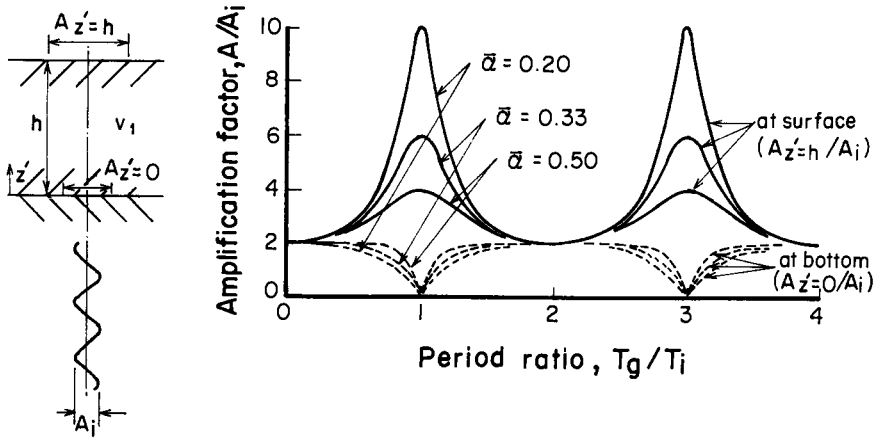

Fig. 11.17. Amplification factors at the surface and base of a soft clay layer for different wave impedance ratios, $\bar{\alpha}$, (Eq. 11.23).

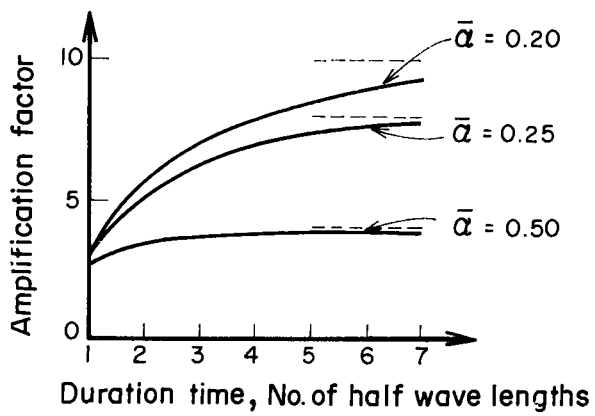

Fig. 11.18. Influence of number of half-wave lengths on the amplification factor at resonance for various wave impedances (Eq. 11.24).

where n is the number of continuous half-wave lengths. A graphical representation of Eq. 11.24 for various $\bar{\alpha}$ values is shown in Fig. 11.18, where it can be seen that a wave train of about 4 to 5 half-wave lengths is sufficient to cause almost the same amplitude amplification as is a train with an infinite number of waves.

Figure 11.19 shows the amplitude of ground surface motion caused by an upward travelling incident wave. This incident wave is of constant amplitude and has a period which is identical to the natural period of the surface layer (i.e. $T_i = T_g$). Its time duration is four half-wave lengths. It can be seen that the surface amplitude increases rapidly until the incident wave ceases to act, and then gradually decreases as the wave energy is transmitted into the bottom

layer through multiple reflection. The impedance ratio is assumed to be 0.25. An incident wave with a duration of only four half-wave lengths is able to generate a surface amplitude which is about 90% of that expected for the case of an incident wave of infinite duration. It should be noted, therefore, that rather short-term durations of incident waves containing only a few half-wave lengths may cause large amplitudes at the ground surface if the period of the incident wave coincides with the natural period of the surface layer.

