Structure and hydrocollapse behaviour of loess

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STRUCTURE AND HYDROCOLLAPSE BEHAVIOUR OF LOESS

by

ALBASHIR MOHAMMED ASSALLAY, BSc., MSc.

A Doctoral Thesis submitted in partial fulfilment of the requirements for the award of the Degree of Doctor of Philosophy of Loughborough University

June 1998

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(SEEK KNOWLEDGE, EVEN IN CHINA)
ABSTRACT

Loess has a metastable structure and undergoes structural hydrocollapse when loaded and wetted, leading to subsidence and damage of overlying structures, which presents enormous engineering problems in many countries in the world. This problem is a focus of much research on both the nature of the phenomenon and the means of overcoming its negative consequences. However, there remains a need for fundamental experimental and theoretical studies aimed at understanding the many uncertainties involved in the hydrocollapse phenomenon.

An experimental programme has been conducted to study the fundamental nature of the hydrocollapse phenomenon in loess, and the primary features of the metastable structure controlling it. High quality disc-shaped specimens were prepared from natural undisturbed, very soft and slightly cemented loess soils using two simple devices which were designed and manufactured in the laboratory. Artificial loess materials were prepared by mixing pure silt particles of different geometrical characteristics (crushed sand, ballotini glass balls) with various clay types (kaolinite, bentonite) to produce mixes with different clay/silt ratios. Soil specimens of metastable structure were created from natural and model loess materials using the air-fall technique, which allowed full control over the critical variables. The index properties of the materials were determined experimentally, together with their geometrical characteristics from scanning electron microscope (SEM) studies. The compressibility characteristics and hydrocollapse behaviour were measured via one-dimensional (oedometer) compression tests. Two testing methods were used: the single oedometer test and the double oedometer test.

The results indicated that the experimental approach used in this research can be used successfully to investigate the hydrocollapse problem of loess deposits. The small-clay loess model was constructed and examined. The results proved that this model is valid. The collapse behaviour of reconstituted and undisturbed loess specimens prepared from the same material was qualitatively similar, although the reconstituted materials exhibited greater collapsibility, particularly under low normal effective stresses. Specimens prepared from pure silt revealed very little collapse in
spite of relatively high initial voids ratios ($e \geq 0.9$). The existence of bonding materials, such as clay minerals at the points of particle contact, is thus apparently essential for hydrocollapse to occur. Specimens prepared from pure silt-bentonite mixtures exhibited low hydrocollapse values in comparison with those for pure silt-kaolinite mixtures. It was observed that there is an optimum clay mineral content for maximum hydrocollapse and it depends on the type of clay mineral and the level of applied stress. Higher values of hydrocollapse were obtained with smooth, spherical glass balls as the silt fraction, thus confirming that the geometrical properties of the silt particles also have a significant effect on the hydrocollapse behaviour of loess deposits. The position and distribution (mixing method) of clay particles inside the soil structure were equally found to have a significant effect on the hydrocollapse behaviour.

The index properties of Libyan loess were found to be very similar to those of other loess deposits world-wide. The Tripoli loess can be classified as silty loess with high susceptibility to structural hydrocollapse in the same manner as other better known loess deposits in the world.
ACKNOWLEDGEMENTS

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\( a_v \)  
Coefficient of compressibility

\( B \)  
Bentonite

\( \text{BGB} \)  
Ballotini glass balls

\( \text{BGB-K} \)  
Ballotini glass balls - Kaolinite Mix

\( C_c \)  
Compression Index

\( \text{CD} \)  
Consolidated-Drained Triaxial Test

\( \text{CEC} \)  
Cation exchange capacity

\( \text{CF} \)  
Clay fraction

\( \text{CMC} \)  
Clay mineral content

\( \text{CP} \)  
Collapse potential in percent

\( \text{C}_u \)  
Uniformity coefficient \((D_{60}/D_{10})\)

\( \text{CU} \)  
Consolidated-Undrained Triaxial Test

\( c_v \)  
Coefficient of consolidation

\( c' \)  
Effective cohesion

\( \text{DC} \)  
Dry Curve

\( \text{DM} \)  
Dry Mixing

\( \text{DOT} \)  
Double Oedometer Test

\( D_{10} \)  
Particle size corresponding to 10% passing

\( D_{40} \)  
" " " " 40% "

\( D_{50} \)  
" " " " 50% "

\( D_{60} \)  
" " " " 60% "

\( D_{100} \)  
" " " 100% "

\( \text{ECC} \)  
English China Clay

\( e \)  
Void ratio

\( e' \)  
Void ratio before wetting

\( e'' \)  
Void ratio after wetting
<table>
<thead>
<tr>
<th>Term</th>
<th>Definition</th>
</tr>
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<tr>
<td>e&lt;sub&gt;L&lt;/sub&gt;</td>
<td>Void ratio at Liquid Limit</td>
</tr>
<tr>
<td>e&lt;sub&gt;0&lt;/sub&gt;</td>
<td>Initial void ratio</td>
</tr>
<tr>
<td>G&lt;sub&gt;s&lt;/sub&gt;</td>
<td>Specific gravity of solid particles</td>
</tr>
<tr>
<td>H&lt;sub&gt;s&lt;/sub&gt;</td>
<td>Equivalent height of solids (oedometer test)</td>
</tr>
<tr>
<td>k</td>
<td>Coefficient of permeability</td>
</tr>
<tr>
<td>K</td>
<td>Kaolinite</td>
</tr>
<tr>
<td>LL</td>
<td>Liquid Limit</td>
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<tr>
<td>Mcs</td>
<td>Pure silt from crushed sand</td>
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<td>Mcs-B</td>
<td>Pure silt from crushed sand-Bentonite Mixes</td>
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<tr>
<td>Mcs-K</td>
<td>Pure silt from crushed sand-Kaolinite Mixes</td>
</tr>
<tr>
<td>MDD</td>
<td>Maximum dry density</td>
</tr>
<tr>
<td>M&lt;sub&gt;ml&lt;/sub&gt;</td>
<td>Pure silt from Malan Loess</td>
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<td>Mcs-Qc</td>
<td>Pure silt from crushed sand-Clay size quartz mixes</td>
</tr>
<tr>
<td>m&lt;sub&gt;v&lt;/sub&gt;</td>
<td>Coefficient of volume compressibility</td>
</tr>
<tr>
<td>N</td>
<td>Normal load</td>
</tr>
<tr>
<td>NC</td>
<td>Normally consolidated</td>
</tr>
<tr>
<td>NP</td>
<td>Non plastic</td>
</tr>
<tr>
<td>OC</td>
<td>Overconsolidated</td>
</tr>
<tr>
<td>OCR</td>
<td>Overconsolidation ratio</td>
</tr>
<tr>
<td>OMC</td>
<td>Optimum moisture content</td>
</tr>
<tr>
<td>P&lt;sub&gt;10&lt;/sub&gt;</td>
<td>Fraction passing No. 10 sieve</td>
</tr>
<tr>
<td>P&lt;sub&gt;200&lt;/sub&gt;</td>
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</tr>
<tr>
<td>PI</td>
<td>Plasticity Index</td>
</tr>
<tr>
<td>PL</td>
<td>Plastic Limit</td>
</tr>
<tr>
<td>Qc</td>
<td>Clay size quartz</td>
</tr>
<tr>
<td>R</td>
<td>Double layer and contact repulsions</td>
</tr>
<tr>
<td>R'&lt;</td>
<td>Residual factor</td>
</tr>
<tr>
<td>S</td>
<td>Average shear stress in the clay</td>
</tr>
<tr>
<td>SEM</td>
<td>Scanning electron microscope</td>
</tr>
<tr>
<td>Acronym</td>
<td>Definition</td>
</tr>
<tr>
<td>---------</td>
<td>------------</td>
</tr>
<tr>
<td>SOT</td>
<td>Single Oedometer test</td>
</tr>
<tr>
<td>$S_0$</td>
<td>Natural degree of saturation</td>
</tr>
<tr>
<td>SPT</td>
<td>Standard Penetration Test</td>
</tr>
<tr>
<td>SSA</td>
<td>Specific surface area</td>
</tr>
<tr>
<td>$u$</td>
<td>Pore water pressure</td>
</tr>
<tr>
<td>UU</td>
<td>Unconsolidated-Undrained Triaxial Test</td>
</tr>
<tr>
<td>WB</td>
<td>Wyoming Bentonite</td>
</tr>
<tr>
<td>WC</td>
<td>Wet Curve</td>
</tr>
<tr>
<td>WM</td>
<td>Wet mixing</td>
</tr>
<tr>
<td>$w_i$</td>
<td>Initial water content in percent</td>
</tr>
<tr>
<td>$W_s$</td>
<td>Water content at 100% degree of saturation</td>
</tr>
<tr>
<td>$W_L$</td>
<td>Water content at Liquid Limit</td>
</tr>
<tr>
<td>XRD</td>
<td>X-ray Diffraction</td>
</tr>
<tr>
<td>$(p_o)_d$</td>
<td>In-situ dry density</td>
</tr>
<tr>
<td>$(p_L)_d$</td>
<td>Dry density at Liquid Limit</td>
</tr>
<tr>
<td>$\rho_w$</td>
<td>Density of water</td>
</tr>
<tr>
<td>$\gamma_d$</td>
<td>Initial dry unit weight in kN/m$^3$</td>
</tr>
<tr>
<td>$\gamma_p$</td>
<td>Unit weight of pore fluid</td>
</tr>
<tr>
<td>$\phi'$</td>
<td>Friction angle</td>
</tr>
<tr>
<td>$\sigma$</td>
<td>Total stress</td>
</tr>
<tr>
<td>$\sigma'$</td>
<td>Effective stress or intergranular pressure</td>
</tr>
<tr>
<td>$\sigma'_n$</td>
<td>Normal effective stress</td>
</tr>
<tr>
<td>$\tau$</td>
<td>Shear strength</td>
</tr>
<tr>
<td>$\mu$</td>
<td>Coefficient of friction</td>
</tr>
<tr>
<td>1-D</td>
<td>One dimensional</td>
</tr>
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</table>
CHAPTER 1
INTRODUCTION

1.1 STARTING POINT

1.2 COLLAPSIBLE SOIL AND METASTABLE STRUCTURE

1.3 LOESS DEPOSITS AND THE PROBLEM OF HYDROCOLLAPSE

1.4 THE OBJECTIVES OF THE RESEARCH

1.5 MATERIALS AND METHODS OF TESTING

1.6 THESIS ORGANISATION
CHAPTER 1
INTRODUCTION

1.1 STARTING POINT

Because of the potential hazards to engineering structures founded on collapsible soils that undergo high volume change, there is a need to identify such soils and to understand clearly the engineering properties of these soils. The more the nature of the metastable soils is known the greater the ability to deal with these types of soil, and in particular the greater the possibility to find the most suitable methods for the improvement of their properties. The major sub-set of collapsing soil is the unsaturated group, of which loess is the most important member. Loess is widespread throughout the world. It has a metastable structure and is renowned as one of the world's problematic soils. The main geotechnical problem is structural hydrocollapse of loess when loaded and wetted, leading to subsidence and damage of overlying structures. The mechanism and influences on the degree of hydrocollapse still require investigation.

A number of attempts has been made by various investigators to study the hydrocollapse problem of loess deposits. Most of these trials have been conducted on the natural undisturbed materials, an approach which does not allow the different variables involved in the problem to be accurately controlled and investigated. In this research a new approach is followed which allows for all variables to be well controlled. It is believed that the approach of using reconstituted specimens prepared from different artificial loess materials, including a very ideal one based on ballotini (i.e. small glass spheres), is a good starting point for investigating the hydrocollapse problem of
loess deposits and this is the main task of this research. The metastable structure of collapsible soil, the problems associated with the hydrocollapse phenomenon, and the materials and methods utilised in this research will be explained in more detail in the subsequent chapters, but a brief explanation of these issues will be presented in the following sections.

1.2 COLAPSIBLE SOIL AND METASTABLE STRUCTURE

Collapsible soils are those soils of metastable structure which become unstable under certain loading or environmental conditions. To be metastable they must have an open structure, that is the granular frame of the solid particles must be in an open packing which is capable of becoming rapidly and significantly a closer packing, producing a stable structure (Sultan, 1969; Rogers, 1995). A granular material with angular particles compacted on the dry side of optimum can form a structure which is capable of significant further densification, but the classic collapsible soils are natural materials where the combination of particle type (size and shape) and sedimentation process combine to give a collapsing deposit of metastable structure.

Several definitions of a collapsible soil have been presented by different investigators (Sultan, 1969; Dudley, 1970; Handy, 1973; Jennings and Knight, 1975; Booth, 1977; Rogers, 1995). Most of these researchers are defining the soil collapse as a significant reduction in volume due to the wetting of a partially saturated soil, either with or without additional loading. This definition is limited only to soils encountered in arid and semi-arid regions, and it does not include some other types of soils which have metastable structures such as residual soils and quick clays. Rogers (1995), however, defined the collapsible soil in its widest possible sense in order that
the different types of these soils encountered in practice could be recognised and classified. He acknowledged that in most collapsible soils (including quick clays) the structural units will be primary mineral particles, rather than clay minerals. The collapse process that occurs in these soils gives them a geotechnical significance.

According to the definition presented by Rogers, the naturally occurring collapsible soils can be broadly divided into three main groups: residual soils, water-sedimented soils, and aeolian soils (airfall-sedimented soils). In residual soils, metastable structures form as a result of leaching of soluble and colloidal materials. Quick clays are fully saturated collapsible soils and one of the main types of collapsible water-deposited soils. They are found in abundance in Eastern Canada and in Scandinavia, and they can exist in a stable condition under a certain level of applied stress, above which a sudden collapse may occur. Water- and airfall-sedimented collapsible soils are usually found in arid and semi-arid regions, the exception being quick clays which are normally found in peri-glacial areas. Airfall-sedimented soils are the most extensive deposits of collapsible soils and have been encountered in many countries of the world. Naturally these types of soil occur in partially saturated or dry conditions. Based on the concept of collapsible soil presented above, it could occur almost anywhere in the world.

The common features of collapsible soil are: an open metastable structure of high void ratio and low density, and low interparticle bond strength. The structural collapse of this type of soil forms a major hazard in large areas of the earth's surface. Fredlund and Rahardjo (1993) reported that research associated with the behaviour of collapsible soils has been limited, and many questions related to the engineering properties of these soils remain to be answered from both research and practical standpoints. An increased awareness of the
existence of collapsible soil and an ability to recognise the engineering problems associated with these types of soil are very important objectives. They stated that there is still a great need for experimental and theoretical investigations which would provide precise qualitative and quantitative information that may present solutions to the many problems involving the collapse phenomenon. This investigation aims to satisfy this need by considering primarily airfall-deposited soil, taking the loess deposit as the most important example.

1.3 LOESS DEPOSITS AND THE PROBLEM OF HYDROCOLLAPSE

An important and widespread geotechnical problem involves soil sediments which possess considerable 'dry' strength in situ that is largely lost when these deposits become wetted. The problem requires particular attention in the design and construction of engineering structures where foundation wetting is to be expected. Probably the best known examples of engineering structures which have settled or damaged due to wetting of foundations are those which were built on loess deposits. Mitchell (1993) stated that "loess deposits, composed of silt-size particles, are of great interest because of their unique structure and properties".

Loess is a very widespread surface deposit. It occurs in large quantities in northern China, in Russia, in central North America, in many parts of Europe (particularly in the eastern regions), and in some parts of South America. It was defined by Smalley and Vita-Finzi (1968) as a clastic deposit which consists predominantly of quartz particles 20-60μm in diameter, and which occurs as wind-laid sheets. This definition has been modified by Smalley and Derbyshire (1990), particularly in order to place some emphasis on the
formation mechanisms. They have stated that "the modal loess particles are largely produced either by glacial grinding or by cold weathering in high regions; major distribution in the landscape occurs by fluvial action; a final short aeolian transportation stage confers on the loess deposit its characteristic properties of collapsibility and permeability".

Loess soil is a porous deposit blanketing the landscape, and usually occurs naturally in a partially saturated state. It is a loose wind-blown deposit mainly composed of uniform silt-sized particles which are generally bonded together by small amounts of clay coatings/bridges (and salt formations) to form an open granular-framed metastable structure of high void ratio and low dry density. The clay bridges and coatings act as a primary cement at interparticle contacts to give the loess structure considerable strength when it is in a dry state. However, since the cementing material constitutes a very small fraction of the total mass of soil, only a slight increase in moisture content may significantly affect the strength of soil. Under extreme conditions such as saturation or flooding, the soil structure may collapse completely under an imposed load (or even under self weight) and large settlements result, a phenomenon termed herein hydro-collapse.

The susceptibility of loess deposits to large reductions in total volume due to structural hydro-collapse may cause severe damage to and sometimes complete failure of engineering structures which are founded on loess strata, or constructed from loess materials such as earth dams and highway embankments. The total collapse of the Teton Dam in Idaho (North America) in 1976 (Smalley and Dijkstra 1991, Rogers et al 1994b, 1994c) and the huge flowslide movements of loess ground in Gansu province in China due to the great 1920 earthquake, in which many thousands of people were killed, are good examples of how the loess deposits can be hazardous. Sowers (1993)
described the failure of the Teton Dam as the worst civil engineering failure of the 20th century.

Loess is a classic airfall deposit. It is an inherently collapsible material because of its particle shape, size and nature, and the tendency to form short-range interparticle bonds. The airfall sedimentation process seems to be essentially responsible for the formation of the metastable open structure that is susceptible to hydrocollapse. In addition, the majority of loess deposit problems are attributed to changes in the state of the soil moisture. Hence a good knowledge of the collapse behaviour of loess material resulting from the initial placement of the engineering structure and the subsequent changes in soil moisture is one of the most important requirements for prediction of the behaviour of an engineering structure which is founded on or in loess deposits. The worldwide problems with loess have recently been reviewed by Rogers et al. (1994a), who indicated some topics for future research. One of these was to obtain a better understanding of the nature of the open soil structure and to generate some model studies.

Particle packing and interparticle bonding are the most important parameters of soil structure. Loess structure needs to be studied and experimented with at a more fundamental level. In geotechnical terms the problem of structure needs to be tackled at the single-particle level, with attention being paid to the nature of the interparticle contacts and bonds. The basic problems are to find ways of investigating and describing the interparticle bonding and particle packing of soil structure, and of relating structural characteristics to geotechnical properties.
1.4 THE OBJECTIVES OF THE RESEARCH

Geometrical properties (size and shape) of the silt particles, particle packing and the nature of the interparticle contact bonds are the key factors which control the hydro-collapse behaviour of loess deposits. The investigation of the role of soil composition and structure is of fundamental importance in the formation and behaviour of the porous metastable structure of loess deposits. It is necessary to specify formation of a wide range of soil mixtures from pure minerals crushed to various sizes, as well as sediments accumulating from primarily silt and clay particles. More attention should be given to the silt-clay (M-C) mixture. It is very important to study the role of clay minerals of different types and in different amounts in the process of modelling the metastable structure of loess as well as their effects on the hydro-collapse behaviour of loess deposits.

The main aim of this research is to determine the key factors that control the hydro-collapse phenomenon of loess deposits, and to evaluate (i.e. quantify) their effects on this problem. To meet this aim, the following objectives were formulated.

1. To examine the possibility of modelling unstable and metastable structures of loess deposits experimentally, and to find out a proper method of simulating the collapsible structure of real loess.

2. To examine the possibility of simulating the natural processes for producing silt particles which represent those natural silt particles of typical loess deposits and thereby to determine the most appropriate grinding technique to model the glacial grinding process.
3. To determine the compressibility properties and collapse behaviour of natural undisturbed and reconstituted loess samples by testing real and model loess specimens.

4. To test the small clay-loess model and to investigate the role of clay minerals of different types and in different amounts in the process of simulating the metastable structure of collapsible soil and to determine the effect of these bonding materials on the hydrocollapse behaviour of silt-clay (M-C) mixtures.

5. To measure quantitatively the effect of the soil fabric (i.e. varying distribution of the clay material between the silt particles) on the degree of hydrocollapse by testing reconstituted soil specimens prepared by different mixing methods.

6. To evaluate the role of the shape and surface texture features of the primary solid particles on the hydrocollapse behaviour of loess.

7. To study the geometrical characteristics and hydrocollapse behaviour of peri-Saharan desert loess samples collected from Libya in North Africa. This will provide evidence of the existence of loess material in the peri-Saharan desert region. Natural undisturbed and reconstituted soil specimens from China in the middle of Asia will also be studied for comparison and validation of the models developed.
1.5 MATERIALS AND METHODS OF TESTING

In order to achieve the objectives of this investigation, a research programme has been designed to study the main aspects of the metastable structure and the hydro-collapse behaviour of loess deposits. Two different experimental approaches have been adopted:

(a) Testing of natural undisturbed (real) soil samples.
(b) Testing of artificial reconstituted (model) soil samples.

The real and model soil specimens have been prepared from different materials and tested under various conditions. Different types of soil materials have been investigated, as follows:

1. Undisturbed and remoulded loess samples from Libya.
2. Undisturbed and remoulded loess samples from China.
3. Artificial silt particles produced in the laboratory from crushed sand.
5. Different silt-clay (M-C) mixtures with different clay/silt (C/M) ratios.

Several methods for the preparation of undisturbed and reconstituted specimens have been used. New simple devices for the preparation of undisturbed specimens have been designed and manufactured. Different methods for the preparation of reconstituted soil samples have been attempted.

The compressibility and hydrocollapse behaviour of the real and model specimens have been determined using the oedometer compression test. Two methods of testing were carried out, the single oedometer test and the double oedometer test. The index properties and geometrical characteristics of the materials used in this investigation have also been measured according to British Standard procedures (BSI, 1990) and examined using the scanning electron microscope (SEM).
1.6 THESIS ORGANISATION

This thesis consists of nine chapters. The definition of the problem, the objectives and the research approach used in this study are introduced in this Chapter.

In Chapter two, the current definitions of collapsible soil are discussed and a new definition which includes all types of collapsing soil are introduced. The main types, distribution and features of collapsible soil are presented. This chapter also explains the prediction criteria of soil collapse, the factors which control the collapse phenomenon and the collapse mechanism of this type of soil.

The formation and geotechnical properties of loess are presented in Chapter 3, followed in Chapter 4 by a detailed explanation of the hydrocollapsibility of the metastable structure, the most important and interesting phenomenon of loess deposits. Attention is paid mainly to the structural factors which control this phenomenon and accurate experimental modelling. The natural and artificial materials used in this research, the index properties and geometrical characteristics of these materials as well as the testing methods used to measure these properties are presented in Chapter 5.

Chapter 6 explains the techniques used for the preparation of soil specimens and the experimental methods employed for evaluating the loess hydrocollapse. The results obtained from the laboratory work are presented in Chapter 7, and discussed in more detail in Chapter 8. A summary of a part of this work appears in Assallay et al (1996, 1997) and further details will appear in
Assallay et al (1998a, 1998b). Finally, the main conclusions of this research and recommendations for further studies are summarised in Chapter 9.
CHAPTER 2
COLLAPSIBLE SOIL

2.1 INTRODUCTION

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CHAPTER 2
COLLAPSIBLE SOIL

2.1 INTRODUCTION

Collapsible soils are those soils which have open metastable structures and become unstable under certain loading or environmental conditions. The NATO Advanced Research Workshop on Genesis and Properties of Collapsible Soils held in Loughborough, UK, (11-14 April 1994), edited by Derbyshire et al (1995), concluded with a new definition of collapsing soil which emphasises the structural parameter as a unifying factor. Given that collapse is the transition from an open metastable packing to a closer, more stable packing, then the open structure of quick clays, loess and sands is a unifying entity. According to this definition, the unsaturated soil deposits such as loess and the high moisture content and fully saturated systems, such as quick clays and submarine sands (although they are extremely different in formation process and genesis, environmental condition, and natural water content), may be considered as collapsible soils. Under the proper set of conditions, soil deposits compacted on the dry side of optimum moisture content also appear, in many cases, susceptible to collapse. The natural soil deposits with a collapsible structure may be residual, water-sedimented or airfall-deposited (aeolian) soil.

Collapsing soils have attracted the attention of many research workers over the past two decades. Several research programmes dealing with collapsing soils, their engineering properties, the different factors which affect their deformational behaviour and the engineering problems associated with them upon wetting have been conducted. The most important factors which affect
the collapsibility characteristics of collapsing soils are initial dry density, the initial water content, the type and amount of clay minerals, the grain size distribution, and the applied stress. The most comprehensive state-of-the-art papers on collapsing soils are by Sultan (1969), Northey (1969), Dudley (1970), Clemence and Finbarr (1981), Rogers et al (1994a) and Rogers (1995). Other studies have been devoted to determining the mechanism of collapse, predictive methods, treatment methods, and case histories, including those by Barden (1973), Jennings and Knight (1975), Lutenegger et al (1979), Schwartz (1985), and Machechnie (1989).

Collapsible soils are deposited in such a way that they exist with an open structure having a relatively low density and high void ratio. The collapse of soils forms a major hazard in large parts of Eastern Europe, the United States, Eastern Canada, Southern Africa, China, Eastern Europe, Russia, and other places. The most extensive deposits of collapsible soils are aeolian, or wind-deposited, fine sands and silts (loess). In most cases, the deposits are characterized by loose structures of primary solid particles often in the form of silt to fine sand size (Clemence and Finbarr 1981). A review of the literature indicates that collapsing soils may have many formational origins and geneses; however, they usually have the common property of being highly porous, and of geologically recent age (Sultan 1969). Soil collapse is associated with two fundamental properties of collapsible soils: an open structure of high voids ratio with small dry density and relatively low interparticle bond strength (Feda 1993).

Collapsible soils may cause severe damage to engineering structures which are placed on them or to those structures constructed with them such as earth dams and highway embankments. Damage caused by collapsible soils is reported in different parts of the world such as the U.S.A. (Holtz and Hilf 1961; Bull

The purpose of this chapter is to introduce the definition, types and distribution of collapsing soil in the general sense, and to concentrate on the properties and behaviour of the unsaturated air-fall deposited soils, in particular loess materials.

2.2 WHAT IS COLLAPSIBLE SOIL? (Definition)

Various investigators tried to answer this question in many different ways. Most of them have considered the collapsible soil as a partially saturated soil. For example, Dudley (1970) defined the collapsible soil as "any unsaturated soil that goes through a radical re-arrangement of particles and great loss of volume upon wetting with or without additional loading". In 1975 Jennings and Knight defined soil collapse as an additional settlement due to the wetting of a partially saturated soil, normally without any increase in applied pressure. Collapse settlement in compacted soil was defined by Booth (1977) as a settlement in partially saturated soil due to an increase in the degree of consolidation. Handy (1973) defined collapsible soil as a state of under-consolidation related to apparent cohesive strength of unsaturated soil. Houston and Houston (1990) stated that "when a dry or partially saturated soil
contracts upon wetting it is said to be collapsible". Rizkallah and Keese (1989) from Canada defined soil collapse as a settlement of soil caused by sudden breakdown of grain-fabric due to wetting, under a constant surcharge. From Brazil, Signer et al (1989) defined collapsible soils as those soils which have such a structure that suffers a high volumetric reduction when soaked, whether or not subjected to an additional load. Mackechnie (1989) in his general report to the twelfth International Conference on Soil Mechanics and Foundation Engineering stated that "not all soils with low unit weights are necessarily collapsible, but partial saturation is generally a prerequisite to collapse".

It is clear from these definitions that partially saturated soils form the main types of collapsible soils and that there is a tendency for authors to limit their investigations to soils from particular areas found in and around semi-arid regions. It is also clear that all these definitions, and most others which are found in literature, are based on the concept that the soil collapse happens mainly due to the effect of wetting.

There are very few definitions which present collapsible soils in a more general sense. Two of them are reported here as examples. Sultan (1969) defines a collapsible soil as a soil that undergoes an appreciable amount of volume change upon wetting, load application or a combined effect of both. Zur and Wiseman (1973) stated that "a definition of collapse should be general enough to include the whole variety of collapse manifestations. It is proposed that any rapid decrease in volume brought upon by the increase in any one of or combination of the following: water content, degree of saturation, mean stress, shear stress, or pore pressure, should be termed collapse. This definition recognises that collapse of the soil structure may be triggered by a variety of processes other than wetting".
Rogers (1995) discussed the problem of the definition of collapsible soil and criticised all previous definitions from a geotechnical engineering viewpoint. He presented this issue by considering collapsible soil in its widest possible sense, in order to allow both compacted and natural (saturated soil such as quick clay and unsaturated soil such as loess) materials to be included. In his critique, he determined the points of weaknesses of these previous definitions and noted that:

1. Most of the definitions are restricting the subject to unsaturated soils that are collapsed by wetting.
2. All definitions failed to quantify the amount of settlement that occurs due to the collapse of soil structure.
3. Some authors have used geotechnical terms such as consolidation and under-consolidation, which could result in considerable confusion according to geotechnical terminology.

Considering that the main defining feature of these soils is the initial open structure, Rogers introduced soil collapse as a change in structure from metastable to more stable. Finally he defined collapsing soil as "that soil in which the constituent parts have an open packing and which forms a metastable state that can collapse to form a closer packed, more stable structure of significantly reduced volume. In most collapsible soils the structural units will be primary mineral particles, rather than clay minerals. The collapse process that occurs in these soils gives them a geotechnical significance".

2.3 THE MAIN TYPES OF COLLAPSIBLE SOIL

Rogers (1995) stated that "a granular material with angular particles compacted on the dry side of the optimum moisture content can form a structure which is capable of significant further densification (collapse), but the classic
collapsible soils are natural materials where the combination of particle type and sedimentation mechanisms combine to give collapsibility. According to this statement and his classification of collapsible soil (see Figure 2.1), he divided soil materials into two main groups when considering collapse:

1. Man-made soils that have not been compacted or have been compacted on the dry side of the optimum moisture content.

### 2.3.1 Man-Made Soils

Many cases of substantial damage related to soil collapse in compacted fills have been reported in the literature since 1953 (Lawton et al 1992). Lawton et al indicated that "a literature review has shown that nearly all types of compacted soils are subject to collapse under certain conditions (Lawton et al 1991b). It is important to note that even clean sands, pure clays, and soils containing substantial gravel fraction can collapse (Steadman, 1987; Dakshanamurthy, 1979; Jaky, 1948)." Collapse settlement has occurred in several types of compacted fill, including earth dams, highway embankments, and other deep compacted fills such as filled canyons. Several failures of earth dams have been reported in literature. For example, Peterson and Iverson (1953) described how two earth dams built in Western Canada in the late 1930's and early 1940's failed as a result of collapse. Leonards and Narain (1963) found that the main causal factor of the cracking of three earth dams located in California was differential settlements resulting from collapse within the embankment material. Wetting-induced collapse has also been cited as a key component of the 1976 collapse of the Teton Dam in Idaho (Leonards and Davidson, 1984). Many cases involving damage or distress related to collapse in fills other than earth dams have been mentioned in the literature. Several
cases of deformation, cracking, and failure of highway embankments in South Africa resulting from wetting-induced collapse settlements have been recorded (Booth, 1977; Schwartz, 1985). In the last few years, the southern California area has experienced land subsidence problems that appear to be the consequence of collapse settlement of compacted fills (Lawton et. al., 1992). The National House Builders Council of Great Britain considered the problem of collapse settlement of floor slabs placed on fills to be serious enough to discuss it in their 1977 Guidance Notes for House Construction (Cox, 1978).