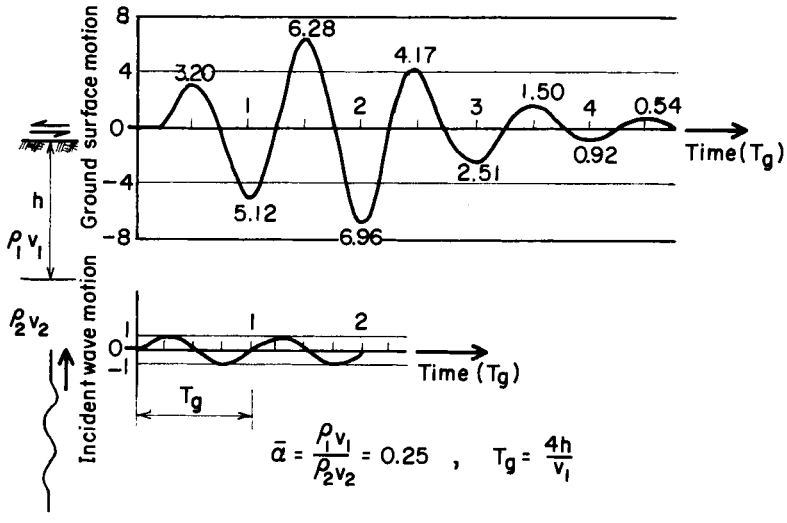

Fig. 11.19. Different wave amplifications at resonance at the ground surface for incident waves of equal amplitude but different durations.

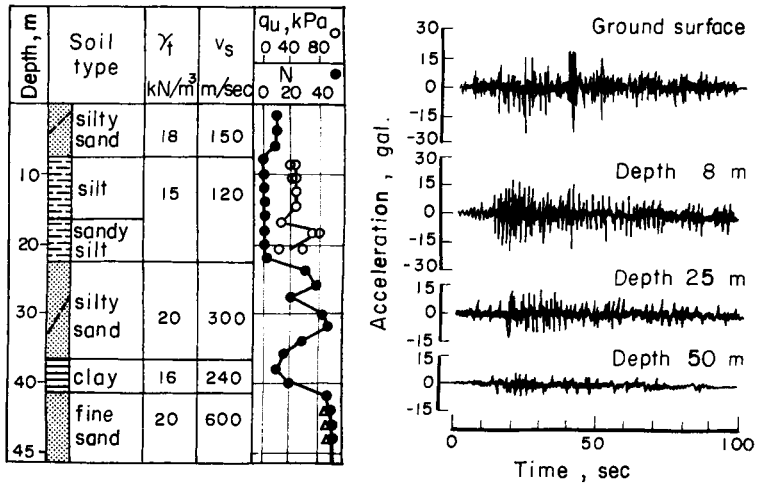

Fig. 11.20. Record of decreasing acceleration amplitudes with depth (Ishihara et al, 1975).

Figure 11.20 shows measured accelerogrammes at several depths in a soft silty clay layer about 20 m thick (Ishihara et al, 1975). The acceleration amplitude was maximum at the surface and decreased with depth. The surface amplitude was several times larger than that in the dense sand layer at a depth of 50 m. It should be noted from Fig. 11.17 that the amplitude at the bottom of the soft layer is theoretically zero when resonance occurs. Because the amplitude at the bottom of the soft layer, $A_{(z'=0)}$, can often be smaller than the amplitude of the incident wave, A_i, the amplification factor, $A/A_{(z'=0)}$, may be greater than that based on the amplitude of the incident wave, A/A_i.

It might be expected that the amplitude of the acceleration at the surface of a thick layer of soft clay would always be larger than for a thinner layer, but this is not necessarily so, as demonstrated by the acceleration records in Fig. 11.21 for two sites in Osaka for an earthquake of magnitude 5.9 and epicentral distance of about 90 km. The soft alluvial layers at the two sites were of different thicknesses, and it can be seen that the amplitude on the surface of the thick alluvial layer was not larger than the surface amplitude of the thin layer. This occurred because the incident wave had about the same natural period as the thinner layer.

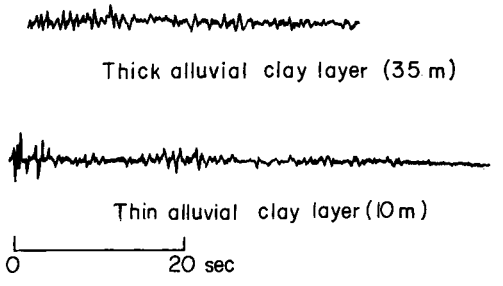

Fig. 11.21. Influence of layer thickness on ground response (accelerograms for two sites in Osaka).

11.3.2.3 Decrease in amplitude by damping

The amplitudes of waves travelling in a soil material decrease because of geometric and hysteretic damping. Damping can be expressed by the quality factor, Q, as:

$$Q = \frac{\text{total energy stored per one cycle}}{\text{energy loss per one cycle}} \times 2\pi \tag{11.25}$$

The damping ratio, D, and the logarithmic decrement, δ, are related to the quality factor, Q, by the equations:

$$\delta = ln\frac{A_1}{A_2} = \frac{\pi}{Q} = \frac{2\pi D}{\sqrt{1-D^2}} \approx 2\pi D \tag{11.26}$$

where A_1 and A_2 are the amplitudes of two successive waves. The decremental ratio of successive waves is defined as:

$$R_d = \exp(-\delta) = \frac{A_2}{A_1} \tag{11.27}$$

In Fig. 11.22, are shown the relationships between R_d, Q & D.

The effect of damping on the response of a surface layer hit by an incident wave with $T_i = T_g$ for a time duration of two wave lengths is depicted in Fig. 11.23 for the three damping ratios $D=0$, 5 & 20%. When $D=20\%$, the amplitude of the wave at the upper surface of the layer is only about one-half of the amplitude of an elastic layer with $D=0\%$.

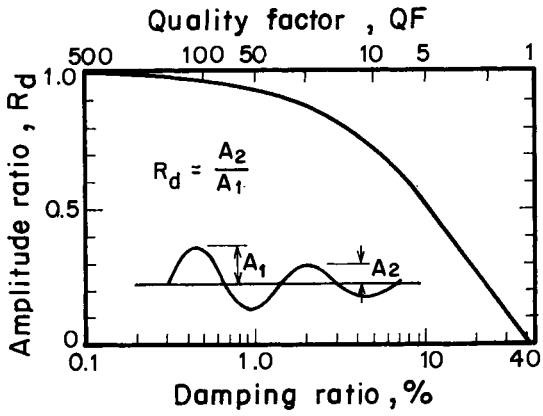

Fig. 11.22. Relationship between amplitude ratio, damping ratio and quality factor (Eq. 11.27).

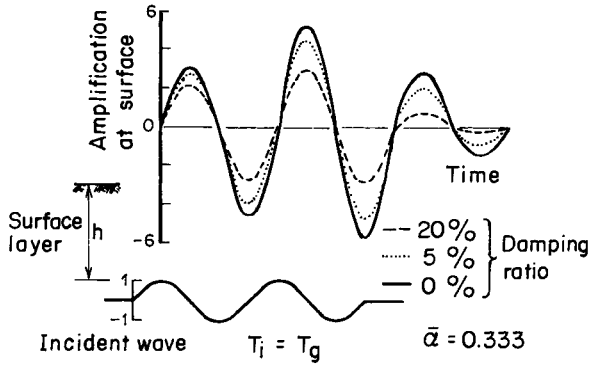

Fig. 11.23. Effect of damping ratio on the response of a surface soil layer at resonance.

With increasing strain level, the rigidity (shear modulus) of soft clay decreases, and the damping ratio increases, as discussed in Section 11.2; the superposition of both effects then determines the final response amplitude of the soft ground layer. Furthermore, it can be concluded that, as a general characteristic, the natural period of a soft clay deposit is increased by stronger base rock motion, because of the loss in soil rigidity, and its amplification factor decreases under stronger earthquake motion because of an increased damping ratio. This is illustrated in Fig. 11.24, which shows the results obtained by Goodman & Apphun (1966) from a laboratory model study in which the response of a 76 mm thick soft clay layer to base motions of different intensities was investigated. It can be seen that an increase in the amplitude of the base motion, $A_{(z'=0)}$, resulted in a decrease in the amplification factor and an increase in the resonance period, T_r, of the surface motion.