2.3.2 Naturally Deposited Soils

Natural soil deposits are derived from some combination of weathering, transportation and deposition of materials which decomposed or disintegrated from rock minerals. Sediments of collapsing structure occur as a result of one or more of these geological processes. The term weathering must include alteration in situ post deposition since this is often the key to the maintenance or creation of a metastable structure of soil deposits (Rogers, 1995). Residual soil, quick clay (water deposited) and loess (wind deposited) are the most important examples of the naturally deposited collapsible soils.

2.3.2.1 Residual Soil

Residual Soil has formed in place due to the effect of weathering processes, i.e. the decomposition, disintegration and mechanical alteration of the components of parent rock. The produced particles may vary in size from large fragments to gravel, sand, silt, clays, and in some cases, organic matter. The deposits of residual soils are characterized by highly variable thickness ranging from a few centimeters to several metres depending on the climate and environmental conditions of the region. The collapsible structure has developed as a result of
leaching out and loss of the soluble and colloidal materials (Clemence, 1981; Blight, 1997). One of the best known examples of weathering in situ to create a collapsible soil is the granitic sands of South Africa. The weathering process is geochemically controlled and is manifested by weathering of feldspar, leaving a sub-rounded sand with an open structure (Jennings and Knight, 1957; Brink and Kanty, 1961; Rogers, 1995; Blight, 1997). Subsidence of residual soils originated from decomposed gneiss was reported (Feda, 1966). Also decomposed granites in Rhodesia have been found to have a total collapse of about 7% to 10% (Brink and Kanty, 1961).

2.3.2.2 Water-Sedimented Deposits Of collapsible Soils
The water-sedimented deposits of collapsible soils can be classified into two main groups: the alluvial deposits and quick clays. In the alluvial deposits the soil particles are carried and deposited by water to form alluvial fans, alluvial flood plains and mud flows. Several cases of collapsing problems associated with these formations have been reported (Holtz and Hilf, 1961; Gibbs and Bara, 1967; Bull, 1964; Clemence and Finbarr, 1981; Rollins et al, 1994). For example Clemence and Finbarr (1981) reported that cases of subsidence up to 15% have been observed in the San Joaquin Valley of California. Rollins et al (1994) presented six case histories involving collapsible problems that were observed in alluvial fans. They stated that "experience clearly shows that coarse-grain soils in a geologically susceptible environment can collapse upon wetting, even those which may have high densities and very high SPT blow counts".

The saturated quick clays can be classified as soils of metastable structures which become unstable under certain conditions (Sultan, 1969; Rogers, 1995; Terzaghi et al, 1996). Rogers (1995) discussed the formation mechanisms and collapse behaviour of quick clay deposits and concluded that quick clay is a
further example of a collapsible soil and thus spreads the distribution of collapsing soil from the arid and semi-arid zones to potentially world wide. Quick clays occur at glacial fringes and consist essentially of fine grained primary mineral particles which have been formed by the action of glacial grinding. Jefferson and Smalley, (1997) defined the sensitive clay (quick clay) as type C (2-5μm) particles (see Figure 2.2) in an open structure with short range bonding between the particles. The shape of these very small particles is typical of the blade-shaped, sub-angular particles similar to that of many of the loess materials which are created by hot or cold weathering (see Fig. 2.3) These particles are then deposited by slow fall sedimentation under shallow marine conditions to form deposits of open structure. The open structure is maintained by small amounts of carbonate cementation or clay mineral materials at the interparticle contacts. Quick clays typically have a water content that is greater than their Liquid Limit, and they might have a sensitivity of 200 and a plasticity index of 5. One of the important properties of sensitive clay is the significant shrinkage that occurs due to the change in moisture conditions, resulting in serious problems for shallow foundations (Bentley and Smalley, 1984). Typical examples of quick clays have been found in Scandinavia and Canada (Bentley et al., 1980).

2.3.2.3 Air-fall-Sedimented Deposits Of Collapsible Soils

The most predominant and diverse type of collapsible soils are the airfall-sedimented (aeolian deposited) group. These types of soil are normally unsaturated and they are found in arid and semi-arid regions. Loose sand and loess deposits are the main kind of wind-blown collapsing soils. Large areas of Southern Africa are covered by a blanket of dune sands which were deposited under desert conditions. These sand deposits were partly weathered in situ and they now contain a small percentage of silt and clay material. Other examples of collapsing sand formations are the shifting sand dunes of the Sahara in
North Africa. These consist of essentially rounded primary mineral particles of 100-200 μm in diameter that are held in position by short-range bonds, and subsequently cementation, under suitably favourable conditions (Rogers, 1995).

The most extensive deposits of unsaturated collapsible soils are loess which primarily consist of silt-sized particles that derive from cold weathering, hot weathering and glacial grinding. These particles are commonly transported by rivers and then picked up and carried by wind to place them into their ultimate position. The aeolian mechanism of transportation allowed some degree of particle angularity to be retained in most deposits. The Chinese loess from the Tibetan mountains and South American loess from the Andes are good examples of loess deposits which have been produced by the cold weathering process. Loess deposits in North America have been reported as being created by a glacial grinding environment (Smalley, 1966). The loess materials observed in the Canary Islands, the Cape Verde Islands and in some other places around the Sahara (Nigeria, Libya, Tunisia) were considered to be formed as a result of the hot weathering process in the Sahara (Coudé-Gaussen, 1991; Rognon et al, 1996; Coudé-Gaussen, 1987).

The formation processes and geotechnical properties of loess deposits will be discussed in more detail in the next chapter. Hereafter the properties and collapse behaviour of unsaturated collapsible soils in general, and the wind-blown deposited material in particular, will be presented.
2.4 DISTRIBUTION OF COLLAPSIBLE SOIL

It is clear from the previous section that the collapsing soil deposits could be encountered anywhere in the world. Here, for simplicity the most notable collapsible deposits found in each continent are detailed as follows:

1. **In Africa** the main deposits are:
   a. The collapsible aeolian silts (loess) and fine to medium sands of the Saharan fringe in the north.
   b. Granitic sands (residual) and Kalahari sands (aeolian) of South Africa.
   c. The cemented soils of Nigeria, in the middle.

2. **In Asia** deposits having collapsible structure are:
   a. Typical loess deposits in China; one of the main deposits in the world.
   b. Central Asian loess in Russia and Central Asian Republics.
   c. Volcanic ash of Japan and New Zealand.
   d. The saline soils of China.
   e. Some other small deposits of loessial soils have been encountered in India, Pakistan, Thailand, New Zealand, Iran, and Middle East.

3. **In Australia** the major collapsing soils are:
   a. Australian parna, an open-structured, carbonate-cemented sandy clay that collapses on wetting.
   b. Dune sands on the coast and in the desert areas.
   c. Mallee soils which are calcareous clays, silts and sandy silts of loessial type.
4. In **Europe** the main collapsible deposits are:
   a. Loess deposits in Romania and Bulgaria, originally transported by the Danube.
   b. Loess deposits in the eastern Europe (Russia and Ukraine); the most important deposits of the collapsing soils.
   c. Small deposits of loess found in France, Germany, Italy, Britain and Spain.
   d. Quick clay deposits of Scandinavia in the north.

5. In **North America** the primary collapsing soil deposits are:
   a. Loess deposits in the south and the middle of the continent.
   b. Quick clay deposits of Canada.
   c. Some small deposits of loess have been noticed in Canada and in other places of the continent.

6. In **South America** the main deposits of collapsible soils are:
   a. Loess deposits in Argentina.
   b. Volcanic dust deposits.
   c. Sand deposits in Brazil.

In addition to these natural deposits of collapsible soils, there are many artificial, or unnatural collapsible soils which could be found almost anywhere. These include:

1- Disturbed soil caused by slope instability, which can produce open structures.

2- Soil fills compacted on the dry side of the optimum moisture content.
Figure 2.4 shows a map of the distribution of collapsing soil deposits as presented by Kriger (1986). Kriger divided collapsible soils into loess and other types, mostly loose sand deposits which are relatively sparse. He considered only partially-saturated soil and thus ignored the open-structured quick clays, which have a very limited distribution in Eastern Canada and Scandinavia.

2.5 THE MAIN FEATURES OF A COLLAPSIBLE SOIL

The common properties of collapsible soils found in the literature were focused on partially saturated metastable soils. Rogers (1995) suggested a wider, definitive list of collapsible soil properties, from which it is possible to conclude that the primary features of collapsing soils are as follows:

1. They mostly consist of primary mineral particles ranging from sand-size or silt-size to clay-size. The shape of the silt-size particles is typical of the blade-shaped, angular or sub-angular particles described by Rogers and Smalley (1993).

The particle size and shape are the factors that control the way in which the soil is laid down in an open packing structure and can maintain this loose structure, either until cementation can withstand the forces involved or until a trigger causes collapse. The particles must be very small such that interparticle forces become significant in comparison with the force of gravity acting on the particles (see Figure, 2.2) Jefferson and Smalley (1995, 1997). Angular and sub-angular particles will more easily maintain an open, metastable structure, a good example of this being the blade-shaped particles of loess (see Figure 2.3). In contrast, a soil made up of uniform spherical particles will readily form
a close packing of relatively small void ratio and high density unless the particles are held apart by some means.

2. *They have an open structure of high void ratio and low density.*
   This property makes the soil easily undergo re-arrangement and volume reduction (structural collapse).

3. *They have a high sensitivity.*
   This property allows the saturated quick clay deposits to be included as collapsible soils.

4. *They are a geologically young or recently altered deposits.*
   This requirement derives from the idea that collapse can happen via natural processes, and that soils are continuously changing as they age and as the environment alters.

5. *They have a relatively low interparticle bond strength.*
   This feature refers to particles that exhibit weak and short-range bonds, the characteristics that cannot exist in classical clay soils. Based on the particle size property, the soil can be classified into clay, silt or sand. In this case these terms are indicators only of size and not of character. For example, the term quick clay was most probably derived to describe particle size rather than material behaviour.

6. *They are often partially saturated.*
   This condition specifically applies to unsaturated soils and was derived from the concept that the collapse trigger is most often wetting.
2.6 FACTORS AFFECTING SOIL COLLAPSE

The effects of various soil parameters on collapse characteristics and behaviour have been reported in several documents. The most important factors which affect the collapse phenomenon of soil are the initial dry density, initial water content or degree of saturation, the type and amount of binding materials (clay minerals or other bonding agents), the type of soil (shape, size and grading of particles), soil fabric and the level of applied stress. Several research programmes on collapsing soils have been conducted, such as those by Dudley (1970), El Sohby and Rabaa (1984), Lawton and Fragaszy (1989), Alwail (1990) and Basma and Tuncer (1992).

The effect of the initial dry density on the tendency of partially saturated soil to collapse when flooded was investigated by El Sohby and Rabba (1984) and by Basma and Tuncer (1992). According to their test results the initial dry density has a significant effect, soil collapse was found to be inversely proportional to the initial dry density as shown in Figures 2.5 and 2.6.

Rizkallah and Keese (1989) and Basma and Tuncer (1992) studied the significance of the initial water content on the collapse potential of partially saturated soil and concluded that the susceptibility of soil to collapse decreases as the initial water content increases (see Figure 2.7).

Dudley (1970) reported that the amount and rate of collapse appears to be affected by the type and quantity of clay minerals in the soil, maximum collapse occurring where the clay content is about 12%. Below 5% clay content there is little collapse, and above 30% the soil swells. Other researchers that have investigated the importance of the clay content on the
collapse behaviour of soil include El Sohby and Rabba (1984), Lawton (1986) and Alwail (1990). They concluded that:

1. The collapse potential is a maximum for clayey sand soils at a clay content between approximately 20% and 40% by weight, and for silt-clay soils between approximately 10% and 30% by weight, as shown in Figure 2.5
2. At lower clay contents silty soil collapses more than the sandy soil, while at higher clay contents silty soil swelled more.
3. In soils of higher clay content swelling of clay particles will reduce the amount of volume reduction, and an increase in vertical stress is consequently required for net reduction in volume.

Alwail (1990) reported that the shape of silt grains plays a significant role in determining the magnitude of collapse. More angular silt grains, with clay as a binding agent, result in a higher magnitude of collapse than more rounded silt with a lower clay content.

Basma and Tuncer (1992) studied the effect of soil type on the collapse potential, and concluded:

1. The soil type, identified by the grain size distribution, has a significant effect on the soil collapse as shown in Figures 2.8 and 2.9.
2. The soil collapse decreases as the difference between sand and clay percentages (s-c) increases.
3. Under similar conditions of loading as well as initial water content and dry density, well-graded soils tend to collapse more than poorly graded ones, as shown in Figure 2.9.

The effect of applied stress at wetting on the magnitude of collapse was investigated by El Sohby and Rabba (1984), Riani and Barbosa (1989) and Basma and Tuncer (1992). They concluded that:
1. The applied stress at wetting has a significant effect on the behaviour of soil.

2. The amount of soil collapse increases with the applied stress up to a certain point, beyond which no appreciable additional collapse occurs (see Figure 2.10). For sandy soils, however, collapse seems to increase continuously with applied stress, as shown in Figure 2.10.

2.7 PREDICTION OF SOIL COLLAPSE

A geotechnical engineer needs to be able to predict whether soils being dealt with are susceptible to collapse and to determine the amount and rate of the collapse that may occur. Once this is known, preventive measures can usually be taken to reduce or remove the possibility of damage to structures built on them. Several investigators have proposed various methods and important criteria to identify collapsing soils, and these will be briefly discussed in this section.

In order to be used easily and widely, the criteria should employ parameters which can be determined easily and relatively quickly, so that if the soil is found to be susceptible to collapse it can be investigated further with direct testing such as in the oedometer (Mellors, 1977). Thus simple routine tests can show whether or not the soil is susceptible to collapse, and the more complex tests, such as consolidation tests, will give an indication about the rate and magnitude of the collapse. However as the tests do not completely reproduce field conditions, correction factors based on experience must be applied to the test data in order to arrive at a computed collapse that is close to that which will occur in the field. For better evaluation of the engineering
properties of collapsible soils, different criteria have been established by different investigators and these can be classified into the following groups:

(i) Criteria based on fundamental properties of soil such as void ratio, Atterberg Limits, moisture content, degree of saturation, and dry density.

(ii) Criteria based on field tests.

(iii) Experimental methods, which include both qualitative as well as quantitative approaches.

Obviously the criteria based on the experimental methods are more reliable than other empirical approaches as they take into account the actual soil structure pattern which is so important for predicting collapse.

2.7.1 Criteria Based on Fundamental Properties of Soil

2.7.1.1 Expression Based on Voids Ratio Relationships

A simple but widely used criterion is that proposed by Denisov (1951), who states that collapse is probable when:

$$\frac{e_L}{e_o} < 1$$  \hspace{1cm} (Eqn. 2-1)

where

- $e_L$ = void ratio at the Liquid Limit
- $e_o$ = natural void ratio

This criterion was successfully used in predicting collapse in a wide range of partially saturated soils, including loess (Holtz and Hilf, 1961). It is interesting to note that this criterion is similar in nature to the well-known characteristics of the saturated collapsible "quick clay", in which the natural water content is usually higher than the Liquid Limit (Skempton and Northey, 1953).
A soil is considered to be susceptible to collapse upon wetting according to the Russian Building Code (Markin, 1969), if the in-situ degree of saturation ($S_r$) is less than 60% and:

$$\frac{e_0 - e_L}{1 + e_0} > -0.1$$  \hspace{1cm} (Eqn. 2-2)

The soil is considered to tend to swelling upon wetting if the value of the above ratio is less than -0.3 (Danilov, 1964). These criteria were used successfully by Feda (1966) for some loessial soils to estimate their collapsing potential.

It can be shown that the above criteria can be rewritten as:

$$\frac{(\rho_o)_d}{(\rho_L)_d} < 1.1 \text{ where there is a tendency to collapse}$$  \hspace{1cm} (Eqn. 2-3)

where

$$(\rho_o)_d = \text{in-situ dry density}$$

$$(\rho_L)_d = \text{dry density at the Liquid Limit}.$$

Other investigators consider soil to be susceptible to collapse when the above ratio is less than unity (eg. Gibbs and Bara, 1967).

It can be noticed that these criteria have only taken into consideration the effect of void ratio on the collapse phenomenon upon wetting, and not the effects of applied stress level, soil mineralogy, type of bond or cementation, and the nature of grain to grain contact.

2.7.1.2 Expression Based on Moisture Content and Atterberg Limits Relationships

Denisov's (1991) criterion has been slightly improved by Priklonskij (1952),
who suggested the following criterion based on the collapsibility index (K) where:

\[ K = \frac{LL-w_o}{PI} = \frac{LL-w_o}{LL-PL} \]  
(Eqn. 2-4)

and

\[ LL = \text{Liquid Limit} \]
\[ w_o = \text{natural water content} \]
\[ PI = \text{plasticity index} \]
\[ PL = \text{Plastic Limit} \]

The following limits were proposed by Prikloonskij:

- \( K < 0 \) for highly collapsible soils
- \( K > +0.5 \) for non-collapsing soils
- \( K > +1.0 \) for swelling soils.

The criterion presented by Gibbs and Bara (1962) and also used by Handy (1973) has shown that when the saturation water content exceeds the Liquid Limit, collapse of the soil structure from an increase in the applied stress may occur. This criterion has been expressed in terms of the Liquidity Index (LI) defined as:

\[ LI = \frac{w_o-PL}{LL-PL} \]  
(Eqn. 2-5)

Thus, when the Liquidity Index approaches or exceeds a value of unity, experience has shown that collapse of soil may occur. It is clear that the equations 2-4 and 2-5 are similar.

Feda (1966) developed the Liquidity Index criterion, stating that collapse is probable when the collapsibility index \( K < 0.85 \) where:
where

\[ S_0 = \text{the natural degree of saturation} \]

Feda also reported that collapse should be expected upon wetting of soils with a degree of saturation \( (S_0) \) less than or equal to 60%. He also indicated that loessial soils whose natural porosity \( (n_0) \) exceeds 40% are susceptible to collapse. This limiting value of porosity is very close to the maximum porosity \( (n_{\text{max}} = 47.6\%) \) of loosely deposited spheres in cubic packing. Feda demonstrated the effect of this porosity criterion by pointing out the negative dilatancy (i.e., compression) experienced by sand samples when sheared at initial porosities approaching 40%. Since higher porosity is accompanied by a lower dry densities, a highly porous soil having low natural dry density and low natural water content is very likely to experience structural collapse. For comparison between Denisov's criterion and Feda's criterion, the collapsibility index \( K \) can be rewritten in the form:

\[
K = \frac{(e_0 - e_p)}{(e_L - e_p)}
\]

(Eqn. 2-7)

Where \( e_p = \text{voids ratio at the Plastic Limit} \)

When \( e_0 = e_L \) then \( K = 1 \). Therefore Feda's criterion is more conservative.

2.7.1.3 Expressions Based on Density and Atterberg Limits Relationships

Based on Denisov's relationship between the natural void ratio and that at the Liquid Limit, Gibbs and Bara (1962) developed a new criterion based on the premise that a soil which has sufficient void spaces to hold its liquid limit
moisture content at saturation, is susceptible to collapse on wetting. This criterion is based on the collapse ratio (R), where:

\[ R = \left( \frac{W_S}{W_L} \right) = \frac{\left[ \frac{(\rho_w/\rho_d) - (1/G_s)}{W_L} \right]}{W_S} \]  

(Eqn. 2-8)

where

\[ W_S = \frac{\rho_w}{\rho_d} - \frac{1}{G_s} \]

and

\[ W_S = \] water content at 100% degree of saturation
\[ W_L = \] water content at Liquid Limit
\[ \rho_w = \] density of water
\[ \rho_d = \] dry density of soil
\[ G_s = \] specific gravity of solid particles

The soil would be susceptible to collapse on saturation when the collapse ratio (R) is greater than or equal to unity. On the basis of this expression, a relationship between dry density and Liquid Limit was created, and a graphical criterion to predict soil collapse was established (see Figure 2.11). The curves on the graph represent the condition at which the void ratio for a particular dry density equals to the void ratio at the Liquid Limit. The chart is divided into two regions by lines drawn at 100 percent degree of saturation for soils with specific gravities of 2.60 and 2.70. Soils which plot above the line of saturation are potentially collapsible, that is they will undergo collapse on wetting, while those which plot below the line behave as stable or heaving soils on wetting. This graphical criterion was successfully applied in predicting subsidence at a number of dam and canal sites on loess foundations in the United States and at sites underlain by low density alluvial deposits along the line of the San Luis Canal in the San Joaquin Valley, California (Gibbs and Bara 1962).
An extensive investigation has been carried out by Anderson (1968), in which two hundred randomly chosen samples were tested. Based on the results he attempted to find a correlation between the collapse ratio (R) and the basic parameters of the soil by using a computerized step-wise regression programme. The following equation was generated:

\[
R = 5.5 - 3.82 \log (\frac{LL}{PL}) - 1.63 \log (PL) - 1.24 \log \left(\frac{D_{60}}{D_{40}}\right) \\
- 0.918 \log P_{10} + 0.465 \log \left(\frac{D_{60}}{D_{40}}\right) \\
- 0.45 \log \left(\frac{D_{100}}{D_{50}}\right) - 0.303 (P_{200})
\]  
(Eqn. 2-9)

where

\[
\begin{align*}
C_u & = \text{uniformity coefficient (D}_{60}/D_{10}) \\
D_{10} & = \text{particle size corresponding to 10\% passing}.
\end{align*}
\]
\[
\begin{align*}
D_{40} & = \text{" " " " 40\% "} \\
D_{50} & = \text{" " " " 50\% "} \\
D_{60} & = \text{" " " " 60\% "} \\
D_{100} & = \text{" " " " 100\% "} \\
P_{10} & = \text{fraction passing No. 10 sieve} \\
P_{200} & = \text{fraction passing No. 200 sieve}
\end{align*}
\]

2.7.2 Criteria Based on Field Tests

Most of the proposed criteria for detecting collapsible soils in the field are site specific. Several field tests have been developed by various researchers.

A simple field test proposed by Jennings (1975) to identify potentially collapsible soils is reported by Knight and Dehlen (1963). This test uses a cylinder 100mm high and 50mm in diameter to cut into the undisturbed soil. The sample was split into two 50mm long portions. One portion was wetted,
moulded by hand, then placed into the cylinder. Collapse may be suspected if the height of the wetted and remoulded soil was less than the unwetted, undisturbed portion. Jennings and Knight (1975) proposed the "sausage" test to identify potentially collapsible soils in the field. The test is carried out as follows: a hand-size sample of the soil is taken from the site and divided into two equal pieces. One piece is wetted and moulded by hand to form a ball. If the volume of the wetted piece is smaller than that of the unwetted one, then collapse can be suspected. The unwetted piece is then moistened to form a ball. If the two wetted balls were equal in volume, then the original unwetted pieces were equal in mass and the test can be considered accurate. These field criteria have the advantages of simplicity and immediacy of the results via qualitative evaluation, but they are not adequate as an engineering tool other than in alerting the engineer to a potential problem worthy for further investigation.

It should be made clear, however, that most of the criteria, and the associated values of the parameters indicating susceptibility to collapse, are based on the observations of the various authors. Furthermore, they very often impose limitations to their criteria. For example, Denisov's criterion is only strictly applicable to cases where wetting of the soil occurs at low stresses. Feda's method is only applicable when the natural degree of saturation is much less than unity and the porosity must be at least 40% (Fookes and Best 1969, and Mellor 1977). Most of the prediction criteria cited above are qualitative methods, that is they do not provide any magnitude or rate of the expected collapse. They have been derived on the basis of the effect of void ratio (porosity), LL, and degree of saturation only, without any consideration to the effect of the applied stress level, type and amount of cementing materials, particle packing (the nature of the grain to grain contact), the soil composition
and its formational origin. The following are the methods which provide quantitative answers regarding the rate and magnitude of soil collapse.

2.7.3 Criteria Based on Experimental Methods

The use of experimental methods will give not only a qualitative determination of the possibilities of collapse but also quantitative information to be used in estimation of the magnitude and rate of the collapse. Under the proper conditions, many types of soil will densify when they get free access to water. The most critical conditions are the initial water content, the dry density and the applied stresses. To accommodate these variables, collapse tests are typically conducted utilizing standard oedometer or triaxial equipment. In both types of laboratory test, the soil specimen is provided with free access to water after equilibrium conditions are reached. In oedometer tests, the sample is submerged; in triaxial tests, water is typically introduced from the bottom of the sample under a small positive head. Two loading-wetting sequences have been used in the laboratory to study the collapse phenomenon. The first method consists of loading the soil specimen incrementally to a specific state of stress, allowing the sample to come to equilibrium under the applied stresses, and then flooding the specimen by water. This soaked-after-loading method has been used in both one-dimensional analyses using oedometer, which is known as the single-oedometer collapse test, and three-dimensional analyses using triaxial equipment (Lawton, 1992). A second load-wetting sequence is known as the double-oedometer procedure (Jennings and Knight, 1957), and is based on the assumption that the deformations induced by wetting are independent of the loading-wetting sequence. In this method, two oedometer tests are performed using nominally identical specimens. In one test, the specimen remains at its initial water content; in the other test, the
specimen, under a small applied stress, is first inundated with water and allowed to come to an equilibrium. The loading sequence is identical for both specimens and the difference in void ratio between the two specimens at any vertical stress is assumed to represent the volume change that would occur in the specimen if it was loaded to that level of stress and then soaked. This technique has recently been extended to three-dimensional testing (Lawton et al, 1991) and has been termed the double-triaxial test. An advantage of this procedure is that it allows a determination of collapsibility under any level of stress within the range tested, instead of at a single stress; a disadvantage of this method concerns the effect of sample variability, since tests are performed on different samples. Lutenegger and Hallberg (1988) stated that the most common method used to identify a collapsible soil material, and the one with the most meaning to geotechnical engineers has been the one-dimensional compression test using an oedometer. The test procedure involved applying a vertical stress to a laterally confined sample of soil at a natural moisture content, and then introducing water to the specimen. A decrease in volume at constant vertical stress is an indication of the collapse of the material. The coefficient of collapse, i, as defined by Lutenegger and Hallberg is often determined for a stress of 300 kN/m² and is given by

\[ i = \frac{\Delta e}{1 + e'} = \frac{e' - e''}{1 + e'} \]  
(Eqn. 2-10)

where

\( \Delta e \) = decrease in void ratio caused by wetting
\( e' \) = void ratio before wetting
\( e'' \) = void ratio after wetting.

Generally, values of \( i \) greater than 0.02 are indicative of soils considered dangerous with respect to collapse.
Lawton et al. (1992) stated that the few published comparisons between the two methods (single and double oedometer tests) generally indicate only small differences in the predicted magnitudes of collapse. Booth (1977) reported that the double-oedometer technique over-predicted the amount of collapse by about 10%. Justo et al. (1984) studied the influence of loading-wetting sequence on the wetting-induced volume change of soils and determined that in the collapse region the volume change was essentially independent of the loading-wetting sequence. Lawton et al. (1989) came to similar conclusions for the slightly expansive clayey sand they studied. Lawton (1992) recommended that double-oedometer or double-triaxial tests be used only after verification tests have been conducted for the soil and conditions to be evaluated.

Based on data obtained by conducting a series of oedometer tests a multiple regression analysis was performed by Basma and Tuncer (1992) to produce a prediction collapse potential model. The best-fit equation obtained was:

\[
CP = 48.496 + 0.102 \ C_U - 0.457 \ W_i - 3.533 \ \gamma_d + 2.8 \ \ln (P_w) \quad \text{(Eqn. 2-11)}
\]

where \(CP\) = the collapse potential in percent
\(C_U\) = the coefficient of uniformity of the soil
\(W_i\) = the initial water content in percent
\(\gamma_d\) = the initial dry unit weight in kN/m\(^3\)
\(P_w\) = the pressure at wetting in kPa

The correlation coefficient \((r^2)\) and the standard error of estimate (SEE) of this equation are 0.94 and 1.94 respectively. The solutions of this model are presented in a nomographic chart as shown in Figure 2.12. The purposes of this chart are:
1. the straightforward determination of the collapse potential (CP) for different soils under various conditions (dashed line 1);
2. to determine the critical initial water content above which no collapse is expected (dashed line 2); and
3. to estimate the critical applied pressure below which the soil will not collapse (dashed line 3).

2.8 MECHANISMS OF COLLAPSE

Generally, there are four factors necessary for collapse to occur in a partially saturated soil (Barden et al 1973, Lawton and Fragaszy 1992 and Mitchell 1993).
2. A high enough total stress that the structure is metastable.
3. A bonding or cementing agent that stabilises the soil in the partially saturated condition.
4. The addition of water to the soil, which causes the bonding or cementing agent to be weakened and the interaggregate or intergranular contacts to fail in shear, resulting in a reduction in total volume of the soil mass.

Houston et al (1988) reported that Casagrande (1932) hypothesized a conceptual model for the particle arrangements of collapsible loose silty sand. In this model, the fines consolidate and gain strength under the effect of overburden stress, and they act as a binding agent between sand grains. The addition of stress to the soil at its natural water content causes a slight compression of the fines with no significant movement of soil particles. This
phenomenon is shown in Figure 2.13(a). If water is added to the soil until it reaches a certain water content, the fines bridges can no longer resist the local micro-shear forces at the sand particle interface and therefore the soil structure collapses, as shown in Figure 2.13(b).

Dudley (1970) indicated that the grains of collapsible soils are held in place by a force and/or a material. As the soil dries below the shrinkage limit, menisci are formed which result in the pore water pressure becoming negative, causing the effective stress to be greater than the total stress. The addition of water to the soils removes this suction force, and thereby the grains are able to slide (shear) on one another moving into vacant spaces. In sandy soils with a silt binder, the capillary forces are applied around the silt to silt contacts and silt to sand contacts. In sandy soils with clay binder, the resistance to consolidation caused by the presence of the clay buttresses is a function of salt concentration of the water, void ratio of the clay structure, and water temperature. He added, however, that the capillary tension is the principal factor contributing to the temporary strength of the soil.

An intensive microscopic study on the collapse mechanism of partly saturated soils collected from different parts of the world was conducted by Barden et al (1973). This study has been done by using the scanning electron microscope (SEM) on air-dried specimens of collapsing soils representative of various types of soil. The soils investigated include aeolian sand from South Africa, loess from Europe, clay from America and a range of compacted soils. The investigation was based on the idea of considering two components of effective stress: one due to the applied stress, and the second due to the suction forces. These two components develop intergranular stress by different mechanisms; thus the applied stress develops shear stresses and hence potential instability at intergranular contacts, while the suction is strictly a normal stress and hence
increases the stability at intergranular contacts. Collapse requires the following three conditions:

1. An open, partially saturated, unstable structure.
2. A high enough applied stress.
3. A high enough value of bonding strength to stabilize intergranular contacts.

Barden et al. suggested that the basic concept is of an open structure, often of bulky-shaped grains held together by some bonding materials or force. Three types of bond are postulated as follows.

1. For soil that consists of sand with silt as binder, the silt-silt and silt-sand contacts are assumed to be bonded by simple capillary forces, as shown in Figure 2.14(a).
2. Most collapsing soils, however, involve the action of clay plates in the bonds between the bulky sand and silt grains. Several structural arrangements of the clay plates are possible, depending on the geological origins and history of the soil. When the clay is formed in place it could form a parallel plate onion-skin effect around the sand or silt particles, as illustrated in Figure 2.14(b). If the clay was originally in suspension, then gradual evaporation would cause the clay colloids to withdraw into the narrow spaces at interparticle contacts. In this case the clay would form a random flocculated structure, giving a buttress type of support to the bulky sand or silt grains as illustrated in Figure 2.13(c).
3. In some collapsible soils the bonding effect may be due to chemical cementing agents such as iron oxide, calcium carbonate, and some salts.

The collapsible volcanic soils of low bulk density have been studied by Uriel.
and Serrano (1973), who have adopted the classical structural engineering approach to study the collapse problem. They considered collapsible soils in general and produced an interesting, diagramatic soil structure as shown in Figure 2.15. The open structure of the cemented soil is schematically simulated, each rectangle symbolizing a solid particles of the soil fabric, and the hatched strips representing the bonding material. Uriel and Serrano (1973) concluded that the structural collapse of the porous cemented soil can be produced by tensile stress at the bond due to bending, by compressive stress, and by shear stresses.

Clemence and Finbarr (1981) described the fabric of natural collapsible soils and discussed the causes of the phenomena. Typical fabric and binding agents of collapsible soils are shown in Figure 2.16. For partially saturated soils, consisting of sand with silt as fines, the capillary suction increases the apparent strength. The addition of water reduces this strength. If the soil is porous, a rapid decrease in volume will occur. For soils with clay fines, the arrangement of the clay particles depends on the history of the soil. For example, Figure 2.16(c) shows clay plates forming an onion-like structure, and Figure 2.16(d) shows clay buttresses. The onion-like structure indicates that clay formed in-place, whereas the clay buttresses indicate a gradual evaporation of the water in the soil pores. Clemence and Finbarr (1981) added that the contribution of each capillary effect and electrochemical effect on the nature of the clay bond is not understood. It has been observed in most cases, however, that the bond strength increases as the water content decreases.

Recently, the scanning electronic microscope (SEM) has been used to investigate the fabric of collapsible loess deposits by different workers in various parts of the world. Gao (1988) used the SEM to determine the development of the structures of collapsing loess soil in China. A relationship
between the regional variation of loess microstructure and the climatic condition and geological environment was found. The SEM revealed that loess in arid regions was still developing and had high collapsibility potential. The loess in the humid regions, on the other hand, possesses less collapse potential due to the cementation strength between grains.

Derbyshire and Mellors (1988) compared the fabric of loess soils in China and Britain using the SEM. It was found that clay size particles and true clay minerals were distributed in three major forms:
1. as coatings to the grain: the coatings were discontinuous, of non-uniform thickness, and laid parallel to the surface of the silt particles;
2. as "clay bridges" between silt grains: these bridges consist of face-to-face aggregation of clays peeling away from the surface of one grain and linking up with clays on the surface of an adjacent grain;
3. as buttresses comprising silt-sized aggregations of face-to-face clay particles between quartz grains.

Alwail (1990) studied the effect of fine material on the collapse mechanism of remoulded sandy soil specimens. The change in the soil fabric due to collapse was examined qualitatively by the SEM and quantitatively by the air permeability technique. He concluded that the dominant mechanism in the collapse of sandy soils with fines is the disintegration of clay macropeds (clay balls) into either a continuous or discontinuous clay blanket, depending on the quantity of the clay. There is no change in the clay micro-structure due to collapse unless the quantity of the clay in the soil is at least 55%.

Lawton et al (1992) reported that the collapse in partially saturated soils occurs in a different manner, as was first proposed by Jennings and Burland (1962) and later verified using a scanning electronic microscope by Barden et al.
(1973) and Booth (1977), and refined by Hodek and Lovell (1979). In this model of collapse, the soil is assumed to consist of a combination of brittle coarse grains and aggregations of clay minerals that may be either brittle or plastic, depending on their moisture condition. The collapse characteristics and behaviour of the soil mass depend primarily on the distribution and size of the interaggregate (pores inbetween the aggregates) and intra-aggregate (pores inside the aggregates themselves) pores. The interaggregate pores are considerably larger than the intra-aggregate pores and can be affected during the loading processes, whereas the intra-aggregate voids are relatively very small and little affected by load. The aggregates in a dry condition are brittle and shrunken. With the addition of water, the aggregates increase in volume and soften with the absorption of water to the clay particles. According to the initial dry density and loading conditions, collapse can occur as the softened plastic aggregates are remoulded at the aggregate-granular or aggregate-aggregate contacts and the soil is rearranged into a denser state of packing.

2.9 SUMMARY

* Collapse is a change in soil structure from unstable or metastable to more stable; the structural parameter defines a collapsing soil. Then soil collapse can be defined as a rearrangement in the major structural units from an open packing metastable to more stable, closer packing with very important geotechnical consequences. Mostly structural collapse is caused by loading and wetting in unsaturated materials, although high moisture content and fully saturated systems such as quickclays and submarine sands can be classified as collapsing soils.
The major type of collapsing soil is the unsaturated group, of which loess deposits are the most important materials. Loess consists mainly of silt-size grains, which are loosely deposited by air and bonded together by cementing material such as clay minerals and calcium carbonates. The properties of loess are unique and deserve special consideration. It is widespread in many parts of Asia, Europe, U.S.A., Russia and Africa.

Simple routine tests may show whether or not the soil is susceptible to collapse and the necessity for further investigation. The more advanced tests, such as the hydro-consolidation test, will give an indication of the magnitude and rate of soil collapse. Both oedometer and triaxial laboratory experiments have successfully been used to evaluate soil collapse. The single and double-oedometer testing methods have both been shown to be reliable predictors of collapse potential, similar results having been obtained by the two techniques.

An increased awareness of the existence of collapsible soil and a recognition of the engineering problems associated with these types of soils are very important objectives. There is a great need for experimental investigations and field studies which would provide precise quantitative information that may present greater understanding of the many uncertainties involving the collapse phenomenon.
Figure 2.1 The Main Types of Collapsible Soils. (Modified after Rogers, 1995)
Fig. 2.2 The R-Size Diagram: Bond/Weight Ratio Versus Nominal Diameter of Soil Particles, (after Jefferson and Smalley, 1997)
Fig. 2.3 The Shape of Chinese (Malan) Loess Particles
Fig. 2.4 Distribution of Collapsible Soils, (after Kriger, 1986)
Fig. 2.5 Effect of Dry Density, Clay Content and Applied Stress on Deformation of Unsaturated Soils upon Wetting.
(after Elsohby and Rabban, 1984)
Fig. 2.6 Effect of Initial Dry Unit Weight on Collapse Potential.
(after Basma and Tuncer, 1992)

Fig. 2.7 Effect of Initial Water Content on Collapse Potential.
(after Basma and Tuncer, 1992)
Fig. 2.8 Effect of Difference between Sand and Clay on Collapse Potential at Various Wetting Pressures (after Basma and Tuncer, 1992)

Fig. 2.9 Effect of Coefficient of Uniformity on Collapse Potential at Various Wetting Pressures. (after Basma and Tuncer, 1992)
Fig. 2.10 Effect of Stress at Wetting on Collapse Potential (after Basma and Tuncer, 1992)

Fig. 2.11 Stable and Metastable (Collapsible) Soil. (after Gibbs and Bara, 1962)
Fig. 2.12 Nomograph to Determine Collapse Potential
(after Basma and Tuncer, 1992)

Fig. 2.13 The Change in Soil Particles Arrangement for Natural Loose Silty Sand Suggested by Casagrand (1932).
(from Houston et al., 1988)
Fig. 2.14 Fabric of Natural Collapsible Soils Suggested by Barden et al., (1973). (after Barden et al., 1973)
Each rectangle symbolizes a major structural particle, and the hatched strips representing the bonding-cementing material.

Fig. 2.15 Structural Model for Collapsible Soil, as Proposed by Uriel and Serrano (1973).
Fig. 2.16 Typical Structure for Natural Collapsible Soils. (after Clemence and Finbarr, 1981)
CHAPTER 3
FORMATION AND GEOTECHNICAL PROPERTIES OF LOESS

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3.6 SUMMARY
CHAPTER 3  
FORMATION AND GEOTECHNICAL PROPERTIES OF LOESS  

3.1 INTRODUCTION  

The term loess (in English) was first used by Lyell (1834) to describe friable, silty material utilized in the brickmaking industry in the Rhine Valley in Germany. There have been numerous arguments relating to the origin, formation and constitution of loess. Charlesworth (1957) described loess as "by far the most important periglacial accumulation". Flint (1957) observed that loess is "one of the most remarkable of the pleistocene deposits, around which an extensive literature accompanied by much controversy has accumulated". 

Smalley (1975) stated that to define loess is not easy, because there is still considerable discussion about what constitutes its truly characteristic features. Pye (1987) reported that there have been many attempts to define loess on the basis of its physical and mineralogical properties, but none has proved entirely satisfactory. Many earlier workers described loess as a uniform, yellow, calcareous silt, or unstratified, homogeneous, calcareous silt, light yellow or buff in colour". However, more recent work has shown that loess is not always calcareous, it may be clayey or sandy, and the colour varies considerably; dark grey, yellow, brown, white and red loess deposits are known, although buff is most common. A great deal of effort has been expended in attempts to distinguish true aeolian loess from loess-like sediments. The latter have been variously designated by such terms as river loess, lake loess, marine loess, loess loam, alluvial loess, proluvial loess, colluvial loess and deluvial loess.
The difference between loess and loess-like deposits has been described by many investigators. In some cases the loess-like deposits represent wind-deposited dust which has been re-deposited by water or slope processes, while in other cases aeolian transport appears to have played little or no part in the formation of the deposits. Partly due to the way in which terminology has been employed, confusion has arisen regarding the origin of fine-grained deposits in different areas. In an attempt to clarify the situation, Pye (1984) proposed a series of simplified definitions which are adopted here as follows.

Loess is defined simply as a windblown silt deposit consisting mainly of quartz, feldspar, mica, clay minerals and carbonate grains in varying proportions. In a fresh (i.e. unweathered) state, loess is typically homogeneous, non- or weakly-stratified and highly porous. When the loess deposit is dry, it has the ability to stand in vertical sections and sometimes shows a tendency to fracture along systems of vertical joints, but when saturated with water the shear strength is greatly reduced and the material is subject to hydro-collapse, subsidence, and sliding. The grain-size distribution of typical loess shows a pronounced mode in the range 20-40μm, and it is negatively skewed (i.e. towards the finer sizes).

Typical loess often contains 10% to 20% clay minerals and up to 10% fine sand (> 63μm). In cases where the clay content exceeds 20% the material can be described as clayey loess, and if the deposit contains more than 20% fine sand, the term sandy loess can be used (Pye, 1987).

Reworked loess consists of primary wind-deposited material which has been eroded and re-deposited by running water and/or slope processes at some considerable distance from its original site of deposition. Reworked loess usually accumulates in valley bottoms, lakes and on river terraces.
Weathered loess is primary loess whose sedimentary characteristics have been markedly modified by weathering and soil formation. Weathered loess is usually decalcified and contains more clay minerals (up to 60%) than unweathered loess. Some deposits are composed of mixtures of dust material and other sediments, such as fine sand or alluvial fill, and they are described as a loess-soil. Loess-like deposits are sediments which possess many of the sedimentological properties of aeolian loess. They include overbank silts, lacustrine silts and some colluvial deposits. Differentiation of such deposits from reworked loess is often difficult, but may be possible on the basis of geochemical, mineralogical, and textural criteria.

Russell (1944) reported that loess is unstratified, homogeneous, porous, calcareous silt; it is characteristic that it is yellowish or buff, tends to split along vertical joints, maintains steep faces, and ordinarily contains concretions, and snail shells. From the quantitative standpoint at least 50 percent by weight must fall within the grain size fraction 0.01 - 0.05 μm, and it must effervesce freely with diluted hydrochloric acid.

Flint (1957) has defined loess as a sediment, commonly non-stratified and commonly unconsolidated, composed dominantly of silt-size particles, ordinarily with accessory clay and sand, and deposited primarily by wind. Smith and Norton (1935) stated that loess is the sediment which is believed to have been produced by the grinding action of glaciers, deposited in the lower lying areas from streams carrying water from the melting glaciers and then picked up and redeposited on the uplands by the wind.

Smalley and Vita-Finzi (1968) defined the loess as a clastic deposit which consists predominantly of quartz particles 20-50 microns in diameter and
which occurs as wind-laid sheets. Pettijohn (1957) defined loess as an unconsolidated porous silt, commonly buff in colour (locally grey, yellow, brown or red), characterized by its lack of stratification and remarkable ability to stand in a vertical slope. It commonly shows a crude columnar structure. It is generally highly calcareous and effervescent in weak acid. Loess is essentially a silt. Table 3.1 summarizes the loess definitions which have been provided by the Loess Commission of INQUA (the International Union for Quaternary Research).

Loess is friable material and easily eroded by rivers. It is a wind-blown deposit and has accumulated over many parts of the world during the last two to three million years. It is mainly of silt-size particles and characterized by a lack of stratification and uniform grading and occurs as a blanket deposit (Derbyshire and Mellors, 1987). It is reported that some deposits of loess are of alluvial origin (e.g. in Poland, see Grabowska-Olszewska, 1988). This type of loess deposit is stratified. The morphology of the surface of loess sheets is strongly controlled by the underlying topography. Loess deposits normally form a blanket of different thicknesses, covering a variety of relief features including steep valley slopes, terraces, plateaux, and alluvial fans. Loess which has been eroded from the upper slopes often forms weakly stratified accumulations on the lower slopes.

Mineralogically, the dominant mineral in loess is quartz (50% - 80% by weight), the silt-sized grains of which are angular or sub-angular and occasionally sub-rounded (Embleton and King, 1968). The other major constituents are feldspars (up to 25% by weight), clay minerals and carbonates. The amount of heavy minerals is somewhat variable, but does not exceed a few per cent (Kukla, 1970). Other minerals that may be present are micas, pyroxenes and hornblende.
Calcium carbonate can occur as discrete grains, coatings to quartz grains and also nodular concretions. These concretions, known as loess-dolls, are usually hollow inside and fissured and are generally small (up to 60 mm diameter), although larger samples have been found. They are thought to have been formed by percolating groundwater charged with calcium carbonate precipitating the calcite around a quartz nucleus (Charlesworth, 1957).

The content of organic matter is generally low, consisting typically of 0.2% - 0.3% organic carbon although larger amounts are sometimes accumulated in darker layers which are interpreted as buried soil horizons (paleosols). Typically loess cliffs have vertical faces and a vertical sub-columnar jointing, as illustrated in Figure 3.1. Scheidig (1934) and Smalley (1966a) attribute this jointing to contraction cracking and Flint (1957) stated that the degree of vertical fracturing increases where the dominant grain-size lies near the fine end of the 10 μm - 50 μm particle size range. Another general characteristic of loess is the fossil faunal content, which usually consists of an abundance of terrestrial fauna.

3.2 FORMATION OF LOESS DEPOSITS

Smalley (1995) in quoting Hardcastle (1890) stated that “four factors are required for the production of a massive formation of dust material. These factors are:

1. a source of wind-borne dust;
2. winds to transport the dust;
3. vegetation to entrap the dust;
4. sufficient time for its accumulation.
Hardcastle said that the dust was "rock-meal", produced by the great ice mill, and spread”. This was possibly the first statement on the formation of loess material, although there had been much discussion in the nineteenth century of how loess deposits formed. It is now widely accepted that glacial grinding produces large amounts of silt material and that a considerable proportion of this material is transported and deposited by wind as loess sediments. The importance of this ice-sheet loess was emphasised by Smalley (1966a), but it has become apparent in the last thirty years that there are other loess particle forming mechanisms that need to be considered. There are other sources of energy in nature that can produce fine particles which may go on to form loess deposits.

The origin and formation of loess deposits have therefore been controversial topics of discussion for one hundred and fifty years, but it is now generally accepted that true loess has resulted from the aeolian transportation and subsequent deposition of silty material. The principal processes responsible for the production of the loess particles appear to be subglacial grinding by both ice sheets and glaciers, frost cracking in periglacial climates, breakage by salt crystal growth and hydration in desert regions, fluvial transport impact and aeolian impact of sand grains (Smalley, 1966a; Pye, 1987). Loess material (silt-size particles) are therefore produced by several different processes in a wide range of environments. The key requirements for a high rate of aeolian dust transport are:

1. the existence of unstable geomorphic surfaces composed of poorly sorted sediments with a high silt/clay ratio; and
2. a relatively high frequency of strong, turbulent winds.

Smalley et al (1983) reported that the formation processes of loess deposits consist of four main stages:
formation of loess-size particles (predominantly 20-60μm); transportation of loess material; deposition of this material; and post-depositional modification.

In simple cases, loess particles have been blown by the wind directly from the area where they were formed to the deposition place; in other cases silt has been produced, transported and concentrated by glacial and/or fluvial processes before being blown by the wind. In very complex situations more than one phase of fluvial transport and aeolian transport may have been involved. Smalley (1980) and Smalley and Smalley (1983) have discussed possible event sequences responsible for the formation of some of the world's major loess deposits. Figure 3.2 shows the events sequence in the formation of loess sediments.

Loess is a consequence of energetic processes of weathering and grinding of rocks. Large quantities of loess particles and the eventual formation of major loess deposits can be produced by the action of a large scale energy. The major energetic events can be identified as continental glaciations and tectonic processes, which are responsible for the large loess deposits. Smalley (1995) stated that any high energy process which can release a silt particle from a rock or breaking an existing sand grain can form loess silt. The more powerful processes will produce more loess. Wright and Smith (1993) have compared some processes and shown that there is a wide range of possibilities for producing some silt. Obviously the more powerful the process the greater the amount of loess silt produced. Smalley (1966a) proposed that glacial grinding provided loess material for the bulk of the world's loess deposits. Where there were large continental glaciers in the cold phases of the pleistocene there are related loess deposits. There appears to be a direct correlation between ice volume and loess formation. This is probably a mechanistic connection - the
large ice volume in continental glaciers allows effective glacial grinding to take place and continental rocks are ground to loess size. Glacial grinding does not, however, represent the origin of all loess deposits. The loess of China for example is silty material which was removed by wind from desert basins, wadis and plazas which lie to the north and west. They probably still contribute wind-blown material to the loess belt of China. The collapsible silty material found in the arid south-west of the United States also continues to accumulate at present and has no glacial association. Frost weathering has been shown by experimental investigations carried out over the last fifteen years to be an important mechanism for supplying loess material. These experiments have recently been reviewed by Lautridou and Ozouf (1982), who concluded that loess material can be produced in the required amounts by frost action. Thus cold weathering processes must be considered as a major method for producing loess materials. In conclusion, therefore, glacial grinding accounts quite well for most of the North American loess material, and probably most of the European, but presumably contributes little to the Chinese deposits and possibly not much to the South American deposits.

Whalley et al (1982) propose sand grain impact as the formation mechanism. They collected the products of sand abrasion experiments and observed the presence of loess sized particles. They concluded that an aeolian abrasional origin for some loessic material seemed possible, and in particular that the Gobi desert could supply material for the Chinese loess. Their abrasion experiments indicated significantly different results from those of Kuenen (1960) who found that loess-sized material was not produced. Nahon and Trompette (1982) suggested that chemical weathering, particularly in tropical areas, is an active agent of silt formation. Goudie et al (1979) also proposed a tropical and chemical mechanism - salt weathering by sodium sulphate - and
their experiments did both produce sand grain breakage and hence indicate a possible source of some loess-sized particles (Pye and Sperling, 1983).

More knowledge of the actual particle forming mechanism is required, and there are other high energy environments that should be investigated. For example, Palmer (1982) has shown that high energy rivers associated with deformable rocks can produce loess-size material and it seems likely that in New Zealand, a country of steep rivers, a reasonable proportion of loess material comes from this source. Thus the earlier generalization are becoming more refined and are being incorporated into comprehensive event sequence models for the formation of specific deposits. In addition transportation needs further consideration since it plays a very important role in the formation of loess deposits. Although the role of aeolian transportation is very important, it tends to be overstated in much loess discussion; the aeolian transportation stage is a short final stage. The loess particles are carried in suspension, but only for short distances since the wind is more effective at lifting loess-sized particles than carrying them (Smalley, 1995).

3.3 DISTRIBUTION OF LOESS DEPOSITS

Loess is a very widespread surface soil deposit. It occurs on all continents. About 10% of the surface area of the earth is covered by loess and loess-like deposits, although primary aeolian loess covers only about 5% of the world's land surface area (Pecsi, 1968; Pye, 1987). The most extensive loess deposits occur in China, Russia, North America, Europe, Ukraine and Argentina. Figure 3.3 presents the main regions of loess deposits in the world as proposed by Pye (1986) and according to Schiedig (1934).
Loess deposits are generally less than 30m thick. Larger thicknesses only occur in areas where large quantities of dust material have been supplied and sustained over a very long period. Locally they occur in areas where the blowing dust was trapped within a limited geographical area by vegetation or some other surface roughness element. In North America, Argentina and the Danube Basin loess thicknesses of 20-30m are most common, although they occasionally exceed 60m. Loess deposits in China are widely distributed in the northern, north-west and south-east of the country. The total area of distribution is about 630,000 square kilometres, or about 6.6 percent of the land surface of China. The total thickness of the deposits is variable, with a maximum thickness of about 200 to 300 metres (Lin and Liang, 1982). Loess thicknesses exceeding 300m have been recorded near Lanzhou in China (Derbyshire, 1983), but 80 - 120m is more common on the Loess Plateau. In North America the most extensive loess areas are in Nebraska, Kansas, Iowa, Wisconsin, Illinois, Tennessee, and Mississippi. Other loess deposits also occur locally in southern and northern Idaho and south eastern Washington (Holtz and Gibbs, 1951; Gibbs and Holland, 1960).

Large areas in the Asian part of Russia are covered by loess materials. Loess deposits are found in a broad belt extending from Kivghiz and Kazakhstan, through Uzbekistan and Tajikistan to Turkman. Loess occurs on terraces and slopes in river valleys draining the Altai, Ghissars and Pamirs, and blankets many of alluvial fans, foothills and plains on their north western margins. In some parts of Russia loess deposit exceeds 200 m in thickness (Penck, 1930; Dodonov, 1984; Lazarenko, 1984; Pye, 1987). Kraev (1971) reported that about 65% of the Ukraine land is covered with loess soil. The Ukraine loess belt extends westwards into the Baltic Socialist Republics, Poland and Germany. Important areas of loess are found in the Danube Basin, in Bavaria along the Rhine and in Belgium and northern France. Localized continuations
of the north west European loess belt are found in the Channel Islands and southern England (Smalley and Leach, 1978; Lill and Smalley, 1978; Catt, 1985).

Major accumulations of loess materials which occur in South America are those covering large areas in the Pampas region of Argentina. Loess soils cover 25 to 30% of the total area of the country, equating to 800,000 square km. The average thickness of these sediments is about 30 metres with a maximum of about 80 metres. The well known areas are Cordoba and Santa Fe Provinces in the central part of the country (Moll et al., 1989).

Smaller loess deposits are found in several parts of the world, such as Canada (Sweeney and Smalley, 1988), New Zealand (Smalley and Davin, 1980; Eden, 1982), and the northern part of the Indian sub-continent (Pant et al, 1983; Williams and Clark, 1984; Rendell, 1985). Small pockets of loess or sandy loess are locally found in other parts of the world, such as North Africa (Rathjens, 1928; Schiedig, 1934; Anagnosti, 1973; Coudé-Gaussen et al, 1982; Anketell and Ghellali, 1991), Southern Europe (Brunnacker, 1980; Coude-Gaussen, 1990); Middle East (Fookes and Knill, 1969; Erol and El-Ruwah, 1982), Greenland (Hobbs, 1983), Thailand (Phien-Wej et al, 1992), Northern Nigeria (Bennett, 1980); and Australia (Gill and Reeckman, 1980; Gill and Segnitt, 1982; Dare-Edwards, 1984).

3.4 DESERT LOESS PROBLEM

Smalley (1994) pointed out that the desert loess problem comes in two parts: can loess material actually be produced in deserts, and if it is produced what mechanisms are operating, and secondly are there loess deposits around large
deserts, in particular the Sahara? He stated that the Sahara is certainly a source of dust: the discussion by Rozycki (1991) and by Cooke et al (1993), and the papers in the book by Mores (1979) show this to be true. Dust material is generated in the Saharan region by particle impact, by salt weathering and by cold weathering in the desert mountains, but only a modest amount of loess-sized dust is produced. There is still no way in which the Sahara can be considered a major producer of loess-sized dust. The origin of the desert loess problem can be assigned to Obruchev (1911), as reported by Smalley (1990). Obruchev postulated that loess was formed from dust which was blown out of deserts. He created the division into cold loess and hot loess.

Cooke et al (1993) stated that there seems to be several ways in which dust-size quartz particles can be produced in deserts. They also wrote that the controversy of the 1960s and 1970s (Smalley and Vita-Finzi, 1968) over the existence of loess on desert margins seems now to have been resolved; most authorities accept that there are large quantities of desert-margin loess. They noticed that coarse dust with the potential to travel short distances in suspension has diameters of 20-70 μm. Particles smaller than 20 μm are defined as fine dust, with the potential to travel long distances.

The deposition of aolian dust was often considerable around deserts during the Pleistocene and Holocene. Sheets of peri-desert loess have been formed when topographical and bioclimatological conditions were favourable (Coude-Gaussen, 1987). Pye (1987) said that large amounts of dust do appear to leave deserts. Vast quantities of dust leave the Sahara desert and are transported by wind to deposit in the peri-Saharan regions. Figure 3.4 shows the annual dust export from the Saharan desert (Ganor and Mamane, 1978; Cooke et al 1993).
3.4.1 Peri-Saharan Desert Loess

Recent studies which have been conducted by Coude-Gaussen (1987) suggest that there is a region of peri-Saharan loess although earlier observations performed by Penck (1931) indicate quite clearly that the Saharan desert region is free from loess.

On the presence of peri-Saharan loess Rozycki (1991) reported that the Sahara belongs to the few deserts which are not surrounded by well-developed continuous loess zones. A clear analysis of the position of the land areas of possible accumulation of Saharan dust provides an explanation of the situation; according to the prevailing wind direction, the areas of loess accumulation would be expected to occur in the mid-Atlantic, and for the Libyan desert in the eastern part of the Mediterranean.

Coude-Gaussen (1987) reported that the peri-desert loesses are less known than the periglacial loess. However they are well connected by many sedimentological data and by micro- and macro-morphological characteristics inherited from often comparable pre- and post-depositional evolution. The two kinds of loess belong to the same sedimentological family: the fine aeolianites consisting of silty materials derived from dust deposition. The aeolian fine materials of silt-sized particles occur around the Sahara, the greater desert region. The Matmata loess deposit in the South of Tunisia in North Africa is now a well-known example of these peri-Saharan wind-blown deposits (Coude-Gaussen, 1987). This loess deposit exceeds sometimes 20m in thickness, fills basins and upper valleys and sheet interfluves. Some loesses show sedimentological and lithostratigraphical data inherited from their initial emplacement; others are reworked in slope deposits or wadi terraces.
Other deposits of loessic soil type have also been noticed in North Africa the N Saharan extension zone. They are only sparse loess areas, but the geographical inventory is incomplete. Some surficial sheets of loess materials are known on the Moroccan South-Atlas Piedmont and in Fuerteventura, Canary Islands. In the north west of Libya (Tripoli region), the silty deposits of the Jebel Nefusa plateau are the most likely extension of the Matmata loess, re-worked by wadis in fluvia-tile terraces, they occur in the Libyan Jefara plain. Sediments of loess-like material have been previously noticed by Pachur and Braun (1980).

Figure 3.5 shows the locations of peri-Saharan loess deposits in North Africa (Coude-Gaussen, 1987). The common features of the peri-Saharan desert loess deposits were summarized also by Coude-Gaussen (1987) as follows:

1. They have a sedimentary origin related to dust falls issued from desert sources.
2. They have a frequent re-working by runoff or slope processes; colluvial or alluvial silty deposits derived from primary loess materials have often been noticed.
3. They are relatively young, often Upper Pleistocene or even Holocene.

3.4.2 Loess in Libya

On the Scheidig (1934) world map of loess distribution in the world the only definite loess region in Africa was indicated in North Libya. This region was rather uncritically transferred by Smalley and Vita-Finzi (1968) to their world map. It is probable that Scheidig indicated definite loess in Libya as a result of a paper by Rathjens (1928) in which he showed clearly the presence of loess-like deposits in North West of Libya and described their characteristics, but he did point out that 90% of the granular material was greater than 100 microns in
diameter. Rathjens summarized the characteristic properties of loess material deposits in Libya as follows:

1. the deposits cover all small relief features evenly and are even found on the highest parts of the Tripoli Djebel (Jebel Nefusa) as well as on the gentle slopes; this can only be explained by aeolian sedimentation of these deposits;

2. no layering has been observed, apart from those places where fluvial processes have reworked the loess deposits, or where calcium carbonate eluviation and secondary precipitation has caused the formation of zonally arranged carbonate concretions;

3. the porosity and permeability is influenced by the very small capillary tubes (hair-root channels from the steppe vegetation) causing a tendency for the formation of vertical slopes; and

4. only small amounts of clay particles and an enrichment of calcium carbonates as typical loess dolls.

In typical loess the majority of the particles are smaller than 100 μm whereas in the Tripoli loess about 90% are larger than 100 μm, as presented by Rathjens.

Smalley (1995) noticed that if 90% of the particles are larger than 100 μm it is hard to fit the Tripoli loess into any of the current definitions (see Smalley and Vita-Finzi, 1968; Smalley and Derbyshire, 1990). But Rathjens, based on the four characteristics of the Libyan silty material, called it loess, and so did Scheidig, and with Scheidig's book being the starting point for many loess investigations the North African loess entered the scholarly world. At least it entered the West European world; the best known Russian world map of loess distribution (Kriger, 1990) shows no loess at all in any part of Africa.
Loess deposits range from 5 to 20m in thickness, and cover the valley floors on sites where 30 to 40m high storage dams have to be constructed in North Libya. The deposits were investigated by Anagnosti (1973). Test results showed that 60 to 90% of the soil particles are smaller than 74 μm and 5 to 20% of the material are smaller than 2 μm. Anketell and Ghellall (1991) stated that there were probably blown landwards, blanketing earlier deposits and forming important components of the loess in the northern part of the Jefara plain. To the south and in the Jabal Nefusa, Saharan desert loess is probably more important. They noticed that horizontally laminated beds are interpreted as fluvio-aeolian deposits, mainly dune material reworked by sheet-wash. Modern environments show that there is a complex interplay between aeolian and fluvial deposits, with aeolian reworking of fluvial sands and vice versa. They also pointed out that there are some beds of tan to yellow, coarse, porous silts, characterized by columnar jointing which is often quite regular. These beds are commonly calcareous and the older deposits contain irregular calcrete concretions developed around fine root systems. Landsnails are locally abundant. Grain size ranges from very fine sand to silt but as yet no clear field distinctions have been made. This deposit displays many of the characteristics of loess, and was interpreted as loess deposit by Stella (1914) and Rathjens (1928). On the other hand, Penck (1931) noticed that much of the Tripolitania loess is in fact a fine sand, considered too coarse to be transported by aeolian suspension. Its structural similarity to loess was interpreted as largely due to cementation by calcium carbonate in combination with a small proportion of silt and clay. The deposits are best referred to as loessic or loess-like deposits. They probably comprise a mixture of true loess and loess which has been reworked and mixed with fine dune sands by alluvial processes such as sheet-wash (Butzer, 1972).
Libya represents a good sample slice of North Africa and the Saharan desert region; it seems reasonable that if large quantities of desert-margin loess exist then Libya should have some of this peri-Saharan loess. The whole of the peri-Saharan loess has been studied for many years by Coude-Gaussen, who is convinced of its validity. Coude-Gaussen (1987) has reviewed the peri-Saharan loess and presented test results showing a larger mode size of particles than typical loess. She had few samples from Libya, however, and one of the aims of this research is to fill that gap. More careful analysis of North African materials are needed to establish whether they constitute true loess and if so, to indicate its extent and perhaps propose sources and origins. The aim is, however, to show that Libya does have definite loess and that an awareness of this may be important in fields of civil engineering.