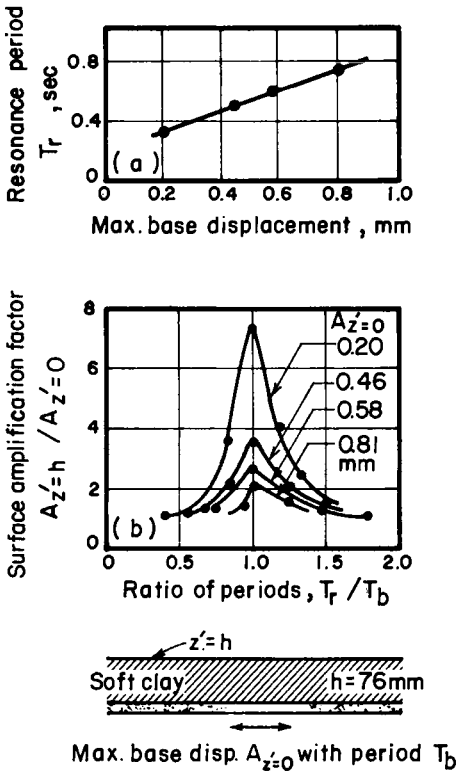

Fig. 11.24. Laboratory results of Goodman & Apphun (1966) on a soft clay layer, which show that an increase in the base displacement amplitude, $A_{z'=0}$, results in: (a) an increased resonance period, (b) a decreased surface amplification factor (i.e. increased damping).

11.3.2.4 In-situ observations of predominant period of soft ground

Kanai (1962) compared the results of microtremor measurements with strong earthquake motion records at the same site, and he concluded that the predominant period of each observed earthquake motion was the same when the earthquake magnitude exceeded 6, and that its value corresponded well to that of microtremors. Kanai also found, however, that the predominant period of the observed earthquake motion changed from one earthquake to another, but, in each case, the predominant period corresponded to one of the several peaks observed in the microtremor records.

Measurements of the predominant periods of two soft soil deposits were presented by Okamoto (1971) on the basis of strong motion records obtained at two soft ground sites which had been shaken repeatedly by earthquakes. At the first site, at Koto, a silty clay layer about 50 m thick (with shear wave velocities between 160 and 170 m/sec) overlies a very dense sand layer; at the other site, at Arakawa, soft clay and loose sand layers about 30 m thick (with shear wave velocities of 150 m/sec) overlie a very dense sand layer. Analysis of several strong motion records from the two sites revealed that the fundamental mode of vibration was not always excited during every earthquake. Only higher modes were generally observed in the records of smaller earthquakes, while the fundamental mode was observed in those of larger earthquakes. Table 11.3 indicates the predominant periods of the strong motion records obtained from earthquakes of various magnitudes at the two sites. The fundamental period at each site was estimated from the thickness of the soft layer and the corresponding shear wave velocity and was about 1.2 sec for the Koto site and 0.77 sec for the Arakawa site.

Table 11.3. Change of predominant period of strong motion

Earthquake magnitude	Predominant period, sec.	
	Koto site	Arakawa site
Large	1.35	0.77
Medium	0.46	0.77
Small	0.25	0.17

Strong motion records of thick soft layers generated by weak earthquakes possibly lack the characteristics pertaining to stronger earthquakes. This could be explained by the smaller periods of the exciting motion of a smaller earthquake and by the lack of energy necessary to excite the long-period fundamental mode of a thick soft layer.

It should be realized that, when the seismic behaviour observed with weak earthquakes and thick soft clay layers is extrapolated to stronger earthquakes, the predictions might be misleading.

11.3.3 Damage to structures on soft clay

Severe damage to structures founded on or in soft clayey ground has been observed in past earthquakes, in addition to damage caused by the liquefaction of loose saturated sand deposits. A brief summary will now be given of earthquake damage to structures such as buildings, embankments, slopes and structural foundations.

11.3.3.1 Housing structures

Kanai (1947) described the two main mechanisms which cause structural damage during earthquakes in soft soil conditions. These are: (1) differential foundation movements caused by large displacement amplitudes of the soft clay which, in turn, produce excessive stresses or displacements in the superstructure, and (2) strong resonant vibrations of structures, like Japanese wooden houses, which have a natural period similar to that of soft clay ground.

Structural damage in the Tokyo area caused by the Kwanto earthquake of 1923 (M=7.9 and epicentral distance=90 km) can be used to illustrate the effect of interaction between soil and structure under resonant conditions. The natural period of Japanese wooden houses is in the range 0.4 to 0.6 sec. This corresponds to the natural period of a soft clay layer 20 to 30 m thick under small strain levels. The damage in the Tokyo area was investigated by Ohsaki (1969) and is shown in Fig. 11.25 in terms of type of structure and soil conditions. It can be seen that the damage rate of wooden houses was higher in the softer ground conditions, while concrete structures and warehouses with mud walls suffered severe damage when located on hard ground. Concrete structures and structures with mud walls have a shorter natural period than wooden houses.