3.5 GEOTECHNICAL PROPERTIES OF LOESS

Sheeler (1968) stated that Terzaghi (1951) reported that, by comparing the loess engineering properties obtained by Scheidig (1934) with those described by Holtz and Gibbs (1951), one can realize that the properties of loess are international. There is not even a significant difference between the typical desert loess of Central Asia and the loess in the Mississippi Valley. Krinitzsky and Turnbull (1967) stated that the properties of Mississippi loess are essentially the same as those of loess in major deposits elsewhere in the world. Sheeler (1968) also wrote that, based on the experimental work carried out by Scheidig (1934), in which data were presented on loess deposits in such widespread areas as China, Soviet Union, Central Europe, Argentina and Central and Midwestern United States, shows that there are remarkable similarities in the loess deposits everywhere. Lutenegger (1988) also concluded that the geotechnical properties of loess soil are universally similar.
3.5.1 Index Properties of Loess

3.5.1.1 Specific Gravity of Solid Particles
Generally loess materials have a narrow range of specific gravity values which are fairly similar in most parts of the world. Sheeler (1968) reported that the specific gravity values of loess in the United States vary between 2.57 and 2.79, and locally the range is much smaller. For example, in Iowa State it is from 2.68 to 2.72, in Tennessee from 2.65 to 2.70 and in Mississippi Valley from 2.66 to 2.73. Loess materials from China gave values of specific gravity ranging between 2.71 and 2.75, as reported by Dijkstra et al (1994). Derbyshire and Mellors (1988) found that the specific gravity values for loess from Kent in the United Kingdom vary from 2.68 to 2.70. Specific gravity values for loess samples from Thailand were found on average to be 2.63, as presented by Phien-Wej et al (1992).

The index properties for loess samples from Libya were determined according to USBR standard testing procedures by Anagnosti (1973). The results showed that the specific gravity values were ranging from 2.60 to 2.70.

A histogram which provides summarized data of the specific gravity values for the loess soils in the world was plotted by Jing (1989) on the basis of test results obtained by several investigators (Akiyama, 1964; Anagnosti, 1973; Bolognesi, 1957; Bradford and Norton, 1983; Clevenger, 1956; Feng and Zhin, 1982; Handy, 1973; Janssen and Dempsey, 1980; Kane, 1968; Kane, 1969; Kane, 1973; Krinitzsky and Turnbull, 1967; Lotspeich and Smith, 1953; Olson, 1958; Rieger and Smith, 1955; Rocca, 1985; Schroeder, 1963). The average value for the 77 measurements is 2.70, as shown in Figure 3.6.
3.5.1.2 Density and Voids

Similarly the range of dry density is very low to low. For example Clevenger (1956) found that the dry density values for undisturbed specimens of loess in the Missouri Basin ranged from around 1.20 to 1.36 Mg/m$^3$.

Sheeler (1968) reported that the in-place dry density of loess deposits in the United States varies from 1.057 to 1.665 Mg/m$^3$ and the void ratio varies from 1.46 to 0.62. Test results indicated that the density is dependent on the depth (overburden pressure) and on the clay content. Results of the standard compaction test showed that the maximum dry density values for American loess range from 1.602 to 1.794 Mg/m$^3$ and the optimum water contents from 13 to 20 percent.

Derbyshire and Mellors (1987) stated that loess typically has a dry density of about 1.4 Mg/m$^3$ but can range from 1.15 to 1.75 Mg/m$^3$. Dry density values of 1.40 to 1.46 Mg/m$^3$ for Chinese loess have been given by Derbyshire and Mellors (1988), and the corresponding void ratios are 0.89 and 0.81. Loess samples from Kent in Britain were tested by the same researcher and it was found that the dry density of this soil varied from 1.48 to 1.73 Mg/m$^3$ with void ratios that varied from 0.81 to 0.55.

Sweeney and Smalley (1988) wrote that the loess of Kamloops, British Colombia in Canada exists naturally in a very loose state with a natural dry density of only 1.12 to 1.28 Mg/m$^3$, compared with the maximum dry density of the standard compaction test of about 1.76 Mg/m$^3$ at an optimum moisture content of about 15% (Hardy, 1950). They also reported that Costa and Baker (1981) stated that one of the particular properties of loess deposits is its very low natural dry density, which ranges from 1.10 to 1.40 Mg/m$^3$. 

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Anagnosti (1973) reported that the natural state of Libyan loess can be expressed in terms of its dry density and water content. Dry density values ranged from 1.30 Mg/m$^3$ to 1.60 Mg/m$^3$ for the upper 5.00m, whereas 1.50 Mg/m$^3$ to 1.80 Mg/m$^3$ for the depths larger than 5.00m were obtained. These density values correspond to void ratios 1.0 to 0.70 for the upper 5.00m, and 0.80 to 0.53 for the deeper layers. The natural water content ranges from 2 to 6% for upper 5.00m, and from 4 to 10% for the lower parts of the deposit. The actual distributions of these values are shown in Figure 3.7.

Jing (1989) reported that the insitu natural dry density of loess deposit depends mainly on the depth of the loess sediment and the conditions during its formation. The dry density of most of the loess soils in the world ranges from 1.04 to 1.76 Mg/m$^3$ and the void ratios corresponding to these values are 1.59 and 0.53 respectively, as shown in the histogram of Figure 3.8.

3.5.1.3 Plasticity Characteristics

In general loess soils are slightly to moderately plastic, the plasticity increasing as clay content increases. Gibbs and Holland (1960) indicated that Liquid Limit values for silty loess vary from 25% to 35%, and for clayey loess the values range up to 45% (see Figure 3.9). Plasticity Index values were between 5 and 22 percent.

Derbyshire and Mellors (1988) found that the Atterberg Limits for Chinese loess are Liquid Limit 29-32%, Plastic Limit 12-22%, and Plasticity index 10-17%, and for British loess the values are Liquid Limit 28-46%, Plastic Limit 17-23% and Plasticity index 9-28%. All values plotted above the A-line on the plasticity chart which indicates that they are inorganic soils of low to medium plasticity. The low Plasticity index is an indication that the soil properties must be very sensitive to changes in water content.
The consistency limits for loess samples from Thailand were determined by Phien-Wej et al. (1992). They found that the Liquid Limit was 18%, the Plastic Limit was 13% and the Plasticity index was 5%.

Erol and El-Ruwaih (1982) demonstrated that the Liquid Limits and plasticity indices of the desert loess vary over a range of 33-38% and 9-13% respectively. When these values are plotted on a plasticity chart, the loess can be classified as a borderline between inorganic silts and clay of low plasticity (ML-CL). Atterberg limits of Libyan loess were obtained by Anagnosti (1973). The Liquid Limit varied from 18 to 25%, the Plastic Limit from 15 to 19% and the Plasticity index from 3 to 6%.

3.5.1.4 Size and Shape of Solid Particles
Loess is an aeolian deposit mainly consisting of silt-sized quartz particles in the range 20-60μm (medium to coarse silt), and usually has a prominent mode of 30μm. These particles are mostly of sub-angular and platy shape, and form the main constituents of the loess fabric that make up the basic skeleton, which is welded by a matrix of clay-sized materials (predominantly primary minerals). The clay particles occur as aggregates and as coatings on the larger grains (Grabowska-Olszewska, 1988; Derbyshire and Mellors, 1988; Dijkestra et al, 1994). For Lanzhou loess in China it was found that the clay fraction ranges from 7.5% to 12.7% of the total solid constituent (Tan, 1988).

Derbyshire and Mellors (1988) said that it is evident that the dominant size fraction throughout the loess plateau of China is coarse silt (20 - 60μm). The percentage of clay material averages 25 - 35% in the Malan and Upper Lishih loesses. Loess from all regions is uniformly graded (grading coefficient ranges from 1.6 to 3.0) and consistently fine-skewed (skewness ranges from +0.3 to
The Malan and Upper Lishih loesses lie predominantly in the silty loess envelope which was proposed by Holtz and Gibbs (1951). Figure 3.10 shows the general shape of grading envelopes for Lanzhou loess deposits (Malan, Lishih and Wucheng) as reported by Derbyshire and Mellors (1988). They have also stated that the predominant size fraction in the loess soils from Britain is coarse silt, i.e. from 20μm to 60μm with all materials being generally in excess of 65% silt content (i.e. 2μm to 60μm).

The general shape of loess gradation curves in the United States is shown in Figure 3.11, as presented by Sheeler (1968). He also indicated that Holtz and Gibbs (1951) classified loess material into clayey loess, silty loess, and sandy loess as shown in Figure 3.11.

The composition of loess in Poland, as reported by Grabowska-Olszewska (1988), are:
- sand-size grains 3 - 28% (usually 11 - 19%);
- silt-size grains 63 - 88% (usually 73 - 76%); and
- clay-size particles 6 - 10% (usually 10%).

Loess soils from Thailand were classified as (SM-ML soil) low plastic silty sand which consists of 40 - 50% sand, 30 - 40% silt and 20% clay (Phien-Wej et al, 1992). The typical range of grain size distribution curves of the desert loess was presented by Erol and El-Ruwaih (1982), as shown in Figure 3.12. The soil is predominantly composed of silt size particles with smaller amounts of clay minerals, along with minor quantities of fine sand. The clay content varies over a range of 16 to 25%, whereas the fine sand fraction comprises approximately 10% of all the material. The desert loess possesses grain size and consistency limits characteristics very similar to those of North American loessial soils. Test results illustrated that loess samples from Libya contain a fraction of silt size particles from 60 to 90% and a clay fraction from 5 to 20%. 

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and the maximum grain size was found to range from 0.50 to 2.00mm (Anagnosti, 1973).

The shape of solid particles of loess material depends largely on their mineral composition and crystallographic structure, the processes responsible for their formation, and the degree of pre- and post-depositional shape alteration (modification) induced by weathering and diagenesis processes. The majority of silt-sized quartz particles are blocky and angular or sub-angular; small amounts of sub-rounded grains sometimes exist. Quartz particles derived from active chemical weathering environments, often show edge rounding due to solution and reprecipitation, whereas fresh grains formed by sub-glacial grinding or frost action often have sharp edges and display conchoidal and stepped fracture surfaces (Pye, 1987). Some investigators have suggested that quartz grains finer than about 100μm show a tendency to become more platy with decreasing size, due to some cleavage control on quartz fracture (Krinsley and Smalley, 1973; Smalley, 1974), but such a tendency has not been observed in loess particles as pointed out by Goudie et al (1984). Equidimensional and blocky grains are often produced by mechanical crushing of feldspar minerals as a result of cleavage effects (Pye, 1987).

3.5.2 Permeability of Loess Deposit

The permeability of a loess deposit is important in evaluating the rate and amount of seepage and the uplift forces beneath hydraulic structures for stability analyses. It is also important in evaluating the rate of settlement where loess volume changes occur as water is absorbed or expelled from the soil pore spaces. The permeability of a natural loess is related mainly to its dry density (see Figure 3.13), and types and amount of pore spaces. Loess
permeability is much higher in vertical than horizontal direction, since it is enhanced by long vertical pore spaces in the loess structure. This voids feature is due to the existence of vertical plant root holes and vertical shrinkage joints within the loess mass (Sheeler, 1968; Bell, 1993).

3.5.3 Strength Characteristics

Loess deposits can support heavy structures with small settlements if loads do not exceed the preconsolidation pressure and the natural water content is relatively low. On the other hand loess can compress substantially if the preconsolidation stress is exceeded. Primary settlements generally occur rapidly, with much of the settlement occurring during the actual application of load. Loess soil may also exhibit creep deformation under loading.

The strength of loess formation is dependent mainly on the initial dry density and moisture content, the type and amount of bonding material, the grain size and grain size distribution, as well as the changes in moisture content.

Apparent cohesion values obtained from unsaturated loess samples are dependent mainly on the water content, clay content, and dry density. The cohesive strength increases with clay content and dry density and decreases with increasing water content. The angle of internal friction for loess deposits increases with increases in density. An increase in water content or clay content may reduce the angle of internal friction.

The effective cohesion values for saturated loess specimens are very low and mainly governed by the clay content. The effective angles of internal friction
are mostly between 25 to 35 degrees (Anagnosti, 1973; Erol and El-Ruwaih, 1982).

Anagnosti (1973) reported that the strength properties of loess soils depend largely on the state of the sample under test. Undisturbed loess samples from Libya of natural density and water content have shown considerable strength and small deformability, but after being saturated the properties of undisturbed samples change considerably and marked weakening in respect to the strength and deformability had been observed. The test results illustrated that the unconfined compression strength values vary from 116 to 270 kN/m² for samples with natural water content from 2 to 6%; and from 28 to 32 kN/m² for samples with water content from 13 to 15%.

Undisturbed loess samples from China were tested in a direct shear box by Dijkstra et al (1994) who concluded that the youngest Malan loess from China has a peak shear strength at relatively low normal effective stresses, but fails plastically at normal effective stresses above approximately 100 kN/m². The older loess (Lishi from China) exhibits greater strength and brittle failure because it is relatively dense and cemented, as shown in Figure 3.14. The behaviour of the remoulded loess is characterized by higher effective internal friction angles and lower effective cohesion values. The change in the effective angle of internal friction is caused by the redistribution of the particles into a denser packing, while the loss of cohesion is caused by the breaking up of the cementation bonds.

The shear strength properties of the desert loess have been investigated by Erol and El-Ruwaih. The results showed that the effective cohesion (c') and the angle of shearing resistance (θ') are 46 kN/m² and 26.5° respectively for the dry soil. These strength parameters c' and θ' are reduced to 10 kN/m² and 24°
respectively as a result of saturation, as presented in Figure 3.15. The angle of shearing resistance was slightly decreased upon saturation because of the possible lubrication effect of water on the grain surfaces. The effective cohesion, however, is remarkably reduced as a consequence of saturation.

The strength characteristics of loess in Thailand have been investigated and presented by Phien-Wej et al (1992). They concluded that the drainage condition is an important influencing factor for shear strength as much as the initial water content and consolidation pressure (density). A loose structure could remain after initial saturation and consolidation at a pressure as high as 400 kPa, leading to a much lower undrained strength than drained strength. Figure 3.16 summarizes the dry density changes of the loess specimens due to the effect of various levels of isotropic consolidation pressures in the unconsolidated-undrained (UU), consolidated-undrained (CU) and consolidated-drained (CD) triaxial tests. The significance of the initial water content on the undrained and drained triaxial strength can be clearly seen from Figure 3.17(a) and Figure 3.17(b). A comparison plot of strength envelopes of the loess samples tested in the triaxial machine is presented in Figure 3.18.

Laboratory tests have been performed by Milovic (1988) on loess samples from Yugoslavia. Based on the obtained results, several correlations have been established. A correlation between the initial dry density, initial water content and unconfined compression strength are illustrated in Figures 3.19, 3.20 and 3.21. It can be clearly seen that the strength properties of loess deposits depend largely on the initial dry density and initial water content. The results show that the shear strength of loess specimens with the same initial dry density decreases and the compressibility increases with increase in water content. On the other hand the soil strength increases and the compressibility decreases
with increase in the initial dry density of loess samples of the same initial water content.

3.5.4 Collapsibility of Loess Deposits

An important problem encountered by geotechnical engineers in many areas of the world involves soils which possess considerable insitu dry strength that is largely lost when these soils become wetted. This problem requires particular attention in the design and construction of hydraulic structures where foundation wetting is to be expected. Even in other structures, the undesirable moistening of foundations may occur when the water table rises, or when surface run-off water is not properly drained from the structure site. Unfortunately, these occurrences are not always anticipated and structural failures result. Probably the best known example of engineering structures which have settled and damaged due to wetting of foundations are those which were built in loessial soil areas located in several parts of the world.

Loess soil is a porous deposit blanketing the landscape, and usually occurs naturally in a partially saturated state. It is a loose wind-deposited soil mainly composed of uniform silt-sized particles which are generally bonded together by small amounts of clay coatings and clay bridges to form an open granular framed metastable structure of high void ratio and low dry density. The clay bridges and coatings acting as a primary cement at interparticle contacts give the loess structure considerable strength when it is in a dry state. However, since the cementing material constitutes a very small fraction of the soil, only a slight increase in moisture content may affect the strength significantly. Under extreme conditions such as saturation or flooding, the soil structure may collapse completely under an imposed load and large settlements result (hydro-
collapse). The dry density or void ratio resulting from wetting was found to be the same regardless of whether the specimen was wetted prior to loading or after the load was applied, as shown in Figure 3.22 (Clevenger, 1956; Sheeler, 1968; Derbyshire and Mellors, 1988; Houston et al, 1988; Bell, 1992).

Loess deposits which have been formed in arid and semi-arid environments and have experienced limited pre-wetting may collapse under their own overburden pressure when flooded, but in more humid regions collapse on wetting normally occurs only if stress is applied. Some loesses are capable of both swelling and collapsing, depending on the magnitude of the applied pressure at the time of flooding. Soils which show a tendency to swell are those which have relatively high dry densities, high clay contents or contain a significant proportion of swelling clay minerals (e.g. montmorillonite) and are flooded at low applied stresses and low initial water content (Pye, 1987; Derbyshire and Mellors, 1986).

From the evidence of loess sediments found in China and Poland, it appears that collapse does not occur in all these soils. Those older deposits in which the particle packing and mineral composition have been modified by weathering processes are not nearly as susceptible to collapse as young unweathered loesses. For example Gao (1988) pointed out that the hydro-collapse of loess in China is related not only to the origin and formation of the material, mode of transportation and deposition, but also to weathering effects. For instance, the weakly unweathered young loess of the north-west of the loess plateau has a high potential for hydro-collapse, whilst the weathered material of the south-east of the plateau is relatively stable and the features associated with collapsible loess are gradually disappearing. In Poland Grabowska-Olzewska (1988) found that hydro-collapse is most frequent in the youngest loess having a clay content of slightly more than 10%. Such loess is
of aeolian origin and is characterized by a random texture and a carbonate content less than 5%. Handy (1973) commented that the tendency for loess samples from Iowa to hydro-collapse is related to the clay content. Whereas loess containing less than 16% clay has a high possibility of collapse, with 24% clay it has a 50% probability of collapse, and with 36% clay the chance of collapse is negligible. The size of the pores is also important. It was found that hydro-collapse occurs as a result of reduction taking place in pores greater than 1μm, and more especially in those exceeding 10μm in size (Grabowska-Olszewksa, 1988). Grabowska-Olszewksa also stated that loess hydro-collapse can be related to natural water content, as shown in Figure 3.23. It is clear from the Figure that loess soil with natural water content less than 6% is potentially unstable, and that soil in which the natural water content exceeds 19% can be regarded as stable.

Bell (1992) reported that the compression curve for a saturated loess is similar to that for a normally consolidated clay (see Figure 3.24). He defined the collapse pressure \( P_{cs} \) as a pressure beyond which deformation increases appreciably. The collapse pressure varies with the initial water content (degree of saturation), so the collapse pressure for loess soil with a natural water content equal to \( P_{cn} \), and for collapsible soils \( P_{cn} \) is bigger than \( P_{cs} \). He also stated that Popescu (1986) defined collapsible soil as that in which the collapse pressure of a saturated soil sample \( P_{cs} \) is less than the overburden pressure \( P_{O} \) of that sample at natural state (see Figure 3.24). This means that such soil will collapse when saturated since the soil structure cannot then support the load due to the overburden weight. When the saturation collapse pressure \( P_{cs} \) exceeds the overburden pressure \( P_{O} \) soils are capable of supporting a certain level of applied stress on saturation, and these soils have been defined by Popescu as conditionally collapsible soils. They can support a certain stress up to the difference between the saturation collapse and overburden pressures.
Hydro-collapse test results for undisturbed samples have been presented by Houston et al. (1988), as shown in Figure 3.25. The results show the collapse behaviour before and after saturation presented in three stages: in situ moisture curve, collapse settlement curve due to saturation and saturated compression curve.

The hydro-collapse mechanism of loess deposits is a complex phenomenon. The amount and rate of hydro-collapse of the soil's structure are dependent on such factors as the initial water content (degree of saturation), the initial dry density (porosity), type and amount of clay minerals, applied pressure at time of wetting and stress history of the loess. Pye (1987) reported that several different mechanisms of loess hydro-collapse have been proposed (Fookes and Best, 1969; Barden et al., 1973; Lin Zaiguan and Liang Weiming, 1980; Lutenegger, 1981; Grabowska-Olszewska, 1975, 1982), but an important process is micro-shearing at points of interparticle contact; this may be related to dispersion of clay bridges at these points when the loess is wetted. It has been shown that the amount of collapse for a given load can depend on the degree of dispersion effected by the flooding liquid (Mellors, 1977; Derbyshire and Mellors, 1986). The hydro-collapse phenomenon will be explained in more detail in the next chapter.

3.6 SUMMARY

* The origins and formation of loess deposits have been controversial topics of discussion for a long time, but it is now generally accepted that true loess has resulted from the aeolian transportation and subsequent deposition of silty material. Loess material (silt-size particles) are produced by several different processes in a wide range of environments. The principal
processes responsible for the production of the loess particles appear to be subglacial grinding by both ice sheets and glaciers, frost cracking in periglacial climates, breakage by salt crystal growth and hydration in desert regions, fluvial transport impact and aeolian impact of sand grains.

* Recent work suggests that there is a region of peri-Saharan desert loess, although earlier observations indicate quite clearly that the Saharan desert region is free of loess. The desert loess problem persists. There is a lot of confusion with respect to the peri-Saharan loess. The nature and sources of desert loess have generated some controversy. North Africa represents a good example of peri-Saharan desert region and Libya is in the middle of this region. More careful studies and analysis of North Africa silty materials are needed to establish whether they constitute true loess and, if so, to indicate its extent and perhaps propose sources and origins.

* It was concluded that the geotechnical properties of loess soil are universally similar. The engineering properties of loess deposit on which most investigators agree can be summarized as follows:

1. loess is mostly buff, pale yellow, or light brown in colour, but may be grey, red or white;
2. loess is a soil of predominantly silt size with small amounts of sand and clay;
3. the physical characteristics of loess such as specific gravity, Atterberg Limits and particle gradation are very similar;
4. the specific gravity values of loess material mostly range from 2.65 to 2.75, and the average value is 2.7;
5. typical loess has a dry density of about 1.35 Mg/m³ and void ratio of about 1.0, but it can range from 1.1 to 1.6 Mg/m³ dry density and from 1.45 to 0.70 void ratios;
loess soils are slightly to moderately plastic, the plasticity increasing as clay content increases. Generally the Liquid Limit values vary from 25 to 45 and the Plasticity index from 5 to 20;

natural water contents are generally well below saturation and generally range from 4 to 12 percent;

the structure is open and porous;

it has a low interparticle bond strength, and this bonding is primarily due to clay bridges and coatings;

a loess deposit of 1.60 Mg/m3 dry density is generally stable;

structural hydro-collapse happens rapidly under load when loess is saturated or near saturation;

shear strength and bearing capacity are largely dependent on water content and dry density of the loess deposit.

* Normally, loess has a moderately high dry strength and can support relatively high loading when the water content is low. When the soil is wetted, however, this strength is lost and collapse of the loose natural structure results. The strength of loess deposits is dependent mainly on the initial dry density and water content, the type and amount of interparticle bonding, the grain size and grain size distribution, as well as the changes in moisture content. It was concluded that the apparent cohesion values obtained from unsaturated loess samples are dependent chiefly on the water content, dry density, and clay content. The cohesion strength increases with clay content and dry density and decreases with increasing water content. The angle of internal friction for loess soils increases with increases in density. An increase in water content or clay content may reduce the angle of internal friction. The effective angles of internal friction for saturated specimens are generally between 25 to 35 degrees, and the effective cohesion values are very low.
Loess is a porous soil blanketing the landscape, and usually occurs naturally in a partially saturated state. It has a loose and open granular framed metastable structure which is generally bonded together by small amounts of clay coatings and bridges. Hydro-collapse is one of the most important properties of loess deposits, and consists of a sudden reduction in volume under the effect of wetting and loading. Loess hydro-collapse of the soil structure is dependent on such factors as the initial water content, initial dry density, type and amount of clay minerals, applied stress at time of wetting and stress history of loess deposit. Different mechanisms of loess hydro-collapse have been proposed, but an important process is micro-shearing at points of interparticle contact; this may be related to dispersion of clay bridges at these points when the loess is wetted. It has been shown that the amount of collapse for a given load can depend on the degree of dispersion effected by the flooding liquid.
Table 3.1 Loess definitions as provided by INQUA Loess Commission
(From Smalley, 1975)

<table>
<thead>
<tr>
<th>Name</th>
<th>Definition/description and synonyms</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loess</td>
<td>German synonyms: <em>Loess</em>, typischer <em>Löss</em> (<em>typical loess</em>). Characteristics: the definitely dominant fraction of the sediment is within 60-20μm (coarse silt, very fine sand), unstratified, primarily calcareous, quite porous capillary network; on the whole, dry material is yellow, buff, brownish yellow.</td>
</tr>
<tr>
<td>Sandy loess</td>
<td>German synonyms: <em>Sandlöss</em>, <em>Flotsand</em>, lösiger Sand, sandiger Löss. Characteristics: mixture of grains sized 60-20μm and 500-200μm (fine sand, medium sand); often the distribution of particle sizes show a major peak within the silt range and a lesser peak within the medium sand range; sometimes there is an equal distribution among silt, (very) fine sand, and medium sand fractions; very often they are unstratified or in thin beds, usually noncalcareous, not so porous as loess, color similar to loess.</td>
</tr>
<tr>
<td>Clayey loess</td>
<td>German synonyms: Tonlöss, toniger Löss, tonreicher Löss. Characteristics: peak particle size of the sediment is within the range from 6-20μm with 25-30% of particles being smaller than 2μm (clay size); unstratified, low porosity; similar carbonate content and color to loess.</td>
</tr>
</tbody>
</table>
| Loess-like sediments| Germany synonyms: *Lössderivate*, lösartige Sediments. General characteristics: the term covers primarily eolian material that has been moved or redeposited in various (secondary) processes (allochthonous loess-like sediments) and/or modified in situ (autochthonous loess-like sediments); relevant processes are:  

*Deluvial* (colluvial) processes and solifluction:  
hill-washed loess, solifluction loess, solifluxion loess (German terms: *Solifluktionslöss*, *Fließlöss*, *Berglöss*, *Hanglöss*).  
Fluvial (proluvial) processes:  
brickearth, brickearth (German: *Schwemmlöss*, *subaquatischer Löss*).  
Modification caused by cryoturbation:  
cryoturbation loess (German: *Kryoturbationlöss*).  
Eluvial and pedogenic processes:  
loess loam (German: *Lösslehm*, *Gleylöss*, *Staublehm* (*dust loam*), *Decklehm* (*covering loam*).  
Thorough, intense pedogenic modification and transformation (redeveloping):  
"semi-pedoliths" and "pedoliths" (these terms are proposed by M. Pecsi for lithified soils formed from loess material).  

Loess-like sediments may have originated from either loess, sandy loess, or clay loess; in any case their porosity is less than that of the original material; great variation of carbonate content, some may be essentially noncalcareous; colors may differ considerably in particular cases.  

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Fig. 3.1 A Loess Landscape in North China, (from Richthofen, 1877)
quartz-particle production by glacial grinding

transportation of rock debris by glacier

formation of mixed deposit by outwash and after glacier melts

silt-sized particles lifted by wind and transported by eolian action

loess deposit formed

erosion of primary loess and fluvial transportation of particles

floodplain deposition; secondary deposits

eolian transportation from floodplains

loess deposit formed the second-cycle loess

introduction of clay minerals

introduction of organic material

variations in carbonate content

Figure 3.2 Events in the formation of loess deposits. Events in the right column cannot be assigned a definite place in the left sequence (after Smalley, 1979)
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1. Matmata Deposit (South Tunisia) 2. Fuerteventure Deposit (Canary Islands)
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5. Abu Ballas Deposit (Egypt) 6. Fouta Djallon (Southern Manding Plateau)
9. Northern Cameroon
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Figure 3.12 Grain-Size Distribution of Desert Loess as Presented by Erol and El-Ruwaih (1982)
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1 lb/ft$^3$ = 16.018 kg/m$^3$

1 ft = 0.3048 m
Figure 3.14 Stress-Strain Relationship for Undisturbed Loess Specimens from China Tested in the Direct Shear Box by Dijkstra et al. (1994)

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CHAPTER 4
HYDRO-COLLAPSE PHENOMENON
OF LOESS DEPOSITS

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CHAPTER 4
HYDROCOLLAPSE PHENOMENON
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4.1 INTRODUCTION

The hydro-collapse phenomenon is basically a change in soil structure by the effect of water. It is a transformation process from a metastable open structure to a more stable structure of closer packed particles. This process involves a re-arrangement of the solid particles, modification of the inter-particle bonds and alteration to the pore spaces (voids) between the granular particles and particle aggregates.

Structural hydro-collapse, and consequent subsidence, of loess deposits presents enormous engineering problems in many countries in the world and is the focus of much research on both the nature of this phenomenon and the prevention of the negative consequences of these problems (Rogers et al, 1994a). Although some significant earlier studies (e.g. Abelev 1931) had been carried out, hydro-collapse was first introduced into the literature by N. Ya Denisov (1940).

Sajgalik (1990) defined sagging (hydro-collapse) as a process in which a sudden reduction in soil volume takes place due to over moisturing and vertical loading. He said this definition follows Denisov (1972), Larionov et al (1959), Lysenko (1980), Kriger (1986), and others. He concluded that hydro-collapse is very complicated, depends on many factors and still requires much research.
effort. He classified the causal factors into internal and external factors and divided the process of hydro-collapse into three time phases.

Sajgalik concluded that the hydro-collapse of loess formations depends mainly on the characteristics of pore spaces and structural bonds of the soil, in which the main role must be ascribed to the quantitative and qualitative representation of the clay minerals and the capillary adhesion forces. He reported that Kriger (1986) had said that loess hydro-collapse is a particular natural phenomenon closely connected with the natural environment. To understand the mechanism of loess hydro-collapse, the process has to be reproduced experimentally, so that the variables can be controlled, for example using the oedometer, and data obtained must be analysed in conjunction with a study of micro-structural changes.

4.2 GEOTECHNICAL PROBLEMS RELATED TO THE HYDRO-COLLAPSE OF LOESS DEPOSITS

Hydro-collapse was observed during the soil investigation for storage dams in Libya. The project was located in regions covered by loess deposits of 10 to 20m in thickness. The laboratory tests indicated the possibility of having 1.0 to 2.0m settlement upon saturation of the natural loess deposits resulting in danger of cracking of the dam body during filling of the reservoir (Anagnosti, 1973).