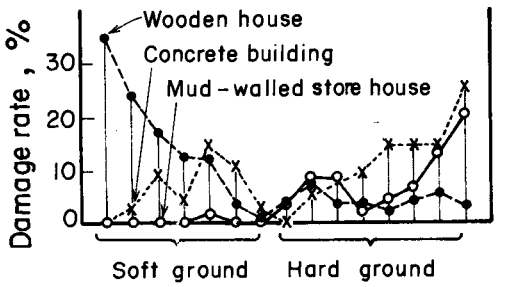

Fig. 11.25. Structural damage rate of various types of building in the Tokyo area related to different ground conditions (Ohsaki, 1969).

A tendency towards greater damage of 'short-period' structures, such as masonry or brick structures, on hard soil was also reported for the 1933 earthquake in Long Beach, California (M=6.3) and for the 1949 earthquake in Imaichi, Japan (M=6.4, epicentral distance=10 km).

For the Caracas earthquake of 1967 (M=6.4, epicentral distance approx. 60 km), the distribution of structural damage for buildings with different numbers of storeys and located on various foundation soils was explained by Seed et al (1972) by means of the interaction between soil conditions and structural response.

Table 11.4. Failure rate of wooden houses on different subsoils

Subsoil type	Failure rate, %
Clayey	26.1
Sand	1.4
Sand dune	3.5
Hard or rock	0.2

A study of damage to wooden structures on clayey soil and sandy soil in the Enshu district (Pacific coast of central Japan) caused by the Tonankai earthquake of 1944 (M=8.0, epicentral distance about 200 km), was made by Ooba (1957), and his findings are shown in Table 11.4. Clay subsoils appear to result in greater damage to such structures than sandy or hard ground. One reason for this might be the lower damping associated with clayey soils, which produces stronger ground motion. Figure 11.26 shows a comparison by Iwasaki (1969) between computed ground motions at the surface of a clay layer and a sand layer with the same thickness subjected to the same base rock motion. This figure shows that the maximum accelerations and the response spectra are nearly identical when the layer is thin, but that the values for the clay layer are almost double those of the sand layer when the thickness of the layer exceeds 10 m.

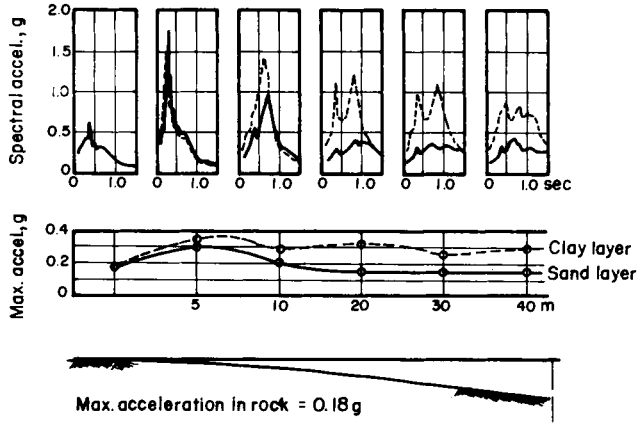

Fig. 11.26. Predicted surface accelerations of a sand and a clay layer of equal thickness, illustrating the influence of soil type and stratum thickness (Iwasaki, 1969).

11.3.3.2 Embankments and slopes

It has been observed that embankments on soft ground and slopes containing soft clay layers are prone to failure during earthquakes. An embankment 15 m high across a paddy field near Tottori City in western Japan, for example, collapsed over a length of 150 m and settled a maximum of 7 m during the Tottori earthquake of 1943 ($M=7.4$, epicentral distance about 20 km). During the Fukui earthquake of 1948 ($M=7.3$, epicentral distance about 20 km), an embankment 8 m high on a deep alluvial layer of soft silty clay failed and settled about 4 m over a length of 400 m; several longitudinal fissures were observed on the crest of the embankment, and the embankment widened to almost twice its original width.

Ikehara (1970) and Kobayashi & Fujiwara (1974) reported four types of embankment failure on soft ground observed during the Tokachioki earthquake of 1968 ($M=7.9$, epicentral distance about 150 to 200 km). In this earthquake, more than 500 embankments were damaged. The failure types distinguished were: (1) settlement, (2) surface slip of a thin layer on the slope, (3) crest splitting, and (4) complete sliding.