Another example: a mill building was constructed on a loess stratum of 6.5 - 7.5m thickness. The building suffered considerable deformation upon wetting of the loess. Deformation of the structure was noticed shortly after its erection in 1972, and the maximum settlement reached 14cm (Wahed Hassani and Goel, 1982).
Kraev (1971) reported that experience in the construction of different buildings and structures on loess deposits provides numerous examples of deformation and settlement problems. In several areas of the Ukraine collapse may be as great as 1.0 to 1.5m, with thickness of the loess averaging 20 to 40m.

The Teton Dam was a very large earth dam built in Idaho, North America, in 1975. It was a large core design, with the core accounting for approximately half of the dam's volume. The dam collapsed catastrophically in June 1976. It was really a disaster. Sherard (1987) said that "the Teton Dam failure is one of the most important single events in the history of dam engineering". Sowers (1993) described the Teton Dam failure as "probably the most catastrophic and costly civil engineering failure of this century: costly in money and costly in loss of professional status". There was a large agreement that the cause of the disaster was seepage erosion (piping) in the dam core. Two geological factors appear to have played key roles in the failure: the fissured rhyolite rock on which the dam was built and the brittle, permeable and easily erodible loess material used for the dam core (Smalley, 1992; Rogers et al, 1993). Seed and Duncan (1981) concluded that "it is hoped that we can all benefit from the lessons provided by this failure".

These field examples of geotechnical problems are presented here to emphasize the practical importance of hydro-collapse studies. The lesson from these structural failures is that better knowledge of geology and particularly Quaternary geology is essential for civil and especially geotechnical engineers. A good knowledge of loess properties and its hydrocollapse behaviour will help prevent such geotechnical problems.
4.3 HYDRO-COLLAPSE CHARACTERISTICS OF LOESS DEPOSITS

Loess hydro-collapse is one of the most interesting soil phenomena. Existing literature on this subject is considerable but its ordering appears rather difficult. Of all the problems in geotechnical engineering, the sorting out and ordering of the literature on loess collapse is probably the most challenging (Grabowska-Olszewska, 1988). Three main observations on loess collapse have been obtained by Grabowska-Olszewska:

1. Collapse is most intense in the stratigraphically youngest loesses in a given section.
2. Collapse is almost exclusively restricted to loesses of dust size particles (silt-size particles) with 10-15% clay minerals.
3. The collapse loesses are connected with primary aeolian accumulation and they are usually characterized by a random texture, pale yellow colour, and a carbonate content over 5%.

Trofimov (1990a) states that collapsibility is a specific property of loess deposits, i.e. they have the ability to decrease their volume under a constant acting pressure after wetting. This property makes loess soils fundamentally different from many other types of soils. He has reported that the genesis of collapsibility is a complex question, with which many Soviet researchers have been engaged, e.g. Abelev, (1948); Anan'ev, (1983); Denisov, (1946); Kriger, (1987); Larionov, (1959, 1965); Mavlyanov, (1958); Minervin, (1979). Recent specific reviews have been undertaken by Kriger (1986), Minervin (1982), Minervin and Komissarova (1983), Sergeev and Komissarova (1986), Trofimov (1980, 1986a, 1986b, 1987a, 1987b) and Shaevich (1987). In spite of this, still many propositions are unclear and/or debatable, and a theory of the formation of collapsibility in the full sense of the word has not been created.
According to Trofimov, a collapsible structure can be formed as both a syngenetic and an epigenetic property of loess. Syngenetic collapsibility concerns collapse properties which are formed either directly in the course of accumulation or early subaerial diagenesis of a silty sediment in arid conditions, and epigenetic collapsibility concerns collapse properties which are formed by progressive lithogenesis of young alluvial and lacustrine-alluvial deposits in subaerial conditions.


1. The characteristics of hydro-collapse may be explained by the under-compactibility of loess sediments, formed as a result of the cementation of loosely deposited open loess structures and by water-soluble structural bonds, and the emergence of internal stresses caused by the overburden pressure. Kriger calls this Denisov's principle.

2. Hydro-collapse properties can be explained by a reduction in density brought about by hypergenetic processes (i.e. an insitu loessification). This view has been supported by Sergeev (1986), the general idea here apparently being that periglacial cold conditions produce an open soil structure of low dry density that can subsequently collapse.

3. The properties of loess hydro-collapse can be explained by the nature of the deposition process of various sediments. Kriger says that this means they belong to a definite genetic type, a view that is widely accepted in Central Asia due to Maulyanov (1958).

Yang (1989) concluded that loess possesses a primary loose structure and a particular grain size distribution (i.e. certain amounts of the frame element, clay cementing agent, and filling element) immediately after it has been
deposited. After deposition the clay would cement the grains together because of the action of water. With time secondary hydro-stable cementation would form, allowing the loess to maintain its structure during subsequent deposition. Loess possesses collapsibility because the cementation agents are hydro-labile, (i.e. they reduce in strength when the soil comes into contact with water). For the treatment of collapsible loess, one method is to modify its loose structure into a tight structure. Another approach is to enhance the clay bonding or enlarge the proportion of hydro-stable cementation.

Browzin (1981) established that the rate of hydro-collapse accelerates until a peak of the rate is reached, the rate of deformation thereafter beginning to decelerate which is the beginning point of the consolidation process (see Figure 4.1). It was found that the post-wetting consolidation curve resembles the consolidation behaviour of classic clay, but has different parameters. He concluded that the subsidence (hydro-collapse) is a rheological process with a very high acceleration of the rate of deformation. The amount of subsidence increases with an increase in the content of the loessial fraction (10 \( \mu m \) to 50 \( \mu m \)).

The connection between loess collapse and soil sensitivity has been explored by Lutenegger (1981). In the classical Terzaghi sense, loess soils would probably not be considered sensitive soils because the ratio of undisturbed to remoulded strength, at constant water content, is usually around 3, depending on clay content. Loess would therefore generally fall into the category of medium sensitivity. If, however, sensitivity is taken as the ratio of undisturbed to saturated strength, as suggested by Feda (1966), then some loess soils would appear to have very high sensitivities. In investigations throughout the mid-western USA, in-situ stability of loess has been related to liquidity index, that is when the in-situ water content reaches the Liquid Limit, usually on
saturation, stability is lost. This can be readily identified by isolated flow in boreholes. Because Liquid Limit is related to clay content, and saturation water content is a function of density, this instability only occurs in special circumstances, typically in low density and low clay content soils.

Kraev (1971) concluded that for equal values of dry density a loess soil has been found more collapsible when it possesses a lower natural water content, contains a small percentage of clay minerals, and has lower plasticity indices.

Tan (1988) has reported that the loess from Lanzhou, the capital of Gansu Province in P.R. China has an average void ratio (e) of 1.05. The loosest theoretical packing of spheres of equal diameter has a void ratio of 0.91. The mixed sand and silt particles in loess, which are sub-angular and sub-rounded, must be mutually separated and he suggested that they can only be interconnected by cementing bonds as shown in Figure 4.2. He also stated that as the water will be the primary influence on the bond material, which comprises only approximately 7 - 12% of the soil skeleton, small changes in water content may have a major influence on the mechanical properties. This must be ascribed to the presence of hydrophilic soluble salts and clay particles. A further important factor is time, since loess masses show time effects as creep deformation and relaxation. Under compression the internal structure of loess may collapse and grains will be squeezed into the pore spaces, resulting in mutual interlocking. In this case the loess is transformed into a soil type of more stable structure. He consequently concluded that typical loess has a collapsible structure with a high initial void ratio of more than 1.0, a mechanical sensitivity to the influence of slight variations in moisture content, and a big change in mechanical properties before and after structural collapse.
Derbyshire and Mellors (1988) concluded that the loess deposits of Central China and south-west England are very similar in terms of their characteristics such as particle size ranges, mineralogy, fabric and their potential to collapse. Some important differences between these two loess types exist, however, and these are predominantly expressed in a higher plasticity, lower porosity and a lower collapse coefficient for the English loess. This is the result of the greater degree of sorting of the English loess which induces the formation of more compact fabrics. Additionally, these loess deposits show a greater degree of reworking and clay particle translocation. Derbyshire and Mellors also indicated that the assessment of the collapsibility of a loess soil depends on the availability of sufficient void spaces in its natural state to hold its Liquid Limit water content at saturation. Such criteria may be applicable to soils which will undergo collapse on wetting under their natural overburden pressure, and they do not necessarily cater for those soils which will collapse only under additional pressure, as for example applied by the weight of a structure, and are considered therefore to be of limited applicability. Greater reliance should be placed on direct measurements, such as the testing of specimens in oedometers.

The hydro collapse behaviour of Palouse Loess was studied by Lobdell (1981). He concluded that the results of hydro-collapse tests indicate that Palouse Loess undergoes very little structural collapse due to wetting. This suggests that Palouse Loess is not a typical loess like those of the mid-west United States.

Phien-Wej et al (1992) have investigated the hydro-collapse characteristics of loess deposit in Thailand. Based on the test results they have concluded that:
1. The metastable loose structure of the soil which makes it susceptible to hydro-collapse is mainly attributed to clay-bridge bonds.
2. Hydro-collapsibility of the loess is strongly governed by the initial water content and the magnitude of applied stress, as shown in Figures 4.3 and 4.4.

3. It was found that the critical pressure at which the hydro-collapse started to occur was greater for the soil specimens with a lower water content, as shown in Figure 4.3. At natural water content the critical pressure was greater than the overburden pressure.

4. Collapse deformation caused by wetting at a given applied stress always resulted in approximately the same final void ratio regardless of the initial water content, as shown in Figure 4.5. This reflected the existence of a unique post-wetted relationship between void ratio and applied stress.

5. The predicted hydro-collapse settlement based on the double oedometer test results compared reasonably well with the settlement measured in the field plate bearing tests, as shown in Figure 4.6.

6. Microscopic studies (scanning electron microscopy and thin section microscopy) revealed that the micro-structure of the loess is of an unconsolidated, loose and open structure type resembling that of honeycomb, as illustrated in Figure 4.7(a). The pore sizes are mostly 200 - 500µm in diameter although some can be as large as 1000µm (1mm). The coarse grain fractions, which are mostly quartz, form skeleton grains of the open structure either in the form of single large grains or aggregation of smaller grains. The skeleton grains are held together mostly by means of clay cementation in the form of clay bridge bonds, as shown in Figure 4.7(b). Most of the clay content, which is predominantly kaolinite, is present in the form of clay bridges even though some portion is in the form of a surface coating on coarse grains.

Collapsing soils are encountered in many areas of the Arabian Peninsula. The hydro-collapse behaviour of desert loess has been investigated by Erol and El-
Ruwaih (1982). Their site investigations showed that collapsible soils are characterized by stiff or hard in-situ consistencies (as reflected by their high standard penetration resistances) but open porous fabric (as indicated by low in-situ densities and high porosities). The soil was termed Desert Loess to emphasize the effect of arid climate conditions of the region. Their main findings were:

1. In the Arabian Peninsula, widespread occurrence of transported soils leads to formation of calcareous loess soil in wide plains and wadis of deserts. The physical properties of the desert loess are found to be quite similar to loess soils of North America.

2. The desert loess deposits are of high collapse susceptibility. Collapse settlements more than ten per cent of the original thickness were measured in single and double oedometer tests upon wetting, as shown in Figure 4.8.

3. The amount of collapse is dependent on the magnitude of vertical stresses acting on the soil at the time of wetting, as illustrated in Figure 4.9. A linear relationship between hydro-collapse settlements and logarithm of applied pressure was obtained.

4. The collapse potential can be attributed to the presence of water-irresistible cementation materials and removal of high capillary suction by water.
4.4 FACTORS CONTROLLING THE HYDRO-COLLAPSE PHENOMENON

4.4.1 Introduction

Many investigators have concluded that the hydro-collapse of loess deposits is very complicated and depends on many factors. These factors can be classified into structural factors and environmental factors. The structural factors are of fundamental importance in determining the properties and behaviour of loess hydro-collapse. The effect of environmental factors on the behaviour of collapsible soil have generally been reviewed in Chapter Two. The structural parameters and their influence on the hydro-collapse phenomenon are presented in this section.

4.4.2 Soil Structure

Soil materials are essentially composed of discrete solid particles and particle groups. They are usually treated as a continuum of soil mass for analysis and design. However, the engineering properties and behaviour of soils are determined directly by the size and shape of solid particles and pore spaces between them, the arrangement of both the particles and the pores, and the interparticle bonds. These basic factors are known as the structural parameters. The soil structure has been defined by Mitchell (1993) as the combined effects of fabric, composition, and inter-particles forces (bonds). He also defined the term fabric as the arrangement of particles, particle groups (aggregates), and pore spaces in a soil. Two types of soil structure can be identified: single-particle structure and multi-particle structure.
To take both soil fabric and its stability into account the term structure is used. The term structure is also used to differentiate between the properties of a soil in its natural state and of the same soil at the same void ratio but thoroughly remoulded. The remoulded and reworked soil is said to be destructured and the natural undisturbed soil is said to be structured. "The effects of soil structure are as important in determining engineering properties and behaviour as are the effects of initial porosity and stress-history, which are the basic concepts of soil mechanics" (Leroueil and Vaughan, 1990). Aggregation is an important feature of loess soil structure. Silt particles and some of the clay material combined together to form globular-shaped aggregates with a size range of 10μm to 100μm (see Figure 4.2). Aggregates have varying water resistance according to the type of structural bonds between the particles from which the aggregate is composed.

Four basic types of loess structure have been defined by Larionov (1971) as graining, aggregative-graining, graining-aggregative, and aggregative (Osipov and Sokolov, 1995). The structure of the first type consists of granular silt-size particles in direct contact with one another. As the clay content increased the formation of clay-silt aggregates increased. Depending on the relative content of aggregates, the loess structure can be classified as aggregative-graining or graining-aggregative. A further increase of clay content in loess materials results in the complete submergence of primary quartz grains into aggregated clay-silt masses and direct contact between grains appears to be impossible. In this state the loess is described as aggregative. It has been established that in most cases loesses which have a grainy structure (mainly composed of primary quartz silt particles) possess maximum collapsibility, whereas loess materials with an aggregative structure (high percentage of clay minerals) undergo minimum collapse. The loess hydro-collapse decreases proportionally with an
increase of clay content. Loess with an aggregative structure collapse more slowly than loess with a graining structure (Osipov and Sokolov, 1995).

Thus it can be concluded that loess hydro-collapse seems to be controlled mainly by soil structure. Fabric and interparticle bonding are the main parameters which form together the structural system of the soil. Soil fabric includes the spatial arrangement of solid particles and pore spaces between them, and the packing of particles.

4.4.3 Characteristics of solid particles

The geometrical properties (size and shape) of loess particles and their effect on the structural collapse of loess deposits have been discussed by Rogers et al (1994a). They reported that the particles comprising loess materials are mainly of silt-size and mostly of blade shape, and the properties of these deposits depend in some part on the properties of the individual particles.

The particle size fraction \(10\,\mu m\) to \(50\,\mu m\) in loess materials has been identified by Browzin (1981) as the loessial fraction, and has been suggested as the basis for a new loess classification. The fraction smaller than \(10\,\mu m\) is defined as the fine or clayey fraction, and that larger than \(50\,\mu m\) as the coarse or sandy fraction. He found that the magnitude of hydro-collapse increases with an increase of the content of loessial fraction.

Yang (1989) divided the loess particles into three size regions:

(a) grains with a diameter bigger than \(10\,\mu m\) make up the frame of loess structure (skeleton), and this fraction is called the frame element;
(b) grains with a diameter of 2 - 10 μm fill in the void space made up by the frame element, and this fraction is called the filling element;

(c) grains with a diameter smaller than 2μm play the role of bonding and cementing agents, and this fraction is called the cementing element.

Generally, the Lanzhou loess consists of 70 to 75% frame element, 10 to 16% filling element and 10 to 15% cementing element.

Miao and Wang (1989) defined three size ranges similar to those of Yang, but with different terminological definitions. The particles larger than 10μm are called 'skeleton framing'. These skeleton particles are cemented by an additional 10 - 15% with diameters less than 2μm. The rest, with diameters 2 - 10 μm are called 'stuffing'. They quoted the same proportional contributions as Yang.

Krinsley and Smalley (1973) suggested that small quartz particles in sedimeted loess material will tend to be blade shaped, as predicted by simple probability theory by Smalley (1966). Rogers et al (1994a) proposed that, as a first approximation, the structural unit in an airfall loess deposit be considered as a primary mineral particle (usually quartz) with an axial ratio of 8:5:2. This really is an ideal particle, generated using random number theory (see Rogers and Smalley, 1993). This particle shape was generated in an isotropic, homogeneous solid material (see Scott and Smalley, 1991).

Experimental results have shown that silt-size quartz particles sedimented in water can have a void ratio as large as 2.2 as compared to an upper limit of about 0.91 for single-grain assemblages of equal spherical particles (Mitchell, 1993). This is because of the fact that quartz particles in this size range tend to be platy. The silt size particles may form multi-particle arrangements during slow sedimentation because they are sufficiently small that their arrangements
can be influenced by surface force interactions. Mitchell (1993) reported that "a honeycomb type of fabric as shown schematically in Figure 4.10 is thought to exist in some silts (Terzaghi, 1925)." Multi-particle arrangements of silt-clay mixtures can be formed because of the importance of particle surface forces in relation to particle weight; the adsorption of clay particles on silt grain surfaces; and because of the chemical reactivity of clay surfaces.

4.4.4 Pores and Space Features

Pore sizes, shapes and distributions (or space factors) are essential fabric elements in loess structure. Engineering properties and collapse behaviour are strongly influenced by the void spaces available to accommodate displaced particles. The hydro-collapse phenomenon of loess deposits is consequently largely related to the space factors of the soil structure, which are in turn related to the granular constituent, the climate conditions, the geographical environment, and the weathering pedogenesis (Lei, 1989).

To be able to assess the micro-structural changes during the hydro-collapse processes measurements of pore sizes and distribution must be made. The space factors for initial metastable specimens must be compared with those for the specimens after hydro-collapse. Test results showed that a smaller volume of pores up to 2μm in diameter, and markedly higher volume of pore spaces varying from 2 to 10μm occur in natural undisturbed samples of loess. This indicates that the translocation of broken clay coatings and bridges is a most significant element of the hydro-collapse process. The hydro-collapse phenomenon seems, therefore, to be influenced particularly by pore spaces of approximately 10μm and greater. The proportion of pore spaces of this size showed most extensive changes, being modified to pores of smaller size. For
example, results have indicated that the total volume of pores of this size changed on an average from 45 to 37% after hydro-collapse had been induced (Kukanova and Sajgalik, 1994).

The distribution of pore sizes (hence total porosity) is not the only space factor which affects the loess hydro-collapse, the shape of pores having a significant effect also on this phenomenon. The types of pores which are observed in loess prior to hydro-collapse have been classified by Klukanova and Sajgalik (1994) as follows:

1. Pores exceeding 100μm in size. These pores most likely result from the weathering of roots and plant capillaries, and activity of organisms or salt dissolution. Their shape is usually grain-like to elliptical.

2. Pores from 10μm to 100μm. These are characteristically inter-granular and inter-aggregate pores and most likely synsedimentary (formed during the sedimentation processes). Their shape varies between isometrical and longitudinal.

3. Pores smaller than 10μm. They are mostly represented by intra-aggregate types.

It was traditionally believed that the main role in the process of hydro-collapse is the destruction of micropores, which are those pores smaller than 1μm (Sajgalik, 1990). He reported that recent research carried out by Grabowsica-Olszewska (1983/1988) and Klukanova (1989) shows that although this contributes to the process, its role has been overestimated. Porosimetric tests show that the main role in the process of hydro-collapse is played by micropores and mesopores (pores of 1 to 1000μm in size). The ultra and micropores (under 1μm) do not take part in the process of hydro-collapse.

The types of loess pores and their relationship with collapsibility have been investigated by Xiangyi (1988). He stated that loess is rich in various types of
pores and that the engineering properties of loess, such as strength, permeability, and collapsibility, are closely related to the characteristics of loess pores. Xiangyi measured the textures of loess pores by means of mercury capillary pressure and has studied pores both before and after structure collapse under polarizing and scanning electron microscopes. It has been found that there has been an obvious regular variation in the distribution of loess pores in regions or in the loess layers of various geological ages and that, because of their different sizes and sorts, they contribute to the significant variability in loess collapsibility. He classified the pores according to their origin and size. On the basis of origin, loess pores have been divided into seven kinds: (i) intergranular openings (including the supporting, or arching and mosaic openings), (ii) pores in cementing materials, (iii) root-pores, (iv) insect-holes, (v) mouse-caves, (vi) joints and fissures, and (vii) solution caves. Based on the size, loess pores were divided into four types:

1. large pores with a diameter larger than 32 μm
2. medium pores with a diameter range of 8 - 32 μm;
3. small pores with a diameter range of 2 - 8 μm;
4. very small pores with a diameter less than 2 μm.

Xiangyi concluded that the porosity (void ratio) of loess is one of the important factors causing structural collapse on wetting. However due to the different size, shape and distribution of the loess pores, the influence of these pores exhibits significant variability on collapsibility. Among them, the medium pores (mainly the supporting openings) may be one of the most significant factors which make the loess produce collapse on wetting. He concluded that loess pores are very complex, and that pores characteristics and their relationship with collapsibility need further exploration.

Yang (1989) studied pore-size distribution in both undisturbed and collapsed specimens. He observed that pores with a statistical diameter of less than
108 \mu m comprise over 97% of the pores space, and that loess collapse is basically caused by these pores, the contribution of this pore fraction to the total collapse space being over 80%.

On the basis of the pore size measured by means of mercury capillary pressure, Lei (1989) classified the loess pores into the following types:

1. Large pores, in which the throat radius is $> 16 \mu m$
2. Medium pores, in which the throat radius is $4 - 16 \mu m$
3. Small pores, in which the throat radius is $1 - 4 \mu m$
4. Very small, in which the throat radius is $< 1 \mu m$

He concluded that the large and medium pores, particularly the medium pores, are important factors controlling subsidence on wetting, and consequently the collapsibility of any particular loess can be distinguished by their content. When their content is more than 40% of the total volume of voids, loess can be identified as collapsible. Figure 4.11 shows the relationship between the contents of the large and medium pores and the coefficients of hydro-collapsibility as presented by Lei.

Two main types of pore spaces have been identified by Suzuki and Matsukura (1992): the 10 \mu m pores, and the 0.05 \mu m pores.

Collapsible loess deposits are characterized by high porosity, which normally varies between 40 and 50%. Osipov and Sokolov (1995) indicated that loess porosity increases with an increase of clay content, and that the most porous loesses are those of an aggregative structure. They identified three types of pores:

1. macro-pores of tubular form (100-500 \mu m);
2. inter-grain and inter-aggregate pores (8-100 \mu m);
3. intra-aggregate pores (< 8 \mu m).
The total volume of the tubular macro pores varies from 1 to 8%. The total volume of the inter-grain and inter-aggregate pores varies from 15 to 28%, and they play an important role in loess collapsibility. The volume of intra-aggregate pores varies from 18 to 25% and increases during the period of transition from a metastable to a more stable structure. Osipov and Sokolov stated that "in spite of the importance of porosity in loess hydro-collapse there is still no precise quantitative relationship between loess collapsibility and its porosity characteristics."

4.4.5 Particle Packing

The particulate nature of soil and the many possible combinations (or arrangements) of discrete particles mean that a soil of given composition may exist over a wide range of structural states, each having its own unique set of geotechnical properties (Mitchell, 1993, page 160). For any fine-grained soil in which particles have an ability to flocculate, the initial fabric after sedimentation will be open and involve a certain amount of edge-to-edge and edge-to-face associations in a "cardhouse" arrangement. During loading this structure can carry effective stress at the same void ratio higher than what would be possible if the particles were arranged in an efficient, parallel array (Mitchell, 1993).

The description of particle packing is one of the most important problems in the scientific study of the hydro-collapsibility phenomenon. The hydro-collapse event is essentially a change of particle packing, from an open packing to a much closer packing, which involves a rearrangement of the modal loess particles. The soil structure that collapses in loess hydro-collapse is an open packing of blade shaped particles with a mode size of around 30μm. The mean
axial ratio of the particles dimension is around 8:5:2 but there is considerable variability about this ratio. The open packing is due to airfall sedimentation, i.e. relatively slow particle arrival and slow build up of overburden structure of the soil deposits. The interparticle contacts may be effectively stabilized by some kind of cementing materials (e.g. clay minerals) before effective overburden pressure develops. The slow deposition of the particles means that the coordination number is low, so there are relatively few particle contacts and these consequently have a greater individual importance in maintaining the structure. From the hydro-collapsibility point of view, the interparticle contacts are surely the most important part of the whole loess system.

Rogers et al (1994a) stated that it would be useful if the initial (open) packing and final (closed) packing could be represented in order to understand the spatial reconstruction which connects the two boundary conditions. It would be useful if the collapsible structures could be modelled, but this is a difficult task. The basic reference to particle packing in sediments is in Allen (1982) and, although it provides a fascinating and sophisticated look at ideal packings, it exposes the enormous gap between real and ideal packings. Allen developed the Smalley (1971) system of regular packing, both by introducing non-spherical particles and considering the random approach to packing. The problem is how to represent the randomness that simulates an in-situ clastic deposit. This approach was first attempted by Smalley (1964a) via two types of radial distribution functions - a density function and a probability function. The density function can be applied with recognized severe constraints to elastic sediments (Smalley, 1964b). This method has been applied to a pebble beach (Smalley, 1964b) and to a post-glacial marine quick clay (Smalley, 1978). It allows particle environments to be demarcated and possibly allows a compaction process to be followed.
The early SEM studies (e.g. Smalley and Cabrera, 1969) hinted at the possibility of relating structure to properties for engineering soils, but the promise was not fulfilled.

A soil structure model can theoretically be constructed using a very simple Monte Carlo method (see Assallay et al., 1997 and Dibben et al., 1997). This method was used by Smalley (1978) to produce a two-dimensional picture of the structure of a post-glacial clay soil, and was adapted to produce the open structure of airfall loess particles by Rogers et al., 1994a. A set of random number tables (e.g. Kendall and Babington Smith, 1939) was used to produce random coordinates and random orientation of particles within a square frame of 100 x 100 units. In case of overlapping the last particle is rejected.

Figure 4.12 shows an initial open structure of high porosity on which compaction operations can be carried out. If a 10 units x 10 units frame is drawn on graph paper and each particle occupies 2 units x\(\frac{1}{2}\) unit, i.e. 1% of the frame, manipulation and calculation are relatively straightforward. For example, the 28 particles placed in Figure 4.12 give a porosity of 72% and void ratio of about 2.57. In this figure the particles are represented as flat plates i.e. the axial ratio of 8:5:2, as proposed by Rogers and Smalley (1993) for typical loess particles, is preferentially orientated to show the 8:2 aspect. The compaction of the system as proposed by Smalley (1978) is achieved by shrinking the vertical axis and redrawing, allowing for contacts to realign particles. Thus as the particles move into contact a logical adjustment has to be made, initially by rotation and in the later stages by translation, as far as required. Three stages of compaction (15%, 30% and 45%) reduced the porosity to about 50% (i.e. to a void ratio of approximately 1). Figure 4.13 shows the system at the final stage of compaction, at 45% linear reduction. Figure 4.12 looks remarkably like the Uriel and Serrano (1973) structure, and
this is possibly a reasonable two-dimensional representation of collapsible soil structure. The Uriel and Serrano structure is basically an arbitrary construction. The structure which is illustrated in Figure 4.12 was produced in a totally random manner, and represents an initial metastable open packing structure.

The relationship between fabric characteristics and hydro-collapsibility of loess has been studied by Lei (1989). He has reported that the hydro-collapsibility of loess is closely related to the size and distribution of pores and the cementation types. Based on the studies of micro-texture using a scanning electron microscope (SEM), together with data on the contact relationship between solid particles, properties of pores, and the degree of cementation, the micro-fabric in the loess plateau of China can be divided into three fabric groups and each group further subdivided into two fabric types (Wang. et al 1982), as shown in Table 4.1. The supporting macropore weak-cementation fabric is formed by the silt grains supporting each other with little cementing material, resulting in large intergranular pores (i.e. the skeleton is formed by solid grains). The mosaic micropore weak-cementation fabric is that arrangement in which the contact area between coarse fragment grains is larger, and the intergranular pores are smaller. Again there is little cementing material at the points of contact of the grains. The semi-cementation fabric group is a transition from the weak-cementation fabric group to the cementation fabric group. The mineral grains are only partially cemented and part of the grains and intergranular pores are clearly visible. The main feature of the cementation fabric group is that the intergranular pores are filled by cementing materials. This group can also fall into two fabric types: the flocculent cementation fabric, which contains some porous and loose cementing materials, and the coagulum cementation fabric, which is characterized by compact cementing materials. In the loess plateau of China,
the loess micro-fabric characteristics change gradually from the weak-cementation fabric group to the semi-cementation fabric group, and then to the cementation fabric group, both from the north-west to the south-east and with depth in sections.

Lei has concluded that the loess deposits having a supporting macropore weak-cementation fabric or a supporting macropore semi-cementation fabric as a dominant texture has a large coefficient of hydro-collapsibility, i.e. its potential of hydro-collapse on wetting is greatest, and its initial pressure of hydro-collapse is the lowest. In contrast, the loess formation of cementation fabric is generally non-collapsing on wetting, and its initial hydro-collapse pressure is very high in cases where collapse does occur. The loess structure of mosaic micropore weak or semi-cemented fabric which has hydro-collapse properties lie between both macropore weak cemented and semi-cemented fabrics.

The study of the loess micro-fabric is of great significance for the primary evaluation of loess engineering properties in a certain region. However, for the present, the study of loess micro-fabric using SEM techniques cannot yet quantitatively estimate the collapsibility because of a lack of experience in this aspect. By further study of the micro-fabric type, it is feasible that in the future the hydro-collapse properties of loess may be reasonably estimated from its micro-fabric.

4.4.6 Interparticle Bonding

One of the key factors in evaluating the hydro-collapse behaviour of loess deposits appears to be the mineralogy of the soil, specifically that of the clay
minerals which comprise the majority of the bonding materials (Lobdell, 1981). Osipov and Sokolov (1995) stated that the clay minerals play an important role in the formation of structural bonds since the contact between the primary silt particles is mostly realized through clay minerals. When typical loess is saturated, the clay cement adsorbs water and a loss of strength occurs. If the effective stress is sufficient, the weakened clay will fail in shear at points of contact between grains. Destruction of the interparticle bonds allows the solid particles to move into void spaces producing a reduction in volume and a permanent settlement. The loss in strength (and hence hydro-collapse behaviour) of a clay-cemented soil thus depends on the nature of the clay mineral since different clay minerals vary in their ability to adsorb hydrogen cations and thus in their ability to adsorb water. The response of the structural bonds to loading and wetting is consequently the main factor affecting the hydro-collapse phenomenon and thus the nature of the bonding between loess particles needs investigation.

Handy (1973) concluded that when density and Liquid Limit data are not available, a preliminary estimation of collapsibility may be made on the basis of clay content. It has been found that the increase of clay mineral content decreases the potential of hydro-collapse. Osipov and Sokolov (1995) stated that loess collapsibility decreases with an increase in content of clay minerals and low soluble salts. However, a linear correlation between these factors and collapsibility index was not established. These observations are of major importance, but only very few researchers have acknowledged it. Much effort is therefore still needed to clarify the relationship between the clay minerals as a bonding force and the hydrocollapse behaviour of loess deposits.

Yang (1989) divided the bonding materials into two types: hydro-stable and hydro-labile cementations (see Table 4.2). Hydro-stable cementation is a kind
of bonding material in which the strength does not obviously decrease when it comes into contact with water, whereas hydro-labile bonding suffers a decrease in strength when it comes in contact with water. The mineralogy of cementing materials for Lanzhou loess was determined as illite 40 - 60%, calcite 20 - 30%, chlorite 9 - 16%, kaolinite 6 - 10% and montmorillonite < 1%, and some traces of quartz and feldspar.

Osipov and Sokolov, 1995 state that the structural bonds in loess deposits are determined by four types of forces:

1. molecular force;
2. ion-electrostatic force;
3. capillary force; and
4. chemical force.

The molecular forces are the weakest ones. The molecular force capacity between two clayey particles in individual contact is less than $5 \times 10^{-8}$ N for low saturation ratios and reduces as saturation increases to reach $5 \times 10^{-10}$ N in the presence of complete saturation (see Table 4.3). The ion-electrostatic forces are widespread in loess and play an important role in its collapsibility. The strength of such bonding depends on the contact area and a number of physio-chemical factors. For individual interparticle contacts in the air-dry condition, the ion-electrostatic force reaches $5 \times 10^{-8}$ N to $4 \times 10^{-7}$ N, which significantly exceeds the strength of contacts formed by molecular forces (see Table 4.3). Water has a significant effect on the ion-electrostatic forces. When loess materials become fully saturated, quick hydration of clay mineral and cation surfaces takes place. This hydration results in an increase of distance between the clay particles and consequently a sharp decrease of ion-electrostatic force strength. In fully saturated loesses ion-electrostatic forces disappear completely as a consequence of the formation of diffused ion layers and the emergence of repulsion forces between the particles.
Capillary forces have a major role to play in loess hydro-collapse. The relationship between loess strength (Pm), measured with a cone penetrometer, and the degree of its saturation (Sr), is shown in Figure 4.14a. Three sections can be identified in this graph. The first is for Sr < 0.15 and is characterized by a sharp decrease in strength as a result of the weakening of molecular and ion-electrostatic forces due to the formation of a film of adsorbed water at the interparticle contacts. The second is for 0.15 < Sr < 0.8 and is characterized by a more gradual loss of loess strength as the degree of saturation increases. This behaviour is explained by the action of capillary forces, which provide additional structural bonds at particle contacts. In the third section (Sr > 0.8) loess stability sharply decreases again, "a step" in the graph being evidence of saturation in this interval (see Figure 4.14b). This is attributed to the disappearance of capillary forces and is associated with the complete destruction of loess structure and the development of structural collapse.

The presence of salts and oxides of iron and aluminium in loesses can provide another type of structural bonding due to chemical forces. These forces are capable of forming the strongest cementation between particles, and normally reach 5 x 10^-7 to 5 x 10^-6 per individual contact. These cementation bonds may be either unstable or stable in water, depending upon salt composition: some are formed by easily soluble salts, such as chlorites, while others are formed by medium soluble (gypsum) and low soluble salts (calcite) as well as iron-oxide and aluminium oxide. Contact bonds formed by non-water-resistant salts quickly lose their own stability when saturation occurs and favour loess collapse.

The simple physico-chemical bonding model developed for quick-clays (highly sensitive post-glacial sediments) by Cabrera and Smalley (1973). can
be applied to the loess situation. The quick clay is in many ways very similar
to collapsible loess since it is largely composed of blade shaped primary
mineral particles, is formed by slow-fall sedimentation of single particles,
develops a cementation bond at particle contacts, is given to sudden failure,
and is directly connected with the cold phases of the Quaternary period.
Denisov (1953), Feda (1966) and Lutenegger (1981) have made this
connection and discussed loess as a sensitive soil. The simple bonding system
proposed by Cabrera and Smalley (1973) consists of two simple bond classes:
long-range bonds and short-range bonds. The long-range bonds are mobile
bonds, whereas the short-range bonds are contact bonds which lose strength
when broken. The ideal long-range bond is that found in a classical clay
mineral system of high plasticity (PI), because for the long-range bond to exist
in a soil system, electrostatically charged soil particles are required. The short­
range bond can operate between two primary mineral particles, indeed it
operates between two quartz sand grains although the weight of the particles
masks its effectiveness. If the particle size is reduced, however, the short­
range bonds become more effective. They appear to be very similar in concept
to the 'atomic' or 'point contact' bonds described by Osipov (1978), whereas an
example of the long-range bond is the 'coagulation' bond.

In the light of this discussion, consider an ideal loess deposit consisting of a
random open packing structure of blade-shaped quartz and feldspar particles,
with no added clay or carbonate or other cementing minerals. The interparticle
contacts would be of the atomic, or short-range, type. This structure would be
rigid and probably dilatant. The quartz-quartz point contact is not particularly
weakened by water, therefore it should not collapse when loaded and wetted.
The system would probably fail at the bond points under shear stress, by
tensile failure. These hard contacts do occur in quick clays, which require an
initial stress to cause failure. In contrast, the structural hydro-collapse in loess
deposits occurs because the contact bonds are modified. The simple quartz-quartz contact bond tends not to exist in collapsing loess. There are two obvious modifiers, clays and carbonates (or other chemical bonding agents). These co-exist in many loesses but initially they will be considered separately.

Four simple possibilities are considered by Rogers et al (1994), as shown in Figure 4.15

1. **Small clay content**: a small amount of clay mineral concentrated at the bond points of the main structure. The main structure is based on that introduced by Uriel and Serrano (1973), (see Figure 2.15) and generated by Smalley (1978), (see Figure 4.12).

2. **Small carbonate content**: here the bond points are jointed by small amounts of carbonate cement to produce a rigid structure, the bond strength is increased but it retains its short-range nature.

3. **Large clay content**: the clay minerals begin to fill the large spaces in the basic structure and may form a continuous clay 'phase', and establish a long-range type of bonding.

4. **Large carbonate content**: the carbonate fills spaces and produces large-scale cementation.

The response to shear stress of these four ideal systems is shown in Figure 4.16. Curve 4.16b shows a classic short range response in which the strength of the original structure is quite high, and it is rigid, but when it fails the strength drops rapidly. Curve 4.16c shows a typical long-range bond response in which the clay minerals have a powerful influence on the system (which, therefore, has a high PI) and deformation does not significantly reduce the strength. The response of a loess deposit with a small clay content (Curve 4.16a) is perhaps the most interesting. If the amount of clay is small there is still considerable short-range bonding in the system. The clay at the contacts modifies, rather than fundamentally changes, the short-range behaviour and the
strength falls on disturbance, but not as rapidly as in the small carbonate case. The most important question is how these ideal bond systems respond in the hydro-collapse situation. The small clay contents model appears to agree with Handy’s (1973) observations on the Iowa loess. He found that collapsible loess had a low clay mineral content and that when the clay mineral content was greater the loess did not collapse. The small clay content system is relatively rigid when it is dry but when wetted bond mobility occurs, only relatively small movement being required for structural collapse to occur. In the large clay content system plastic deformation might occur but structural collapse does not occur (as in the case of New Zealand loesses).

Thus it can be concluded that the system overall has to behave initially as a short-range bonded open structure, in which the initial bonds have to be rigid, but subsequently local long-range behaviour is required as the clay modified bond points move under the influence of water and stress. The initial short-range nature may be emphasized by some small carbonate in the system giving additional rigidity. The carbonate responses in Figure 4.16 are not significantly different, the larger amount giving a higher initial strength, rigidity and permissible deformation, but once failure occurs strength drops very rapidly. Loess cemented with a substantial amount of carbonate will be what Yang (1989) calls hydro-stable and will not be expected to collapse.

4.5 HYDRO-COLLAPSE MECHANISM

The study of structural changes in loess formations during the hydro-collapse processes makes it possible to explain the mechanism of hydro-collapse and predict its structural model. Loess hydro-collapse occurs due to the weakening
of structural bonds at the contacts of structural elements, destruction of less water resistant aggregates and the reduction/disappearance of capillary forces in complete loess soil saturation. Development of these processes results in the continuous transformation of the loess structure, which occurs in three stages: initial metastable structure (before the collapse), destruction of structure during water saturation and (post collapse) formation of a new more consolidated and stable structure (Osipov and Sokolov, 1995).

The mechanism of hydro-collapse cannot be explained by the principles of effective stress (Feda, 1984; Feda, 1988; Sajgalik, 1990). The hydro-collapse phenomenon can be clarified only in the conceptions of structural mechanics. Feda approached this phenomenon very correctly from the structural standpoint. Without structural analysis it is impossible to explain the hydro-collapse acceptably (Sajgalik, 1990). In this respect, Kane (1973) stated that the cause of hydro-collapse is either the exceeding of the shear strength of the bonds between the structural elements due to loading, or the decrease of their structural strength by wetting.

Sajgalik (1990) and subsequently Kiukanova and Sajgalik (1994) divided the process of hydro-collapse into three time phases, which develop gradually. In the first phase deformation of the original micro-structure occurs, caused by moisture increase and due to the effect of external pressure. The deformation is manifested by the disintegration of clay coatings, bridges and supports, the disintegration of aggregates and micro-aggregates, the transformation of clay minerals into suspension, and the increased solution of carbonates. In the second phase the hydro-collapse of the micro-structure occurs, as the clay minerals, carbonates, soluble salts and some metal oxides are leaked from the soil. A considerable compaction of the soil takes place and its volume decreases, with a relatively high reduction in soil porosity. In the third phase
the soil acquires a heterogeneous structure, when compared with the original homogeneous structure. The contacts between the clay minerals are changed into more physically stable ones and the clay particles become more aggregated. The silt and the sand grains are also reorientated and have frequently direct contacts as compared to the original covered and separated ones. Even the relations between the individual structural elements are changed, since the percentages of the individual size fractions of pores are changed. When a flocculent cementing agent comes into contact with water, the thickness of the electric double layer of its colloidal particles increases because of the diffusion of the flocculent ions. Similarly, the thickness of the "hydrous membrane" also increases, so that the strength of the bonding materials between the solid particles decreases or vanishes.

4.6 EXPERIMENTAL MODELLING OF METASTABLE STRUCTURE

Rogers et al (1994a) reported that an insight into metastable structure parameters, in particular the pore spaces, bonding and packing of particles, would certainly help studies on hydro-collapse of soil structure. The initial metastable, open packing structure is due to air-fall sedimentation, i.e. relatively slow particle arrival and slow build up of overburden in which the interparticle contacts may be effectively stabilised before effective overburden pressure develops. Rogers et al also stated that it would be useful to simulate the collapsible structure of loess deposit experimentally.

The formation of collapsible structure is an interesting and complex problem, and a theory of the formation of soil collapsibility, in the full sense of the word,
has not been created. In the literature only a few propositions have been suggested by Russian investigators and summarized by Trofimov (1990a) including trials based on the Denisov's (1953) hypothesis about the formation of aeolian collapsible loess. Denisov (1953) noted that accumulations of aeolian silty soil of open structure could be transformed into slightly cemented material only after its wetting and drying. A physical modelling of the formation of an aeolian collapsible structure of silty material was carried out by Kolomitsev (1985). He used samples of medium and heavy loess-like loams taken from the cross-section of Atel strata in the lower Volga region. The soil used is characterized by a high content of coarse silt (10-50 μm) and fine sand (50-100μm), particle aggregates (up to 80% of the material), and a significant amount of carbonates (about 7.5%) and water-soluble salts (about 10%).

In the course of specimen preparation and testing, an air-dry, powdery mass, formed as a result of pulverization of the natural soil with a rubber pestle, was filled from the top into plastic rings 97.0mm in diameter and 33.5mm high. Then, one series of the sediments formed was wetted from the top with water, with the amount of water being taken so that the degree of saturation of the system should be equal to 80% to 90%. The other series of sediments in the rings was moistened by capillary action from below. After this, the wetted specimens in rings were dried in accordance with Denisov's scheme to an air-dry state at a temperature of 18-20°C, with the drying being done only through the upper plane of the system. During the process of wetting of the silty fill material, some decrease in its volume was observed, which then increased in the process of drying (shrinkage deformation). As a result, highly porous specimens of loess appearance were formed, which displayed collapse properties: the coefficient of collapsibility of artificially formed specimens was about 0.1 with an applied pressure of 0.3 Mpa. The amount of collapsibility of
specimens wetted by soaking from above was somewhat higher (0.140) in comparison with those prepared by wetting from below (0.098).

Galai (1987) performed experimental works on modelling the formation of collapsibility of aeolian loess deposits. The silty material used in the experiments was obtained by pulverization of air-dry samples of natural loess and loess-like material, and sieving through an 0.5mm sieve. Galai substantiated the need for a sieving operation by the fact that during dust storms the maximum size of quartz particles transported by the wind of 150-200mm/sec velocity (the beginning of dust storms) is 0.5 - 0.7mm. The prepared silty soil was filled into rings, set on glass, with an area of 60cm² and height of 2.5 - 3.5cm. On the upper surface of the specimens, filter paper was placed and through the paper the silty material was wetted to the moisture content of the Liquid Limit, which simulated the action of atmospheric precipitation. After that the specimens were dried to an air-dry state and once again moistened to the water content indicated above. The wetting-drying cycle was repeated four times; in the last cycle the specimens were dried to values of water content corresponding to the natural moisture content of the loess soils used in the experiments. In one series, the drying of specimens was done without an additional load on the water-saturated silty soil samples; and in the second series the drying of specimens was done with a surface stress of 50kPa applied. The first series of specimens models the natural situation occurring in the part of an aeolian loess deposit nearest to the surface; and the second series simulates the natural loess structure at a depth of 2.5-3.0m, which corresponds to the thickness of the loess layer soaked by atmospheric precipitation in Denisov's construction. During the wetting-drying cycles it was observed that significant shrinkage of silty soil specimens only occurred after the first cycle. In the subsequent cycles of wetting and drying in the
formed specimens, which already possessed considerable structural bonding, significant changes in porosity were not observed.

Porosity and collapsibility were determined according to standard procedures for both the specimens prepared according to the above procedure and specimens of natural loess deposits of the same materials. The results obtained indicate that practically all of the artificially formed specimens are highly porous and collapsible, the porosity and amount of collapsibility being higher than for the corresponding natural loess deposits for the first series of specimens, in the second series they were generally close to each other.

Similar experimental investigations into the formation of partially saturated aeolian silty deposits were performed by Korolev and Eremina (1986). This was done by means of wetting of silty loess material (powder) by capillary feed of water via moisture-holding media with a given capillary pressure, and by hydration and dehydration in a climate chamber. Rings of standard compression instruments were set on a levelled surface of small masses of wet sandy soils (the initial moisture content in these masses was equal to 3, 5, 6, 7.5, 10 and 20%), which perform the role of a moisture-holding medium with known capillary pressure. An air-dry silty powder prepared by means of pulverization of loess material was filled into the rings; the initial value of the porosity of specimens in various rings varied from 48.5 to 63.5%. Then, all the specimens were placed in a desiccator for 1-2 weeks, during which wetting of the sediment powder in the rings occurred on account of capillary feed of moisture from the sand until a state of equilibrium was reached, in which the capillary potentials in the sand and silt (loess) parts of the system were equalized. After that the rings with the wetted silty sediment were taken off the desiccator and the specimens were dried to an air-dry state. In the course of the wetting and drying process, the silty material was transformed into cohesive
soil. Shrinkage of the specimens was not observed at all in specimens with low initial water content, whereas in specimens of higher water content it was small but insignificant. As a consequence of this the porosity of all of the model specimens remained high. Determination of the collapsibility of one of the formed specimens with hygroscopic water content, done under an applied vertical pressure of 100 kPa, showed that the artificial soil had acquired collapse properties. The coefficient of collapsibility was 0.15.

In another series of experiments a powder of loess material was filled into rings of a compression instrument causing a sediment of aeolian silt-size particles with porosities of 45-61%. These rings, with their contents, were put into a "Feitron" climate chamber, in which the specimens were subjected to cyclic actions of variable temperatures and humidity for a period of one month. For each specimen, 25 cycles of wetting and drying were performed. Each cycle included wetting of the specimens at a temperature of 20°C and relative humidity of 99-100%, condensation of water vapour by lowering the temperature in the chamber to 1°C, and drying of them with a relative humidity of 20-30%. During these processes, structured soils possessing noticeable structural strength were obtained although the strength (6 to 9 kPa) was lower than that of the first series of specimens. Shrinkage of the specimens was not observed and the high initial porosities (45-61%) were maintained. Collapsibility results of air-dry, artificially formed loess specimens tested under an applied stress of 100 kPa showed that they all had acquired collapsible structure. The values of the coefficient of collapsibility varied from 0.12 to 0.16 and again proved to be higher than the values for the original natural loess.

Comparison of the micro-structure natural loess soils with that of loess sediment artificially formed with a small degree of moistening, conducted with the help of electron-microscopic photographs, showed significant similarity. In
both cases the structural elements are globular micro-aggregates of silt size, with phase contacts between individual globules. Specimens formed with a high degree of wetting (water content = 25%), which shrank significantly in drying, have a different structure from the natural soil.

According to Korolev and Eremina, the development of the structural bonds is due to several factors;

1. dissolution of various chemical compounds, including polysilicic acid, on the interphase surface as a result of hydration, and precipitation of fine films of these substances on the contacts of solid particles;

2. formation of argillaceous "bridges" binding silt particles on account of the action of molecular and ion-electrostatic forces, as a consequence of the movement of argillaceous particles in the liquid component with a change in the curvature of the capillary meniscus in the course of the system's dehydration and their concentration in the zone of contacts of the silty components of the soil;

3. the occurrence of molecular and chemical interaction between particles at the point of their direct contact when under pressure from overlying strata of loess deposits;

4. "caking" of the sediment as a consequence of vertical mass transfer in loess strata, due to multiple repetition in natural conditions of variations in moisture potential in the zone of aeration, mostly in a vertical direction.

Korolev and Eremina come to conclusion that, as a result of drying of partially saturated silty sediment, forces of capillary cohesiveness give way to forces of a molecular, ion-electrostatic or chemical nature. Thus coagulation contacts are transformed into contacts of the cementation and transitional type. In this case, the collapsibility of loess strata accumulated by aeolian means in syngenetic, and autohesion cohesion forming with the course of time is
epigenetic. Trofimov (1990a) added to this conclusion that collapsibility as a syngenetic property of loess deposit is formed in natural conditions directly in the course of accumulation and early subaerial diagenesis of silty sediments. In the course of these processes many of the forces named above participate actively, and cohesion, which is syngenetic with respect to the time of its formation, arises. In the course of development of postgenetic processes, epigenetic changes in the amount of structural cohesion occur in loess deposits.

On the strength of this, the cohesion of loess sediments in strata of aeolian formations is both syngenetic and epigenetic.

Results of experimental simulation of the formation of the collapsible structure of silty aeolian genesis have also been obtained by Komissarova and Kolomiitsev (1987). A series of experiments was performed with the use of samples of light-loam, fine-sandy, Upper Pleistocone-Holocone loesses containing 37-58% fine sand and 17-57% silt particles; the clay content was 2-9%. Silty material formed as a result of pulverization of these natural loess soils and sieving the material through a 0.5mm sieve was filled into plastic rings with a volume of 280cm$^3$ to half their height. The initial density of all specimens was 1.15gm/cm$^3$. The sediment in the rings was wetted with water from the top by dripping, to a degree of saturation of 80% to 90%. After that a second portion of silty material was put on top of the moistened soil. Its subsequent moistening occurred on account of capillary rise of water from the lower, previously water-saturated part of the specimen. Then the wetted silty sediment was dried at a temperature of 80º-20º to an air-dry state, as a result of which the artificial loess sediment acquired cohesiveness. The porosity of the original air-dry sediment, which had been 57%, was decreased to 43-46% due to the effect of the process of wetting and drying. For the most part the soil specimens maintained their monolithic nature when they were dried. They were characterized by the presence of large interconnected pores between
particles, 32-64μm in size. Generally both the artificially formed and original specimens of natural soil had an identical skeletal structure. Collapsibility results showed that collapsible structures were formed in the course of modelling. The coefficient of collapsibility with a pressure of 100 kPa was 0.01 for specimens containing 2% clay minerals and 0.068 for silty sediment with a 9% clay content.

Trofimov, Vasil'eva and Kulikova used a different method of modelling the formation of collapsible aeolian deposits. The original material for these studies was two samples of coarse-silty, medium-loam, Upper Quaternary loesses, taken in the region of Alma-Ata at depths of 11 and 13m. The moisture content of the first sample was equal to 2.3%, porosity 43%, Liquid Limit 27, and plasticity index 8. For the second sample, the values of these indices were 1.9%, 44%, 26.5 and 7 respectively. Silty material obtained by pulverizing natural loess and subsequent sieving through a 0.5mm sieve was put into rigid, round, metal moulds with an area of about 400cm² in a layer 2cm high. This sediment was moistened with water from above through a filter paper to the moisture content of Liquid Limit. After that one mould with the given soil was dried through its upper plane at a temperature of 20°C for 4-5 days. Soil of the same composition in a second mould was dried for 8-12 hrs at a temperature of 40°C. Next it was kept for the same time in air at a temperature of 20°C and then it was heated to 40°C for the same time; the specimen was once again dried in air at a temperature of about 20°C and so on until the layer was completely dried out. After completing the first cycle of drying, a second layer of silty sediment 2cm high was put onto the first one, which was cracked as a consequence of shrinkage; it was wetted to the moisture content of the Liquid Limit and then dried as in the preceding series. Onto the dried-out soil once again was put a new layer of silty composition of the same thickness and the process of wetting and drying was repeated.
according to the procedure described above. As a result, four soil masses of loess deposits were formed, with an area of 400cm$^2$ and height of 6cm, fairly strong and monolithic in the lower and middle parts, and cracked to various degrees in the upper one. For each original specimen, one such artificial silty sediment was formed by drying at a temperature of 20°C, and a second artificial silty formation formed with the cyclic action of temperature of 40°C and 20°C. In the process of dehydration to an air-dry state (i.e. a moisture content of about 2%), the wetted silty sediment shrank, and thin cracks broke up the layer into large blocks. In spite of the shrinkage, a dense loess deposit could not form in these blocks. The density of their skeleton was 1.40-1.47gm/cm$^3$ and porosity 46-48%.

In collapsibility tests, the coefficient of collapsibility with a pressure of 30 kPa was 0.02-0.11; and with pressures of 100kPa and 300 kPa, were 0.08-0.15 and 0.19-0.20 respectively. In spite of the fact that the porosity of the silty sediments formed with different conditions of drying proved to be close, their collapsibility was not the same; collapsibility proved to be higher in the series of tests in which drying was carried out with the cyclic action of temperatures of 40° and 20°C.

In the light of the experimental data obtained by these various investigators Trofimov (1990) concluded that the formation of a collapsible structure of aeolian deposits was a syngenetic property, i.e. collapsibility is formed as a result of wetting of silty, highly porous, air-dry sediment and its subsequent drying in the course of subaerial lithogenesis. This property was formed in model specimens corresponding to an elementary layer of the upper part of a loess deposit mass. The degree of hydro-collapse of samples created from different regions was shown to depend on many factors. The effects of the
grain size and distribution of the silty sediment, the manner and degree of its wetting, and the conditions of subsequent drying were evident.

Finally, very important problems for further research have been outlined by Trofimov. He reported that priority problems for further experimental works on modelling of the formation of collapsible structure of loess deposits are as follows:

1. Further laboratory investigations are still needed to study the (apparently significant) effects of the degree of dispersion (grading) of chemical and mineral composition, rate of accumulation and thickness of an elementary layer of forming a silty sediment, the action of the weight of accumulating strata and the conditions of the wetting and drying process on the formation of collapsibility. There is no doubt that the rate of accumulation of aeolian silty sediment and its thickness have a significant influence on the formation of collapsibility. However, there are no such experimental data, and obtaining them in the course of physical modelling presents an interesting problem.

2. Special attention should be paid to the role of the chemical and mineral composition of aeolian sediment of porous metastable structure, since such data are lacking in the literature. This problem must be solved by performing experiments on prepared sediments of different sand-clay-silt mixtures of different mineral composition. In particular, it is necessary to investigate the role of clay minerals of different compositions and in different amounts in the sediment, as well as a wide range of primary mineral particles crushed to various sizes.
4.7 SUMMARY

* It is possible to model experimentally the formation of loess collapsibility of aeolian deposit. By means of physical laboratory modelling, it has been shown that collapsibility is formed as a result of wetting of silty, highly porous, air-dry sediment and its subsequent drying. Investigations performed with the use of silty material prepared from natural loess deposits have shown that the collapsibility properties depend on many factors including the grain size and distribution of silty sediment, type and properties of bonding materials, the manner and degree of its wetting, and the conditions of subsequent drying. Experimental data indicate that these effects are significant and further laboratory investigations are still needed.

* Special attention should be paid to the role of chemical and mineral composition of aeolian sediment in the formation of the porous metastable structure of loess deposits.

* An insight into metastable structure parameters, in particular the packing and bonding of particles, would certainly help the studies on hydro-collapse of soil structure. It would be useful to simulate both the initial metastable structure of open packing and the final stable structure of closed packing, and understand the spacial reconstruction which connect the two boundary conditions.

Finally it can be summarized that:

1. Soil structure proved more complex than expected, and it could not be satisfactorily described.
2. If studies of the metastable structure of an open packing are to progress, they should initially be concerned with relatively simple structures.

3. The airfall sedimentation of loess particles might be the best material on which to start a serious study of collapsible structure.
Table 4.1 The Classification of Loess Microfabric in the Loess Plateau of China. (after Lei, 1989)

<table>
<thead>
<tr>
<th>Microfabric Group</th>
<th>Group</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Weak - Cementation fabric group</td>
<td></td>
</tr>
<tr>
<td>1.1 Supporting macropore weak-cementation texture</td>
<td>I</td>
</tr>
<tr>
<td>1.2 Mosaic micropore weak-cementation texture</td>
<td>III*</td>
</tr>
<tr>
<td>2. Semi-cementation fabric group</td>
<td></td>
</tr>
<tr>
<td>2.1 Supporting macropore semi-cementation texture</td>
<td>II</td>
</tr>
<tr>
<td>2.2 Mosaic micropore semi-cementation texture</td>
<td>IV</td>
</tr>
<tr>
<td>3. Cementation fabric group</td>
<td></td>
</tr>
<tr>
<td>3.1 Flocculent cementation texture</td>
<td></td>
</tr>
<tr>
<td>3.2 Coagulum cementation texture</td>
<td></td>
</tr>
</tbody>
</table>

* The mosaic micropore weak-cementation texture is more stable compared with the support macropore semi-cementation texture.
Table 4.2 Various bonding types in loess deposit (after Yang, 1989)

<table>
<thead>
<tr>
<th>Kinds of Bonding</th>
<th>Hydro-stable cementation</th>
<th>Hydro-labile cementation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Calcium carbonate</td>
<td></td>
<td>Strongly soluble salts</td>
</tr>
<tr>
<td>Hydrous calcium sulphate</td>
<td></td>
<td>Clay cementing agent</td>
</tr>
<tr>
<td>Ferrous oxide and ferrous hydroxide</td>
<td></td>
<td>- Flocculent</td>
</tr>
<tr>
<td>Hydro-stable secondary mica</td>
<td></td>
<td>- Non-Flocculent</td>
</tr>
<tr>
<td>Secondary zeolite</td>
<td></td>
<td>Hydro-labile secondary mica</td>
</tr>
</tbody>
</table>

Table 4.3 Structural bonds in loess (after Osipov and Sokolov, 1995)

<table>
<thead>
<tr>
<th>Forces</th>
<th>Strength of contact between two particles, N</th>
<th>Resistance to water</th>
</tr>
</thead>
<tbody>
<tr>
<td>Molecular</td>
<td>5x10^{-10} to 10^{-8}</td>
<td>Gradually decrease with increase of water saturation</td>
</tr>
<tr>
<td>Ionic-electrostatic</td>
<td>5x10^{-8} to 4x10^{-7}</td>
<td>Unstable</td>
</tr>
<tr>
<td>Capillary</td>
<td>Under degree of water saturation 0.35 ≤Sr ≤0.8</td>
<td>Disappear under Sr ≤0.8</td>
</tr>
<tr>
<td></td>
<td>6x10^{-8} to 10^{-7}</td>
<td></td>
</tr>
<tr>
<td>Chemical</td>
<td>5x10^{-7} to 10^{-6}</td>
<td>Unstable, stable</td>
</tr>
</tbody>
</table>
Figure 4.1 Test Results, Illinois Samples, Load 1 and 2 tsf (1 tsf = 95.8 kPa)

Figure 4.2 Micro-structure Model of a Loess Deposit as Suggested by Tan (1988)
Figure 4.3 Double-Oedometer Test Curves of Loess Specimens with Different Initial Water Contents (after Phien-Wej et al, 1992)

Figure 4.4 Collapse Potential-Applied Pressure Relationship for Loess Deposit (after Phien-Wej et al, 1992)
Figure 4.5 Hydrocollapse Test Results for Loess Specimens from Thailand Loess (after Phien-Wej et al, 1992)

Figure 4.6 Comparison of Settlements from Plate Bearing (field) Tests and Predictions based on the Results of Double-Oedometer Tests (after Phien-wej et al, 1992)
Figure 4.7 Scanning Electron Photo-Micrograph (SEM picture) for Thailand Loess, Udomchoke, 1991), (From Phien-wej et al, 1992)
(a) Skeleton grains of sand and silts forming open structure
(b) Magnification of (a) showing clay bridge bonds at grain contacts
Figure 4.8 Hydrocollapse Test Results for Desert Loess (after Erol and El-Ruwaih, 1982)

(1 kg-force/cm² = 98.07 kPa)

Figure 4.9 The Relationship between the Collapsibility and Applied Pressure at time of Wetting for Desert Loess (After Erol and El-Ruwaih, 1982)
Figure 4.10 Schematic Diagram of a Honeycomb Fabric in a Silt

Figure 4.11 The Contents of the Large and Medium Pores Related to the Coefficients of Collapsibility of Loess
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Figure 4.13 Random Placement Structural Model after 45% Linear Collapse as Suggested by Smalley (1978).