Shibata et al (1971) also reported damage of earth structures and slides on natural slopes which occurred during the Tokachioki earthquake of 1968. The distribution of damage was most closely related to the geological condition in the area, rather than to other factors such as the epicentral distance. The occurrence of damage was concentrated in an area of volcanic ash and alluvial soft ground. Slides occurred on natural slopes with slope angles of about 20° in which the failure surface was found in each case to exist at a few metres below the surface of the slope, where a layer of soft clay of volcanic origin was present. Most of the embankment failures were found to be related to such locations as: (1) narrow valleys, (2) boundaries between soft alluvial soil and hard ground, and (3) soft ground. The Tokachioki earthquake was preceded by a few days of heavy rainfall, which probably caused higher water contents and which, in turn, resulted in a decrease in soil strength and consequent greater damage.

Asada (1975) reported on the failures of dykes in a reclaimed area at Hachirogata which were caused by an earthquake of magnitude 6.9 and epicentral distance of about 100 km. Figure 11.27 illustrates how the settlement of the dykes increased with increase in depth of the soft ground. Asada also pointed out that the periodic distribution of damage observed in several cases along linear structures, such as highways, might have been caused by the action of seismic surface waves. An embankment, when located over soft ground, might have experienced stronger earthquake motions than over hard ground, because of the long period waves from long-distance earthquakes which excited the soft soil layer which had a long natural period.

The designer of embankments on soft ground which will be subjected to seismic loading should take the following precautions:

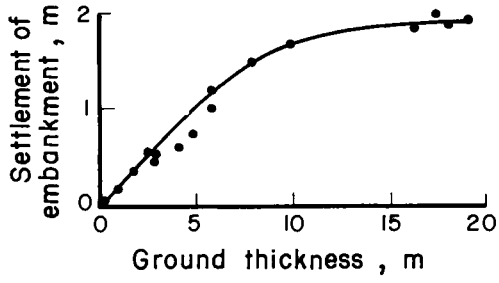

Fig. 11.27. Effect of soft clay thickness on the settlements of embankments caused by an earthquake at Hachirogata.

(1) Provide a large enough factor of safety against static bearing capacity failure, say greater than 3.0.
(2) Construct the embankment of a well-graded material with a high permeability, and avoid poorly graded sand.
(3) Drive piles sufficiently deep at the toe of the embankment to prevent a sliding failure through the soft clay foundation ground.

11.3.3.3 Structural foundations and retaining structures

Foundations and retaining structures in soft ground, such as bridge foundations and quay walls, have repeatedly suffered damage from earthquakes. Iwasaki (1973) summarized the damage to bridge structures which have occurred during past earthquakes in Japan, such as the Kwanto (1923), Fukui (1948), Tokachioki (1952, 1968) and Niigata (1964) earthquakes. He emphasized that soft soils surrounding bridge foundations were one of the major factors which cause foundation damage such as settlement, tilting, sliding and overturning.

Failures of quay walls have been reported by Amano et al (1956), Duke & Leeds (1963) and Hayashi et al (1966). The gravity retaining walls of a caisson structure at Shimizu Harbour suffered 0.9 m of settlement and a sliding displacement of 1 to 5 m as a result of the Shizuoka earthquake of 1935 (M=6.3, epicentral distance 10 km). During the Tonankai earthquake (M=8.0, epicentral distance 130 km), the retaining wall above a relieving platform at Osaka Harbour moved outwards by 3 m and settled about 0.1 m.

The design of retaining structures such as gravity walls and sheet piles under earthquake conditions should take account of the dynamic earth pressure of the soil. For sands, dynamic earth pressures have been studied extensively, both theoretically and experimentally, by Mononobe (1924), Okabe (1924), Matsuo (1941), Matsuo & Ohara (1960), Ichihara et al (1977) and others. The dynamic earth pressure of clayey soil, however, is not yet well understood. An extensive theoretical study of the earth pressure of clayey soil under seismic conditions was described by Ichihara & Nakane (1976); they

expanded the solution for static earth pressure, derived by Sokolovsky (1957), to include seismic conditions, and also developed a computation technique to obtain more rigorous analytical solutions.