Figure 4.14 Relationship between Cone Penetration Strength ($P_m$) and Degree of Water Saturation ($S_r$)
(a) full diagram.
(b) detailed diagram for $S_r > 0.6$
Figure 4.15 Four Ideal Bond Systems: Basic Structure of Blade Shaped Primary Mineral Particles, (Rogers et al, 1994)
(a) a small clay content     (c) a large clay content
(b) a small carbonate content (d) a large carbonate content
Figure 4.16 Strength - Deformation Relationship for Four Ideal Bond Systems (after Rogers et al, 1994)

* Large clay system has long-range bonds which allow deformation without major loss of strength
* Carbonate bonded systems are dominated by short-range bonds, strength drops rapidly after small deformation
CHAPTER 5
NATURAL AND ARTIFICIAL
LOESS MATERIALS

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5.2.2 Loess samples from China

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CHAPTER 5
NATURAL AND ARTIFICIAL LOESS MATERIALS

5.1 INTRODUCTION

In order to achieve the aims of this research, different soil materials have been experimentally investigated. Natural undisturbed and reconstituted loess samples have been collected from different locations in Libya and China. The main purpose of testing these materials is to compare the hydrocollapse test results of model loess samples with those results for real loess. This is in order to validate the airfall technique method used to create soil specimens of open metastable structure. In addition this project aims to examine fundamentally and classify the loess deposits in Libya, and to determine their collapse properties in both undisturbed and remoulded states. Artificial loess materials have been prepared by mixing pure silt particles of different geometrical properties with different clay types. Silt-clay (M-C) mixtures of various clay/silt (C/M) ratios have been produced. Solid quartz particles from Malan loess, and Ballotini glass balls (BGB) were used as a silt material. English China Clay and Wyoming Bentonite were employed as bonding materials. Two different methods of mixing were utilised: dry mixing and wet mixing methods. The index properties and chemical tests were carried out according to British Standard procedures (BSI, 1990). The geometrical characteristics and micro-structure features were examined using a scanning electron microscope (SEM technique).
5.2 NATURAL LOESS MATERIALS

5.2.1 Loess Samples from Libya in North Africa

Four soil samples representing two different types of loess were collected from different locations in Libya. Samples came from Khoms, Garabolli and Gharyan, representing Tripoli loess from the Jefara plain in the north west of Libya, and a further sample was taken from Tahala representing Ghat loess in the south west of the country. Khoms City is located 120 km east of Tripoli, Garabolli Town is 70 km east of Tripoli, Gharyan City is 80 km south of Tripoli and Ghat is located more than 1000 km south of Tripoli, as shown in Figure 5.1. Undisturbed block samples have been obtained from Khoms and Gharyan and disturbed samples from Garabolli and Tahala. All samples were transported from Libya by hand, having been individually put into polythene bags and carefully embedded in soil powder in very strong 250 x 250 x 200mm wooden boxes. The index properties of representative loess samples from Libya are shown in Table 5.1. The particle size distribution curves for Libyan loess are shown in Figure 5.2. The main features of the silt particles from Libyan loess are illustrated in Plate 5.1 to 5.3. The results of the chemical tests indicated that the Tripoli loess contains approximately 12% calcium carbonate, and the pH value is 8.

5.2.2 Loess Samples from China

Disturbed and undisturbed block samples of loess material were collected for experimental investigation from the Lanzhou area in China. The location of the soil samples used in this research is shown in Figure 5.3. Lanzhou is the
capital of the Gansu province region, located slightly to the south of the Huang Ho (Yellow River). It is in the west of the well-known central loess plateau. Three types of loess formation have been identified by Derbyshire and Mellors (1988) in the Lanzhou area. These different types of loess are categorised by age and are known as: Wucheng (early Pleistocene, Q1) loess, Lishi (middle Pleistocene, Q2) loess, and Malan (late pleistocene and Holocene, Q3) loess. Malan loess tends to dominate the Lanzhou landscape. The source of loess material is generally accepted as being the desert basins and silted wadis to the north and west of Lanzhou. Aeolian transport is considered responsible for the movement of materials. Soil samples from Malan loess are used in this investigation. The samples were carefully selected and hand-carved by Tom Dijkstra, an experienced researcher who is also working on the strength behaviour and properties of Chinese loess. The block samples were individually wrapped in cloth, embedded in loess powder, and stored in metal boxes of 200 x 200 x 200mm which were sealed with wax. The samples were transported from China with special care, so as to avoid disturbance. Dijkstra et al (1994) reported that experience has shown that this method of storage and transportation substantially preserves the samples in their field conditions, and they were maintained in this state until immediately before laboratory testing. The material can be described as a uniformly-graded, buff, coarse silt with most of the particles falling into the size range 10 - 50μm. The index properties of Malan loess are presented in Table 5.2. The grain size distribution curve of Malan loess is presented in Figure 5.4. Table 5.3 indicates that the major components of clay minerals in Malan loess are mainly illite and kaolinite (Jia-mao, 1985). The geometrical characteristics for the silt particles of Malan loess are shown in Plate 5.4.
5.3 ARTIFICIAL LOESS MATERIALS

5.3.1 Formation of Silt Particles

Leighton Buzzard sand was crushed in order to produce solid particles of silt-size. The main processes for deriving pure silt from crushed sand are crushing, cleaning and sedimentation. In addition to these stages, the sieving and drying processes were carried out where they were required. The procedures of these processes are explained in detail as follows.

5.3.1.1 Leighton Buzzard Sand

Garside Industrial Sand is a high quality silica sand. It had been quarried in the Leighton Buzzard area in Bedfordshire for around 100 years. Uniformly graded coarse-sand was delivered by CAMAS Aggregates Ltd and used as the original material for producing quartz grains of silt-size particles. Geologically, Leighton Buzzard sand has been classified as a Lower Green sand. Approximately 97% by weight of the material is quartz. It is a uniformly-graded coarse sand. The density of the solid particles is 2.65 Mg/m$^3$. Most of the solid particles are subrounded with sphericity of 0.85 (sphere = 1). The results of mechanical sieve analysis are illustrated in Figure 5.5.

5.3.1.2 Sand-Crushing Process

Silt particles are mostly rock flour, finely ground materials of quartz mineral. They are cohesionless and non-plastic. Soil materials of silt (2-63 μm) size range comprise one of the main detritus sediments and form the widespread aeolian and alluvial deposits of significant engineering importance. The mechanisms of silt formation are still being discussed (Smith, 1994; Smalley, 1990), as explained in Chapter 3. Experimental tests have found that silt
particles of different sizes can be produced by crushing quartz grains using different mechanical processes (Smalley, 1966a; Wright, 1995; Kanda et al, 1990). Solid particles of silt size have been produced by crushing oven dried clean Leighton Buzzard sand. Different types of milling machines were employed. Each crushing method simulates a certain kind of natural comminution process. For example, the end runner mill is used to simulate the glacial grinding mechanism, the ball mill to model the particle impact processes, and the compression machine to mimic the different types of natural compressive forces. All these methods have produced silt particles of similar characteristics. Angular and semi-angular particles of sharp edges and blade shape were formed (see Plate 5.5). More than 70% of these particles fall in the range 10-50μm, which forms the loessial fraction as defined by Browzin (1985). The only difference between these crushing mechanisms is the amount of silt material which can be derived by each method in a certain time. In an attempt to simulate the actual subglacial comminution processes, an end-runner mill was used for the purpose of this research. It was felt to be closer to conditions experienced by glacial environments than the other methods. This grinding technique was already successfully used by Smalley (1966a) to produce model loess particles. The end-runner mill produces very effective disintegration of the sand grains. In this method the automatic pestle and mortar apparatus was used to grind the clean dry sand, as shown in Figure 5.6. The sand grains were crushed between the rotating dish container and the revolving miller. Different grinding times (1, 2, 4 hrs) were tried. It was found that the 2 hr grinding time was the most suitable. In the case of 1 hr grinding the quantity of silt material was too small, and in the case of the 4 hr grinding the silt material was much finer and contained a lot of metallic contamination. The grinding process was repeated many times until the required amount of silt material was obtained.
5.3.1.3 Magnet-Cleaning Process

In order to clean the crushed sand from metallic contamination a magnetic method was employed. In this technique a magnet which was attached to the end of a plastic handle was used. About 200gm of the crushed sand was placed in a 1000 ml beaker. The beaker was then filled with distilled water and stirred thoroughly by glass rod in order to distribute the material evenly throughout the solution. Immediately after stirring and whilst the soil-water suspension was still in motion the magnet was inserted into the middle of the beaker. The magnet was carefully moved in a rotary motion around the solution to collect the suspended metallic material for approximately 10 seconds. Care was taken to ensure that it did not touch the sides of the beaker during this process. The magnet was then removed and cleaned using tap water and a brush. The procedure was repeated until no further metallic material was collected on the magnet. The whole cleaning procedure takes about one hour.

5.3.1.4 Gravity-Sedimentation Process

The crushed sand is a mixture of sand, silt and clay-sized particles. The separation of silt and clay-sized materials from crushed sand was achieved by sieving the material through the 63μm sieve. The clay-sized particles were washed out and separated from the silt-clay mixture using the sedimentation phenomenon. According to the sedimentation theory the silt particles in soil-water suspension settle more quickly than clay-sized particles. Mitchell (1993) reported that separation of the particles (finer than 2μm), or other different small sizes of silty soil, can be conducted by sedimentation process. He stated that the times required for solid particles of 2, 5, and 20μm equivalent spherical diameter to fall through water a distance of 100mm are about 8 hrs, 1.25 hrs and 5 minutes respectively, at 20°C. At 30°C the times needed are about 6.5 hrs, 1 hr and 4 minutes. Table 5.4 shows the terminal velocities of solid particles in soil-water suspension as presented by Head (1980). Based on the
values given by Mitchell and Head, the relationship between the particle size and the time required to fall through water a distance of 100mm was plotted as shown in Figure 5.7. From this figure, the times required to sediment out 2.5 and 3.0μm particle sizes are approximately 5 hrs and 3.667 hrs respectively. In the sedimentation process the solid particles are allowed to settle down under gravity. In this method, about 200 gm of the silt-clay mixture was placed in a 1000ml Gas Jar and distilled water was added to the soil material to form about 1000ml of soil-water suspension. The suspension was treated (to neutralise the soil-particle charges) with about 100ml of a standard dispersant solution. For wet sieving of silty or clayey soils the British Standard specifies mixing the soil with distilled water containing 50ml of standard dispersant solution per litre. For hydrometer sedimentation tests British Standard specifications recommend the addition of 100ml of the standard dispersing solution to the soil-water mixture to make 1000ml.

The silt particles settled quickly to the bottom of the beaker and were thoroughly shaken for about 4 hrs using the mechanical end-over-end shaker to ensure that discrete solid particles were completely separated. The standard dispersant solution as recommended by the British Standard (BSI 1990) was made up of: 33gm sodium hexametaphosphate (NaPO3), 7gm sodium carbonate (Na2CO3), and distilled water to make 1000ml of solution. After mixing, all the soil-water mixture was transferred to a 1000ml glass beaker and allowed to stand for about 4 hrs to ensure that all the silt-size particles had settled down at the bottom of the beaker. At the end of the sedimentation period, the clay-water solution was carefully transferred into another beaker. The beaker was again filled with distilled water and the soil-water mixture was vigorously agitated with a glass rod until all particles went into suspension and a uniform soil-water solution was obtained. Then the solution was again left for about 4 hrs to allow the silt particles to settle down at the bottom of the
beaker. The clay solution on top was carefully poured into another beaker without disturbing the silt material at the bottom. This process was repeated until the clay particles were completely washed out and the top solution was wholly clear. The silt particles remaining in the beaker were then dried in the oven for about 24 hours.

5.3.1.5 Gradation Process

Two different kinds of particle gradings have been prepared by mixing pre-determined quantities of silt particles of different (2-20μm, 20-32μm, 32-45μm, 45-63μm) size ranges. These silt fractions were produced by dry sieving using a set of 20μm, 32μm, 45μm and 63μm sieves. The fractions of the three coarser size ranges were cleaned from those particles which have sizes smaller than these fractions by using the sedimentation method as explained in Section 5.3.1.4. As shown in Figure 5.7 the times required for particles of 20, 32, 45μm to settle in water a distance of 100mm are about 5, 2 and 1.05 minutes respectively. The types of gradings are as follows:

(a) typical grading, in which the grain-size distribution is similar to the distribution of typical Malan loess. The sieve analysis results for this grading performed using the sedigraph technique is presented in Figure 5.4.

(b) coarse-grading, which contained no materials less than 20μm, was produced mainly to simulate the grading of the Ballotini glass balls, as shown in Figure 5.4.

5.3.2 Pure Silt from Chinese Loess

Natural silt particles have been derived from Malan loess in China. The loess material was first wet sieved through a 63μm sieve. The clay particles were
then washed out and separated using the sedimentation method as explained in Section 5.3.1.4. The grain size distribution curve is shown in Figure 5.4. The average value of the specific gravity test results is 2.70.

5.3.3 Ballotini Glass Balls (BGB)

In order to investigate the effect of grain shape and grain surface texture on the hydrocollapse behaviour of loess deposits, small transparent lead glass spheres (Ballotini) have been used for this research. Ballotini balls are manufactured by a variety of processes dependent on the final diameter required. The consistent sphericity of the resultant balls is related to the original particles of glass (usually obtained by crushing larger glass lumps) and the actual melting process used. A small percentage of irregular shaped pieces may exist. The results of the grain size analysis test using the sedigraph technique, showed that about 90% of the particles were in the range 20-60μm, the loessial fraction (10-50μm) as defined by Browzin (1985) being about 70% of the whole material (see Figure 5.4). More than 85% (60% of the whole material) of this fraction is 30-50μm in diameter. It can be seen from these grain-size distribution results that there are no particles less than 20μm (see Figure 5.13). Thus the Ballotini glass balls can be classified as a coarse silt, based on the British Standard classification system.

Ballotini glass balls are available in two types: soda glass and lead glass. The physical characteristics of the lead glass Ballotini used in this work are presented in Table 5.5, and their chemical analysis is shown in Table 5.6. The results of the SEM analysis indicated that the Ballotini glass balls are spherical, as shown in Plate 5.6.
5.3.4 Bonding Materials

5.3.4.1 Introduction
Two types of forces act on the soil particles.
1. self weight or gravitational forces; and
2. surface or physico-chemical forces.
Physico-chemical forces (surface forces) occur because soil particles carry an electrostatic charge (negative charge). The magnitude of this charge depends on the mineral type and the chemical properties of the pore fluid. The two types of force are related to the size of the particle: gravitational forces are proportional to a particle volume whereas surface forces are proportional to the surface area (Terzaghi and Peck 1967; Atkinson and Bransby 1978). Thus as the size of the soil particle reduces the surface forces become more dominant, and as a consequence control the engineering properties and behaviour of fine-grained soils like clays. Therefore, clays can be considered to be electro-chemically active and can play an important role on the physical and mechanical properties of soils even when present in small amounts (Grim, 1968; Jefferson, 1994).

5.3.4.2 Clay Minerals
The most common bonding materials in loess deposits are clay minerals and calcium carbonate (CaCO3). A knowledge of the mineralogical composition of loess constituents can be of great value for understanding and explaining the collapse properties and behaviour of loess deposits. The term clay is used to refer both to a size and to a mineral type. As a size term it refers to all soil particles smaller than 2μm, and this classification is used for engineering applications. As a mineral term it refers to specific clay minerals which are characterized by the following properties.
1. Very small particle size
2. A net negative electrical charge
3. Plasticity when mixed with water
4. High weathering resistance

The size and shape of soil particles are the main factors responsible for the cohesive properties of the clay minerals. Individual clay particles are flat and plate-like, which gives them very high specific surface areas (the total surface area of solid particles per unit mass, expressed as m²/gm), depending on the type of clay mineral. Typical values of specific areas for the three most common types are approximately 10 m²/gm for kaolinite, 100 m²/gm for illite and 1000 m²/gm for montmorillonite (Lambe and Whitman, 1979).

Clay minerals may be classified into three principal classes with respect to the engineering property which shows the widest variation between clay types - the swelling and shrinkage characteristics (susceptibility to volume change) of a clay: low to very low (e.g. kaolinite), moderate (e.g. illite) and high to very high (e.g. montmorillonite).

Kaolinite and montmorillonite are the most commonly occurring clay minerals. Although the basic structure of these two minerals is composed of the same two units (the silica tetrahedron and the alumina octahedron), their swelling and shrinkage behaviour is significantly different. The structural difference between the two clay minerals is that kaolinite is composed of alternate layers of silica tetrahedra and alumina octahedral units, whereas the montmorillonite is composed of two silica tetrahedra to every octahedral unit. As a result of this structural difference, the engineering properties are largely different. It can be noted that the particle size of kaolinite is considerably larger than montmorillonite, and that the cation exchange capacity is a small fraction of that of montmorillonite.
5.3.4.3 English China Clay

China clay is commonly known as kaolin, relating to Kaolin in the north west of China, the place where it was first used as an industrial material to produce a good quality porcelain. Samples of English China Clay have been used in this research to produce silt-clay mixtures. This clay was supplied by Whitfield Ceramic Materials Ltd in a dry powdered form identified as 50 China Clay (Airflo). According to the plasticity results kaolinite can be classified as a low plasticity clay (see Table 5.7). The average value of the specific gravity test results is 2.62. About 90% of the clay particles are less than 5\(\mu\)m and about 50% are less than 2\(\mu\)m. The physical and chemical properties of the English China Clay are represented in Table 5.7. The grain size distribution results are shown in Figure 5.4. The mineralogical composition and chemical analysis as presented by the manufacturer are illustrated in Tables 5.8 and 5.9 respectively.

5.3.4.4 Wyoming Bentonite

Wyoming Bentonite is a very high plasticity clay. It was first found near Fort Benton, in the Cretaceous beds of Wyoming in the U.S.A. (Grim 1968). There are numerous uses for Bentonite because of the excellent performance of the material as a binder, suspension agent and sealant. Because of the industrial importance of this clay mineral, a considerable amount of work has been done to investigate the engineering properties of this material.

Light grey powder clay samples of Wyoming Bentonite were purchased from Steetly Bentonite & Absorbents Limited and used in this study as a bonding material. The mineralogical analysis results as presented by the manufacturer showed that the Wyoming Bentonite consists of 92% montmorillonite, 5% feldspars and 3% quartz. Montmorillonite particles may occur as equi-dimensional flakes. They are very small. The thickness of these particles
ranges from 10 Å to about 1/100 of the width, and their long axis is usually less than 1 or 2μm (Mitchell, 1993). This type of clay mineral has a very large specific surface area, and it can be up to 840 m²/gm (Mitchell 1993). Montmorillonite has a very high susceptibility to volume change (swelling and shrinkage). It has a Liquid Limit (LL) of 680% and a plasticity index (PI) of 550%. The chemical analysis of Wyoming Bentonite obtained from the manufacturer is shown in Table 5.9. The physical properties are shown in Table 5.10.

5.3.5 Artificial Loess (Model Loess)

5.3.5.1 Introduction

As explained previously the compositional properties and structural characteristics are the main principles which determine the definition of the typical loess. Compositionally, loess material consists mainly of silt particles in which more than 50% of the particles fall in the range 10-50μm (Browzin, 1985). These particles are normally held together in an open structure by a certain amount (10-20%) of clay content. It was concluded that more attention should be given to the need for investigation of the role of the compositional parameters in hydro-collapse properties and behaviour of loess deposits. Size, shape and surface texture of silt particles and type and amount of clay minerals are of the most importance. At present such studies are very limited in the literature. This problem can be solved by carrying out experimental tests on prepared soil mixtures with different mineral compositions (qualitatively and quantitatively) of silt-clay dimensions.

In order to simulate the compositional constituents of typical loess, silt particles of different origins and geometrical characteristics and a variety of
clay minerals were mixed together. Different proportions of silt and clay were used to make a variety of (silty-clay) mixtures of different clay/silt (C/M) ratios. According to the different materials used, the silt clay mixes can be identified as follows:

5.3.5.2 Pure Silt from Crushed Sand-Kaolinite (Mcs-K) Mixes:
Pure silt from crushed sand was mixed with kaolinite to produce a silt-clay mixture of different clay/silt (C/M) ratios. Twelve ratios (0/100, 5/95, 10/90, 15/85, 20/80, 25/75, 30/70, 35/65, 40/60, 45/55, 50/50, 60/40) have been used.

5.3.5.3 Pure Silt from Crushed Sand-Bentonite (Mcs-B) Mixes:
Different amounts of smectite were added to various quantities of pure silt from crushed sand to make up six (silt-clay) mixtures. The clay/silt ratios used were 5/95, 7.5/92.5, 10/90, 12.5/75.5, 15/85, 20/80. Wyoming Bentonite was used here as a source for the montmorillonite clay mineral.

5.3.5.4 Ballotini Glass Balls-Kaolinite (BGB-K) mixes
Smooth spherical solid particles of silt size were mixed with china clay to form several (silt-clay) mixtures of different clay/silt (C/M) ratios (10/90, 15/85, 20/80, 25/75, 30/70, 35/65, 40/60, 45/55, 50/50).

5.3.5.5 Pure silt from Chinese Loess - Kaolinite (Mml-K) Mixtures
Three (silt-clay) mixes of 20/80, 30/70, and 40/60 clay/silt (C/M) ratios have been prepared by mixing china clay with pure silt particles derived from Malan loess in China.
5.3.6 Mixing Methods

Two methods of mixing have been utilized: dry-mixing and wet mixing methods. The purpose of this mixing process is mainly to distribute uniformly the clay minerals in between the silt particles in order to produce a homogeneous artificial loess material in the form of silt-clay mixtures. The effect of the bonding system on the collapse behaviour of the (silt-clay) mixes depends largely on the degree of clay particles distribution.

5.3.6.1 Dry-mixing method
The silt and clay fractions were placed in a mortar and rubbed together very well manually by hand using a rubber pestle for 10 minutes. This mortar and rubber pestle method was described in the 1967 British Standard and recommended as an effective method for dispersion of the soil particles in a soil suspension. The mixture was then sieved through a 63μm sieve. The mixed material was again placed in the mortar and mixed very well for about 20 minutes. Care was taken to avoid breakdown of individual particles. Finally the (silt-clay) mixture was sieved again through a 63μm sieve.

5.3.6.2 Wet-mixing method
In this method the silt and clay particles were mixed together thoroughly using the dry-mixing method. The (silt-clay) mixture was then transferred to the mortar and distilled water was added to form a soil-water slurry at approximately the Liquid Limit. The slurry was then mixed very well by hand using a rubber pestle for about 10 minutes. The wet soil was then placed on a glass plate and partial evaporation of water was permitted. This procedure was continued for about 10 minutes until a soil of plastic state was formed. Finally the soil lumps were oven-dried and sieved through a 63μm sieve.
5.4 INDEX PROPERTIES

Representative soil specimens from different loess materials were tested for solid particle density (numerically equal to specific gravity), plasticity properties (Atterberg Limits), and grain-size distribution according to British Standard procedures (BSI, 1990). The grain size analysis for soil materials passing the 63µm sieve was conducted using the sedigraph technique.

5.4.1 Sedigraph Technique

A sedigraph 500ET particle size analyser was used. Soil specimens were first treated with a standard dispersant solution of sodium hexametaphosphate and sodium carbonate as explained in Section 5.3.1.4. This technique uses the same sedimentation principles based on the Stoke's law as discussed previously but it has the advantage that the testing is conducted in a more carefully controlled environment. It also analyses specimens via finely collimated beams of low X-ray energy. This allows quick calculations of equivalent spherical diameters and gives relatively accurate results.

5.4.2 Index Properties of Libyan Loess

A summary of index properties for representative specimens of soil samples collected from different locations in Libya is presented in Table 5.1, from which the variation in properties between Tripoli loess and Ghat loess can be easily recognised. The values of the specific gravity of Tripoli loess fall within a very narrow range (2.66 - 2.68) and they are below the specific gravity of Ghat Loess (2.73). Generally the specific gravities of Libyan loess are very similar to those for loess from many parts of the world. The specific gravity of
loess in the United States, as reported by Sheeler (1968), varies between 2.57 and 2.79. The range is much smaller in local areas. For example, in Iowa State it is from 2.68 to 2.72, in Tennessee from 2.65 to 2.70 and in Mississippi from 2.66 to 2.73. For Chinese loess the values vary from 2.65 to 2.70, as reported by Bell (1992). The specific gravity values for loess from Kent in the United Kingdom vary from 2.68 to 2.70, as stated by Derbyshire and Mellors (1988).

Dry density values for Khoms soil in the range between 1.30 Mg/m³ and 1.50 Mg/m³ for the upper 5.0 metre thick layer have been reported by Anagnosti (1973). Undisturbed soil specimens from Khoms and Gharyan have been tested in the laboratory and dry densities range between 1.40 Mg/m³ and 1.46 Mg/m³, and 1.36 Mg/m³ and 1.41 Mg/m³ respectively. The corresponding void ratios are from 0.84 to 0.91 for Khoms samples and from 0.89 to 0.96 for Gharyan samples. On the basis of the collapse settlement criteria developed by Holtz and Hilf (1961), presented in Table 5.11, the Tripoli loess can be considered as of medium density and moderately susceptible to collapse settlement, particularly for critical or heavily loaded structures.

The plasticity properties for samples from Tripoli and Ghat loess have been determined and presented in Table 5.1. The results can be seen to fall into two groups: the Ghat Loess group has a plasticity higher than that of Tripoli loess. The Liquid Limits and Plasticity Indices of Tripoli loess are from 25 to 31% and from 8 to 11% respectively, whereas for Ghat loess the Liquid Limit is 41% and Plasticity Index is 17%. Loess soils are slightly to moderately plastic, the plasticity increasing as clay content increases. Gibbs and Holland (1960) indicated that Liquid Limits for silty loess varied from 25 to 35%, with values for clayey loess ranging up to 45%. All the plasticity values of Libyan loess plot above the A-line and before the vertical line of 50% Liquid Limit on the Casagrande plasticity chart, which indicates that the material is an inorganic
soil of low plasticity (see Figure 5.8). Soils of low plasticity are generally very sensitive to changes in moisture content. Gibbs and Holland (1960) classified loess material into three types based on plasticity characteristics: sandy loess, silty loess and clayey loess, as shown in Figure 5.8. Test results for samples from Libyan loess appear to plot within the zones determined by Gibbs and Holland (1970).

Generally, more than 50% of loess particles are of silt size. Holtz and Gibbs (1951) have investigated the Missouri Basin Loess and found that most of the grains were between 19 and 74\(\mu\)m and more than 70% of the samples were classified as a silty loess. They similarly divided the material into sandy loess, silty loess and clayey loess and produced an envelope of grading curves for each group. The grain size characteristics of Libyan loess are presented in Figure 5.2. It can be seen that all of the grading curves fall predominantly into the zones of loess-types defined by Holtz and Gibbs (1951), as shown in Figure 5.9. The majority of the materials from Khoms and Gharyan fall broadly into the silty loess group, whereas the samples from Tahala loess predominantly fall into the clayey loess group. According to both the plasticity and grading results, therefore, the Libyan loess can be divided into two groups: silty loess from the North West in the Tripoli region (Jefara Plain) and clayey loess from the South West in the Ghat region (Tahala area).

5.4.3 Index Properties of Malan Loess in China

The index properties for representative samples from Malan loess are summarized in Table 5.2. The specific gravity of Malan loess is 2.72, which is slightly higher than average. The specific gravity of loess material in the world
generally ranges from 2.65 to 2.75, with an average value of 2.70, as reported by Jing (1989) and shown in Figure 3.6.

Derbyshire and Mellors (1987) reported that typical loess has a dry density of about 1.4 Mg/m³ but it can range from 1.15 to 1.75 Mg/m³. The dry density values for Malan loess were found in the range of 1.34 to 1.4 Mg/m³, which are thus close to the value of typical loess. The corresponding void ratios for these density values are 1.03 and 0.94 respectively.

The Atterberg Limits of Malan loess are similar to those limits of silty loess as defined by Gibbs and Holland (1960). The Liquid Limit of Malan loess is 29 and the plasticity index is 12, compared with a range of 25 to 35 and 5 to 22 per cent respectively.

The general shape of grading envelope for Malan loess as presented by Derbyshire and Mellors (1988) is shown in Figure 3.10. Sedigraph results for representative specimens from Malan loess are shown in Figure 5.4. Figure 5.10 presents the histogram of the particle-size distribution for soil sample from Malan loess. From these figures it can be seen that more than 70% of the particles fall into the size range 10-50μm and it can thus be described as a coarse silt with a nominal diameter of around 20-30μm. The percentage of clay minerals is about 12 per cent.

5.4.3 Index Properties of Pure Silt

The solid particle density of the pure silt from crushed sand was determined according to British Standard procedures (BSI, 1990). The average value of the test results obtained was 2.66 Mg/m³ which is very close to the value
reported in the literature for similar material, for example the density of solid quartz particles is 2.65 Mg/m³.

It has been reported that typical unweathered loess is composed mainly of solid particles between 10 and 50 μm in size, and usually has a prominent modal size between 20 and 30 μm (Browzin, 1985; Tsoar and Pye, 1987).

The results of a sedigraph analysis for pure silt from crushed sand are presented in Figure 5.4, from which the uniformity coefficient \( (C_u) \) is approximately 2.80 and curvature coefficient \( (C_c) \) is approximately 1.16. Based on the particle-size classification given by British Standards (BSI, 1990) the material consists of 64% coarse silt, 30% medium silt, 4% fine silt and only 2% of clay-sized particles. Hence it can be described as a uniformly-graded coarse silt. A comparison between the grain-size distribution curve for pure silt from crushed sand and that curve for pure silt from Malan loess can be done from Figure 5.4, from which it can be concluded that the characteristics of pure silt from crushed sand are similar to those for silt derived from typical loess. Figures 5.11 to 5.14 show histograms of particle-size distributions for pure silts from different materials. From Figures 5.11 and 5.12 it can be seen that for pure silt from Malan loess and pure silt derived from crushed sand, the concentration of material is in the size range 10-40 μm, where more than 75% of the soil particles fall. Another important characteristic presented in Figures 5.11 and 5.12 is that the material has a silt mode size of approximately 25 μm. For ballotini glass balls and pure silt having similar grading, the concentration of the material is in the size range 30-60, and they have a prominent modal size between 40 and 50 μm.
5.5 SCANNING ELECTRON MICROSCOPE ANALYSIS

5.5.1 Introduction

In this research the geometrical characteristics (shape, size, angularity) of the individual silt particles of different Loess Materials were studies using SEM techniques following cleaning by treatment with Hydrochloric Acid (HCl). The same technique was also used to investigate the structural features of the natural and model loess materials.

5.5.2 SEM Technique

Scanning electron microscopy (SEM) is widely used as an analytical tool. SEM analysis can be conducted on a wide variety of materials, for example individual particles, aggregated materials, pieces of rock, core samples, drill cuttings and thin sections (Welton, 1984). The most important requirement is that the specimen must be small enough to fit into the SEM sample chamber. It was successfully used in this project to investigate:

1. grain shape and texture,
2. soil fabric and details of the interparticle pore structure,
3. the identification of the soil minerals, and the distribution of clay minerals within the soil system.

Other advantages of the SEM tool over optical petrograph are ease of sample preparation, resolution, and a higher magnification range. In addition, less training is required to interpret the SEM micrograph.

The scanning electron microscope consists mainly of an electron optics column and an electronics console (see Figure 5.15). The coated SEM sample is placed in the sample chamber, in the electron optics column and evacuated to
high vacuum (approximately up to $5 \times 10^{-6}$ torr) A Hitachi-made S-520 scanning electron microscope of very high magnification (20x to 200,000x) has been used to investigate the shape features of the solid particles of different loess materials, the distribution of clay minerals in-between the silt particles in the silt-clay mixtures (artificial loess), and the identification of the clay minerals that exist in Libyan loess. For geometrical characteristics (shape, size and angularity) observation, the silt particles were firstly cleaned by treatment with Hydrochloric Acid (HCl). For the purpose of determining the nature and properties of the interparticle contacts and pore spaces and to identify the clay minerals, soil specimens were prepared from oven-dried blocks by gently breaking off a small piece of material. The specimens prepared were attached to the SEM sample plug and the free soil particles which were existing on the surface of the sample were carefully removed by using a jet of compressed air (a Freon duster). In order to obtain a clear image and to avoid the problem of specimen charging, the specimens were coated with gold (a very good conductive material) by using the E5200 Auto Sputter Coater. After coating, the specimens were placed in the sample chamber in the electronic optics column and evacuated to a vacuum of approximately $10^{-5}$ torr. The samples were then ready for the SEM analysis. The scanning results of the SEM analysis are illustrated in Plates 5.1 to 5.14.