Settlements of piles as a result of earthquake action produce differential movements in superstructures which cause severe damage. In the Kwanto earthquake, settlements of piles for railway bridges increased with decrease in the ratio Q_u/Q_d, where Q_u is the failure load from a pile load test and Q_d is the dead load on the pile. Figure 11.28 shows that excessive settlements occurred when the load ratio, Q_u/Q_d, was less than 3.

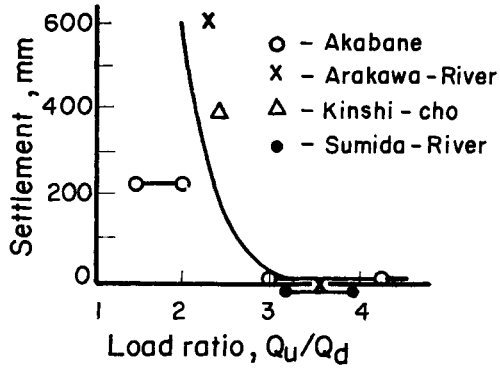

Fig. 11.28. Settlements of piles during the 1923 Kwanto earthquake related to the ratio of static failure load to the dead load on the pile (Kodera, 1964).

Vertical piles in soft soil give larger horizontal displacements when acted on by lateral loads. It is common practice, therefore, to use batter piles, which are better suited to absorb both axial and horizontal loads. At present, however, the behaviour of a batter pile system under seismic loading in soft clay has not been fully investigated.

The recent practice of using long piles in deep soft ground (Okamoto, 1971) suggests that a pile may move with the surrounding soil, and that non-uniform displacements along the pile axis are likely to cause bending failure of the pile.

Observations on the behaviour of pile-supported structures in soft clay have been reported by Sugimura (1975), Yamada et al (1975) and Masaki et al (1975). Observational results of submerged tunnels on soft clay layers have been reported by Nakayama & Kiyomiya (1975), Hamada et al (1975) and Okamoto et al (1973).

11.3.4 Analytical methods

A number of analytical and numerical methods are available today to obtain the earthquake induced behaviour of soft soil layers.

For a horizontally layered system, Kanai et al (1965) developed analytical procedures based upon multiple reflection theory and the assumption that the soil was elastic and visco-elastic. Idriss & Seed (1968) modelled a layered soil deposit as a lumped mass system and introduced strain-dependent stress-strain relationships. Richart (1975) utilized the method of characteristics to solve wave propagation problems with incorporation of nonlinear stress-strain characteristics, and he compared the results with other methods.

In order to model realistically, it is now common practice to assume nonlinear stress-strain relationships for soils. Step-by-step integration techniques allow the use of any type of stress-strain relationship, but they are rather time-consuming. Computations based on modal analysis usually applied with lumped mass systems are more efficient. Damping values, however, have to be assigned for each mode, and difficulty arises in assigning suitable values. The Fourier transform method, which solves the wave equation in terms of complex numbers, such as the program 'SHAKE' developed by Schnabel et al (1972), may at this time be the most economical approach to give reasonable solutions for horizontally layered sites. This program can take into account different damping ratios for each layer, as well as strain-dependent material characteristics.

The finite element method (FEM) of analysis can also be utilized, particularly for three-dimensional problems of arbitrary configuration. In FEM analysis, the equations of motion are often solved in the time domain. Lysmer (1970) also treated the wave equation in the frequency domain and showed how to solve the problem of surface wave propagation with irregular boundary conditions.

The versatility of FEM analysis allows the detailed treatment of many complex problems in soil dynamics, provided that the accuracy of the input data for the material involved (i.e. the dynamic constitutive equations discussed in Section 11.2) is commensurate with the degree of sophistication of the method of analysis.

The present state of knowledge of the seismic behaviour of soft clay deposits is still rather limited. This knowledge is derived mainly from laboratory experiments, while in-situ observations are scarce. On the other hand, high speed, large capacity computers have enabled the development of sophisticated methods of analysis. Extensive in-situ observations of the seismic behaviour of soft clay deposits and of structures on such soils are badly needed, and these will be the most valuable contributions to a deeper understanding of earthquake damage and its prevention in soft clay areas.

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