Elemental and qualitative analyses were made possible by the use of an Energy Dispersive X-ray (EDX) Analyser Unit attached to the SEM machine (see Figure 5.15). Qualitative analysis for clay minerals in Libyan loess has been carried out using the EDX system. The results of the EDX and XRD analyses suggest that the Tripoli loess has inactive clay minerals, such as kaolinite or illite.
5.5.3 Geometrical and Structure Features of Loess Materials

The shape, size and angularity are the main geometrical features of the solid particles of soil mass. The geometrical properties of solid grains of loess material depend mainly on their mineral composition and crystallographic structure, their formation mechanisms and the modifications induced by weathering and diagenesis processes. Pye (1987) reported that the quartz particles of silt size are mostly blocky and angular or sub-angular. Quartz grains formed by sub-glacial grinding or crushing by frost action often have sharp edges and show stepped fracture surfaces, whereas quartz particles derived from active chemical weathering environments often display edge rounding due to solution and reprecipitation.

Recent investigations carried out by Smalley and Glendinning (1991) and Mahaney (1992) indicated that the study of individual loess particles may give useful information on material formation and transport mechanisms. Rogers and Smalley (1993) reported that the most important single-particle problem which needs to be addressed is that of the structure of loess deposits and their collapse behaviour on loading and wetting. The shape of loess particles has also been studied by Rogers and Smalley (1993) who state that in order to build soil models of loess structure one needs to know the general shape of the basic building unit (e.g. $30 \mu m$ quartz particles). They concluded that the basic shape is more important than the angularity, weathering status or surfacial features.

Plates 5.1 to 5.5 show the SEM pictures for silt particles from different soil materials. It can be seen that most of the particles are blade shaped, as described by Smalley (1966) who classified particle shape into four Zingg shape categories. It is also clear that the predominant particles are angular or semi-angular, particularly the particles from the crushed sand.
Plates 5.1 to 5.4 show the shape of the silt particles derived from Libyan and Chinese loesses. Most of the silt particles are of blade shape and angular and sub-angular. Also from these figures it is clear that both the silt materials have similar geometrical characteristics.

The shape properties of the pure silt from crushed sand and the Ballotini glass balls are shown in Plates 5.5 and 5.6. These photographs illustrate the degree of angularity of the crushed sand-pure silt and the degree of roundness of the spherical balls. It can also be seen that the geometrical properties of the crushed sand-silt are similar to those for silt particles derived from natural loess materials.

The characteristics of the micro-structure for the natural undisturbed loess materials from Libya are illustrated in Plate 5.7. This picture shows the size and distribution of the pore spaces inside the soil mass and reveals that the Libyan loess is of an open metastable structure having a considerable percentage of large pores (20-80μm). The silt particles are partially coated by the clay and calcium carbonates which form the major bonding materials.

Plates 5.8 and 5.9 show the distribution of the clay particles inside the loess material for pure silt-kaolinite (MCS-K) mixes having a 30/70 K/MCS ratio prepared by different mixing methods. From these photographs it is clear that the wet mixing method gives better results of clay particle distribution. In the case of the dry mixing method most of the silt particles are clean and the clay material is concentrated in certain places, whereas in the case of the wet mixing method the silt particles were partially coated by the well-distributed clay particles, giving a high percentage of the silt particle contacts supported by clay minerals as a bonding material.
The micro-structural features for the model loess are shown in Plates 5.10 to 5.14. The aspects of the open packing metastable structure are shown in Plates 5.10 to 5.12, from which the pore size and pore distribution can be noticed as well as the distribution of the clay materials in between the silt particles. Very large pores (> 50µm) can be seen in these photographs. The silt particles are held together by means of clay cementation in the form of clay bridge bonds, while some portion of the clay material is in the form of a surface coating on the silt grains.

Plates 5.13 to 5.14 show the micro-structural features of dense specimens prepared from different BGB-K mixes. These pictures illustrate how the loess fabric changes by changing the clay content in the mix. For example in the specimens of 20/80 C/M ratio the silt particles are partly covered by the clay materials, with a relatively high percentage of contact points bonded by the clay minerals. The clay material did not fill all the void spaces in the soil. For the specimens having 40/60 C/M ratio most of the silt particles were covered by the clay particles and most of the pore spaces were filled with the finer fraction.
### Table 5.1 Index properties of Libyan loess

<table>
<thead>
<tr>
<th>Property</th>
<th>Sample Locations</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>GS</td>
</tr>
<tr>
<td>Natural moisture content (%)</td>
<td>6</td>
</tr>
<tr>
<td>Specific gravity (SG)</td>
<td>2.66</td>
</tr>
<tr>
<td>Dry density (Mg/m³)</td>
<td>1.38</td>
</tr>
<tr>
<td>Void ratio</td>
<td>0.93</td>
</tr>
<tr>
<td>Porosity (%)</td>
<td>0.48</td>
</tr>
<tr>
<td>Grain size distribution</td>
<td></td>
</tr>
<tr>
<td>Sand (%)</td>
<td>38</td>
</tr>
<tr>
<td>Silt (%)</td>
<td>51</td>
</tr>
<tr>
<td>Clay (%)</td>
<td>11</td>
</tr>
<tr>
<td>Atterberg Limits</td>
<td></td>
</tr>
<tr>
<td>Liquid Limit (%)</td>
<td>27</td>
</tr>
<tr>
<td>Plastic Limit (%)</td>
<td>19</td>
</tr>
<tr>
<td>Plasticity Index (%)</td>
<td>8</td>
</tr>
</tbody>
</table>

**Legend**
- GS - Gharyan Soil (undisturbed)
- KS - Khoms Soil (undisturbed)
- GBS - Garaboli Soil (disturbed)
- TS - Tahala Soil (disturbed)

### Table 5.2 Index Properties of Chinese Loess

<table>
<thead>
<tr>
<th>Property</th>
<th>Malan loess</th>
</tr>
</thead>
<tbody>
<tr>
<td>Natural moisture content (%)</td>
<td>4-6</td>
</tr>
<tr>
<td>Specific gravity (SG)</td>
<td>2.72</td>
</tr>
<tr>
<td>Dry density (Mg/m³)</td>
<td>1.36</td>
</tr>
<tr>
<td>Void Ratio</td>
<td>1.02</td>
</tr>
<tr>
<td>Porosity (%)</td>
<td>50%</td>
</tr>
<tr>
<td>Grain size distribution</td>
<td></td>
</tr>
<tr>
<td>Sand (%)</td>
<td>5</td>
</tr>
<tr>
<td>Silt (%)</td>
<td>83</td>
</tr>
<tr>
<td>Clay (%)</td>
<td>12</td>
</tr>
<tr>
<td>Atterberg Limits</td>
<td></td>
</tr>
<tr>
<td>Liquid Limit (%)</td>
<td>29</td>
</tr>
<tr>
<td>Plastic Limit (%)</td>
<td>17</td>
</tr>
<tr>
<td>Plasticity Index (%)</td>
<td>12</td>
</tr>
</tbody>
</table>
Table 5.3 Quantitative Clay Mineralogical Analyses of Chinese Loess
(after Jia-Mao, 1985)

<table>
<thead>
<tr>
<th>Number of sample</th>
<th>Name</th>
<th>Clay minerals</th>
<th>Nonclay minerals</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>K I M Ch V</td>
<td>C G</td>
</tr>
<tr>
<td>81-G-01</td>
<td>Malan Loess</td>
<td>16.80 48.70</td>
<td>3.74 5.74 4.69</td>
</tr>
<tr>
<td>02</td>
<td>Malan Loess</td>
<td>14.80 45.45</td>
<td>4.34 5.42 5.02</td>
</tr>
<tr>
<td>03</td>
<td>Paleosol</td>
<td>18.20 57.41</td>
<td>7.92 2.60 7.93</td>
</tr>
<tr>
<td>04</td>
<td>Lishi Loess</td>
<td>18.49 42.09</td>
<td>10.77 5.26 4.82</td>
</tr>
<tr>
<td>05</td>
<td>Paleosol</td>
<td>20.80 53.60</td>
<td>8.41 5.35 4.82</td>
</tr>
<tr>
<td>06</td>
<td>Lishi Loess</td>
<td>12.97 55.74</td>
<td>5.31 4.50 5.30</td>
</tr>
<tr>
<td>07</td>
<td>Paleosol</td>
<td>17.35 63.21</td>
<td>1.14 6.41 5.56</td>
</tr>
<tr>
<td>08</td>
<td>Lishi Loess</td>
<td>23.38 42.87</td>
<td>12.52 9.22 1.33</td>
</tr>
<tr>
<td>09</td>
<td>Paleosol</td>
<td>19.23 61.31</td>
<td>3.18 9.22 1.99</td>
</tr>
<tr>
<td>10</td>
<td>Lishi Loess</td>
<td>16.38 51.24</td>
<td>7.29 7.06 3.83</td>
</tr>
<tr>
<td>11</td>
<td>Lishi Loess</td>
<td>20.73 45.03</td>
<td>7.51 7.78 3.29</td>
</tr>
<tr>
<td>12</td>
<td>Paleosol</td>
<td>21.32 50.87</td>
<td>11.68 6.97 2.54</td>
</tr>
<tr>
<td>13</td>
<td>Paleosol</td>
<td>21.36 53.26</td>
<td>10.27 8.03 2.30</td>
</tr>
<tr>
<td>14</td>
<td>Lishi Loess</td>
<td>17.74 55.17</td>
<td>13.00 8.10 1.85</td>
</tr>
<tr>
<td>15</td>
<td>Wucheng Loess</td>
<td>19.77 48.95</td>
<td>8.14 6.60 4.06</td>
</tr>
<tr>
<td>16</td>
<td>Wucheng Loess</td>
<td>20.79 49.23</td>
<td>12.37 7.62 3.36</td>
</tr>
<tr>
<td>17</td>
<td>Wucheng Loess</td>
<td>20.28 50.02</td>
<td>11.14 5.34 4.76</td>
</tr>
<tr>
<td>18</td>
<td>Wucheng Loess</td>
<td>14.68 52.72</td>
<td>7.29 4.33 6.81</td>
</tr>
<tr>
<td>19</td>
<td>Wucheng Loess</td>
<td>23.12 40.70</td>
<td>13.99 2.71 6.89</td>
</tr>
<tr>
<td>20</td>
<td>Red clay</td>
<td>21.51 51.50</td>
<td>11.75 5.39 3.76</td>
</tr>
<tr>
<td></td>
<td>Malan Loess</td>
<td>15.80 47.08</td>
<td>4.04 5.38 4.86</td>
</tr>
<tr>
<td></td>
<td>Lishi Loess</td>
<td>18.28 48.69</td>
<td>9.40 6.99 3.39</td>
</tr>
<tr>
<td></td>
<td>Wucheng Loess</td>
<td>19.73 48.32</td>
<td>10.59 5.32 5.18</td>
</tr>
<tr>
<td></td>
<td>Paleosol</td>
<td>19.21 56.61</td>
<td>7.10 6.43 4.19</td>
</tr>
<tr>
<td></td>
<td>Red clay</td>
<td>21.51 51.50</td>
<td>11.75 5.39 3.76</td>
</tr>
</tbody>
</table>

K—Kaoilinite, I—Illite, M—Montmorillonite, Ch—Chlorite, V—Vermiculite, C—Calcite, G—Goethite, Others include Apatite, Anatase, Pyrite and MnO.

Table 5.4 Terminal Velocity-Particle Size Relationship for Soil-Water Suspension (Head, 1980)

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Approx. particle Diameter (μm)</th>
<th>Terminal Velocity (mm/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse Silt</td>
<td>35</td>
<td>1</td>
</tr>
<tr>
<td>Medium Silt</td>
<td>12</td>
<td>0.1</td>
</tr>
<tr>
<td>Fine Silt</td>
<td>3.5</td>
<td>0.01</td>
</tr>
<tr>
<td>Clay</td>
<td>1.2</td>
<td>0.001</td>
</tr>
</tbody>
</table>

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Table 5.5 Physical Properties of Ballotini Glass Balls (BGB) as Presented by the Manufacturer

<table>
<thead>
<tr>
<th>Property</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific gravity (approx. at 20°C)</td>
<td>2.95</td>
</tr>
<tr>
<td>Linear coefficient of expansion (between 0°C to 300°C)</td>
<td>9x10⁶</td>
</tr>
<tr>
<td>Refractive index</td>
<td>1.6</td>
</tr>
<tr>
<td>Specific heat (Cal./gm deg. C⁰) (between 20 and 100°C)</td>
<td>0.156</td>
</tr>
<tr>
<td>Hardness (Moh's scale)</td>
<td>5-6</td>
</tr>
<tr>
<td>Maximum working temperature (C⁰)</td>
<td>350</td>
</tr>
</tbody>
</table>

Table 5.6 Chemical Analysis of BGB as Presented by the Manufacturer

<table>
<thead>
<tr>
<th>Compound</th>
<th>Percentage Content</th>
</tr>
</thead>
<tbody>
<tr>
<td>Si O₂ (Quartz)</td>
<td>57%</td>
</tr>
<tr>
<td>Ph O</td>
<td>28%</td>
</tr>
<tr>
<td>K₂O</td>
<td>9%</td>
</tr>
<tr>
<td>Na₂O</td>
<td>4%</td>
</tr>
<tr>
<td>S₂O₃</td>
<td>2%</td>
</tr>
</tbody>
</table>
### Table 5.7 Physical and Chemical Properties of China Clay as Presented by the Manufacturer

<table>
<thead>
<tr>
<th>Property</th>
<th>Description as Presented by the Manufacturer</th>
<th>Measured Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Appearance</td>
<td>Creamy-white powder</td>
<td>----</td>
</tr>
<tr>
<td>Odour</td>
<td>None</td>
<td>----</td>
</tr>
<tr>
<td>Specific Gravity</td>
<td>2.6</td>
<td>2.62</td>
</tr>
<tr>
<td>Liquid Limit (LL)</td>
<td>------</td>
<td>58%</td>
</tr>
<tr>
<td>Plastic Limit (PL)</td>
<td>------</td>
<td>30%</td>
</tr>
<tr>
<td>Plasticity Index (PI)</td>
<td>------</td>
<td>28%</td>
</tr>
<tr>
<td>Surface Area</td>
<td>10-14 m$^2$/gm</td>
<td>----</td>
</tr>
<tr>
<td>pH</td>
<td>5.0</td>
<td>4.8</td>
</tr>
<tr>
<td>Melting point</td>
<td>&gt; 1200° C</td>
<td>----</td>
</tr>
<tr>
<td>Boiling point</td>
<td>Not applicable</td>
<td>----</td>
</tr>
<tr>
<td>Solubility</td>
<td>Generally insoluble in common solvents</td>
<td>----</td>
</tr>
<tr>
<td>Water solubility</td>
<td>Generally &lt; 100mg per Kg of water</td>
<td>----</td>
</tr>
</tbody>
</table>

### Table 5.8 Mineralogical Composition for China Clay as Given by the Manufacturer

<table>
<thead>
<tr>
<th>Type of Mineral</th>
<th>Percentage Content</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kaolinite</td>
<td>67%</td>
</tr>
<tr>
<td>Potash Mica</td>
<td>24%</td>
</tr>
<tr>
<td>Soda Mica</td>
<td>1%</td>
</tr>
<tr>
<td>Quartz</td>
<td>2%</td>
</tr>
<tr>
<td>Carbonaceous Material</td>
<td>&lt;4%</td>
</tr>
</tbody>
</table>
Table 5.9 Chemical Analysis of China Clay and Wyoming Bentonite as Presented by the Manufacturer

<table>
<thead>
<tr>
<th>Chemical Element</th>
<th>China Clay</th>
<th>Wyoming Bentonite</th>
</tr>
</thead>
<tbody>
<tr>
<td>Si O₂</td>
<td>48.0%</td>
<td>63.59%</td>
</tr>
<tr>
<td>Al₂ O₃</td>
<td>36.1%</td>
<td>21.43%</td>
</tr>
<tr>
<td>Fe₂ O₃</td>
<td>1.0%</td>
<td>3.78%</td>
</tr>
<tr>
<td>Ca O</td>
<td>0.1%</td>
<td>0.6%</td>
</tr>
<tr>
<td>Mg O</td>
<td>0.2%</td>
<td>2.03%</td>
</tr>
<tr>
<td>K₂ O</td>
<td>2.8%</td>
<td>0.31%</td>
</tr>
<tr>
<td>Na O</td>
<td>0.1%</td>
<td>2.70%</td>
</tr>
<tr>
<td>Bound water</td>
<td></td>
<td>5.50%</td>
</tr>
<tr>
<td>Loss on ignition</td>
<td>11.6%</td>
<td></td>
</tr>
<tr>
<td>at 950°C</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 5.10 Properties of Wyoming Bentonite as Given by the Manufacturer

<table>
<thead>
<tr>
<th>Property</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Appearance</td>
<td>Light Grey Coloured Powder</td>
</tr>
<tr>
<td>% passing 75µm</td>
<td>85% by weight passing</td>
</tr>
<tr>
<td>Moisture content (% by weight dried at 100-110°C)</td>
<td>10-14 %</td>
</tr>
<tr>
<td>Liquid Limit (LL)</td>
<td>680%</td>
</tr>
<tr>
<td>Plasticity Index (PI)</td>
<td>500%</td>
</tr>
<tr>
<td>Specific gravity</td>
<td>2.68</td>
</tr>
<tr>
<td>Specific area</td>
<td>800m²/gm</td>
</tr>
<tr>
<td>pH</td>
<td>9 (measured)</td>
</tr>
</tbody>
</table>
Table 5.11 Collapse Settlement Criteria (after Holtz and Hilf, 1961).

<table>
<thead>
<tr>
<th>Dry Density (Mg/m³)</th>
<th>Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 1.28</td>
<td>Loess is considered loose and highly susceptible to settlement</td>
</tr>
<tr>
<td>1.28-1.44</td>
<td>Loess is medium dense and is moderately susceptible to settlement, particularly for critical or heavily-loaded structures</td>
</tr>
<tr>
<td>&gt; 1.44</td>
<td>Loess is quite dense and may be capable of supporting ordinary structures without serious settlement.</td>
</tr>
<tr>
<td>1.36</td>
<td>A more general criterion has been used for earth dams: this density is used as the division between low and high density loess or the division where special foundation treatment is required for lower densities.</td>
</tr>
</tbody>
</table>
Fig. 5.1 Locations of Soil Samples from Libya
(1) Khoms         (2) Garaboli
(3) Gharyan       (4) Tahala (Ghat)
Fig. 5.2 Grain-Size Distribution of Libyan Loess
Fig. 5.3 Location of Soil Samples from China
Fig. 5.4 Particle Size Distribution Curves for Different Soil Materials
Fig. 5.5 Grain Size Distribution for the Sand Before Crushing
Fig. 5.6 End-Runner Mill, Apparatus Used for Grinding Sand
Fig. 5.7 The Relationship Between The Particle Size and Time Required to Fall Through Water a Distance of 100mm
Fig. 5.8 Plasticity Properties of Libyan Loess as Compared with Loess Types Defined by Gibbs and Holland (1960)
Fig. 5.9 Particle Size Distribution Envelopes of Missouri Basin (after Holtz and Gibbs, 1951) with Curves of Libyan Loess.
Fig. 5.10 Histogram of Particle Size-Distribution for Malan Loess
Fig. 5.11 Histogram of Particles Size Distribution for Pure Silt From Malan Loess

Fig. 5.12 Histogram of Particle Size Distribution for Pure Silt from Crushed Sand
Fig. 5.13 Histogram of Particle Size Distribution for Ballotini Glass Balls (BGB)

Fig. 5.14 Histogram of Particle Size Distribution for Pure Silt from Crushed Sand Having Grading Similar to that for Ballotini Glass Balls (BGB)
Fig. 5.15  Schematic Diagram for Scanning Electron Microscope (SEM) and Energy Despersive X-Ray (EDX) Spectrometer Systems
Plate 5.1 SEM Picture of Silt Particles from Gharyan Loess in Libya

Plate 5.2 SEM Picture of Silt Particles from Khoms Loess in Libya
Plate 5.3 SEM Picture of Silt Particles from Garaboll Loess in Libya

Plate 5.4 SEM Picture of Silt Particles from Malan Loess in China
Plate 5.5 SEM Picture of Silt Particles Derived from Crushed Sand

Plate 5.6 SEM Picture of Ballotini Glass Balls (BGB)
Plate 5.7 SEM Picture of Undisturbed Gharyan Loess in Libya
Plate 5.8 SEM Picture of Mes-K Mix (K/Mes=30/70) Prepared by Dry Mixing Method

Plate 5.9 SEM Picture of Mes-K Mix (K/Mes=30/70) Prepared by Wet Mixing Method
Plate 5.10 SEM Picture for BGB-B Mix (B/BGB=10/90) of Open Packing Structure (Mag.=200)

Plate 5.11 SEM Picture for BGB-B Mix (B/BGB=10/90) of Open Packing Structure (Mag.=500)
Plate 5.12 SEM Picture for BGB-B Mix (B/BGB=10/90) of Open Packing Structure (Mag.=1000)

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CHAPTER 6
EXPERIMENTAL MODELLING
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CHAPTER 6
EXPERIMENTAL MODELLING AND
METHODS OF TESTING

6.1 INTRODUCTION

An experimental programme has been designed to study the hydro-collapse of metastable structures that are prevalent in collapsible soil. Natural undisturbed (real) and artificial reconstituted (model) loess specimens of different soil materials were prepared and have been tested under different conditions. Soil specimens of an open packing were created to simulate the unstable and metastable structures of loess deposits by using the Free Air Fall technique. The compressibility and hydrocollapse behaviour of real and model loess specimens have been measured by utilising the oedometer compression test. Two methods of testing were conducted, the single oedometer test and the double oedometer test. The procedures of specimen preparation and testing are explained hereafter.

6.2 EXPERIMENTAL PROGRAMME

Laboratory tests on loess specimens were conducted in two stages. Tests in stage 1 were performed on natural undisturbed (real) loess samples collected from Libya and China in order to compare the hydro-collapse results of real loess with those for model loess to validate the dry particles-air fall technique which has been employed for reformation of loess specimens of metastable structure. The fundamental properties and the hydro-collapse behaviour of
Libyan loess have also been determined, together with the effect of the stress level on the collapse quantity. Tests in stage 2 were conducted on reconstituted (model) loess specimens to study the influence of the solid particle characteristics and the type and amount of bonding materials on the hydro-collapse behaviour of loess deposits. As shown previously several different loess mixtures have been used.

6.3 PREPARATION OF UNDISTURBED SPECIMENS

High quality disc-shaped specimens for oedometer compression tests were prepared from undisturbed block samples of loess. Two simple hand operated devices were designed and manufactured in the laboratory for the preparation of 50mm or 75mm diameter specimens of very soft, slightly cemented loess soils. One device was used for preliminary cutting of the specimen. It had a rotating sharp steel cutter of slightly greater diameter than the specimen, as illustrated in Figure 6.1. The second device was used for final finishing in order to prepare accurate specimens. This was composed of two rotating cylindrical discs of equal diameter mounted centrally. The space between the two platens was greater than the thickness of the specimen and was controlled by the movable platen. The diameter of the two platens nearly equals the specimen diameter, as shown in Figure 6.2. Before mounting the specimen in the finishing device (the lathe), it was first cut and subjected to preliminary preparation, using the core cutter device. The specimen was mounted between the platen of the lathe device and the movable platen, which was brought firmly into contact with the end surface of the specimen and locked into position so that the specimen was securely held. Surplus material was removed progressively from the sample using a sharp, straight knife. The specimen was rotated slightly between each cut and trimmed accurately to the final diameter.
by using the platens of the lathe as a guide for the knife when making cuts, until a smooth circular surface was obtained. Any stones or hard nodules (normally calcium carbonate concretions) were removed carefully, and resulting cavities were filled with trimmed loess particles (i.e. the same material as the specimen). The prepared specimen was then carefully placed in the oedometer ring and the ends were trimmed flat and parallel to give the correct thickness.

6.4 UNSTABLE AND METASTABLE STRUCTURES OF LOESS DEPOSITS

6.4.1 Introduction

Simulation of the formation of the collapsible structure in loess deposits has been attempted by a few Russian investigators (Trofimov, 1990a). Results of experimental investigations indicated that it was possible to re-fabricate an open structure of a soil sediment having a high porosity. In order to represent the initial unstable and metastable structure, and hence to model the collapsible formation of loess deposits experimentally, a number of methods of rebuilding artificial soil samples of an open structure have been tried and the more workable and successful techniques have been adopted. These techniques are described below.

6.4.2 Zero-Time Unstable Structure

6.4.2.1 Introduction

The initial unstable structure of very high voids ratio ($e \geq 1.35$) is the first stage in the formation of metastable structure of loess deposits. It occurs when the
solid particles of silt-size are first deposited in the air fall sedimentation mode. This can be simulated in the laboratory by allowing the dry silt particles to fall freely and directly into a mould (e.g. an oedometer ring). Soil specimens of unstable structure were produced by using the air-fall technique as follows.

6.4.2.2 Dry Particles-Air Fall (DP-AF) Method
Reconstituted loess specimens with a highly porous unstable structure have been prepared from the various silty materials described previously. The materials were sieved through the 63\textmu m sieve. The soil particles and particle groups of silt size were then allowed to fall freely under gravity directly into the oedometer ring by using two methods: funnel method and sieving method. In the funnel method the dry silty material was slowly poured through the orifice of the funnel directly into the ring, as shown in Figure 6.3. In the sieving method the loess material was slowly sieved through the 63\textmu m sieve from a certain constant height of 200-300mm directly into the oedometer ring, as shown in Figure 6.4. The second method was considered to simulate best the natural air-fall sedimentation. The soil structure produced by the funnel method had a random orientation, whereas the sieving method gave the particles the chance to adopt an orientation according to their properties and ambient conditions.

The main procedures used to determine the dry density and void ratio of this unstable structure are as follows:
1. Assembling the oedometer ring, porous stone disc and filter paper together, as shown in Figure 6.4, and determining the mass of the assemblage (m1).
2. Putting the assemblage in a wide tray and placing the dry silt particles directly into the oedometer ring using the air-falling technique, as shown in Figure 6.4.
3. Levelling off carefully the top surface of the specimen, cleaning the assemblage very well using a wire brush and weighing the assemblage and the soil specimen together (m2). Any sudden shock to the specimen was avoided since the loose material was very sensitive to vibrations.

4. The dry mass of the specimen (Md) was determined by subtracting m1 from m2.

5. The dry density (ρd) and the void ratio (e) of the specimen was calculated as follows:

\[
\rho_d = \frac{M_d}{V_t} \text{ (gm/cm}^3\text{)} \quad \text{(eqn 6.1)}
\]

\[
e = (\rho_s/\rho_d)-1 \text{ (dimensionless)} \quad \text{(eqn 6.2)}
\]

where

\[V_t = \text{total volume of the specimen (ring volume)} \text{ (cm}^3\text{)}\]
\[\rho_s = \text{dry density of solid particles (gm/cm}^3\text{)}\]

6.4.3 Metastable Structure of Loess Deposits

6.4.3.1 Introduction

The initial unstable structure was very open and sensitive to any kind of vibration. It could not support even a small load, and indeed tended to collapse as it was forming. The structural elements, essentially the solid particles and pore spaces, were re-oriented and re-arranged with respect to each other as this occurred. In so doing, an initial stress-resisting structure started to form. The initial metastable structure was developed as a result of this primary deformation process. The second stage in the formation of a metastable structure thus started when the initial unstable structure partially collapsed.
The metastable structure can be rebuilt in the laboratory by applying a very small statically induced-vertical stress (e.g. 5kPa) to the initial unstable structure or by increasing stability by using the wetting method described below.

6.4.3.2 Preparation of Reconstituted Specimens of Metastable Structure

(a) **Dry method**

Soil specimens of metastable structure were produced using a simple device designed specifically for this purpose. The device consists of a cylindrical steel mould and steel piston. The internal diameter of the mould is equal to the internal diameter of the oedometer ring. Two different mould-sizes have been manufactured, one for the small ring (50mm diameter) and the other for the large ring (75mm diameter). The mould was designed so that the oedometer ring could be inserted from the bottom into the mould. Full details of the assembly of this device and the oedometer ring are shown in Figure 6.5. The specimens were statically compacted directly into the oedometer ring. Utilising this method, sets of identical specimens could be made with different void ratios and dry densities. The preferred initial void ratio was obtained by predetermination of the mass of soil required in relation to the volume of the oedometer ring to give the dry density corresponding to the required void ratio. The mould, oedometer ring, filter paper, and porous stone were assembled together. The required amount of dry soil was gently placed into the ring using the airfall technique. The piston was then placed into position and pressed slowly until the flanges touched the mould. In order to prevent the soil particles coming in between the wall of the mould and the side of the piston, a filter paper of diameter equal to the diameter of the mould was placed on the
top surface of the specimen. The compacted specimen along with the oedometer ring and porous stone were weighed prior to mounting them into the oedometer. This allowed for the determination of the dry density of the specimens and hence the corresponding void ratio.

(b) Dry Particles - Air fall and subsequent wetting method

The dry particles - air fall method is the primary step for the wet method. The purpose of subsequently moistening the material is to allow the bonding substances to weld the silt particles together and form a loose and open granular framed metastable structure. Two methods of wetting can be utilised, by a fine spray of distilled water to the top of the specimen which is allowed to flow through the soil particles by gravity, and by applying the distilled water to the bottom of the specimen such that it permeates through the silty material due to capillary action. These two methods of wetting have been used by the Russian investigators and proved to be reliable. However, there are three main disadvantages of these methods of wetting:

1. It is difficult to control the initial properties (mainly the dry density and voids ratio) of the soil specimen because of the shrinkage due to the wetting and drying processes.
2. Cracking problems due to the shrinkage phenomenon.
3. These methods of wetting make the clay particles migrate towards one of the end surfaces of the specimens due to the effect of the water flow and produce soil specimens of non-uniform structure. This may affect the hydrocollapse characteristics of the soil structure.

These problems can partially be solved by applying the distilled water simultaneously to both surfaces of the specimen, by wrapping it carefully with wet cloth after placing a filter paper and porous stone at each side of the specimen, and keeping this assemblage in a plastic bag for 24 hours. Also
these problems can be treated by keeping the sample in a controlled humid environment.

6.5 TESTING METHODS

6.5.1 Introduction

The experimental methods used for the determination of the index properties, chemical tests, geometrical characteristics and mineralogical analysis were explained in Chapter 5. The methods employed for the investigation of the collapse behaviour of loess deposits are described in this section as follows.

6.5.2 Classical Consolidation Test

The oedometer machine has been used to determine the compressibility and consolidation parameters since 1934 when Terzaghi designed the first consolidation apparatus, which he called an "oedometer" from the Greek oedema, meaning swelling (Head, 1980). Head (1982) stated that: "in 1938 Skempton at Imperial College, London, developed an oedometer for a 25mm thick specimen based on the Casagrande principle". The oedometer consolidation test became a standard laboratory procedure after 1945, and it is now a well established technique (BS 1377: BSI 1990). This technique was modified and used to investigate the collapse behaviour of metastable soils Kane (1969); Jennings and Knight (1975); Handy, (1973); Mellors (1978); Lutteneger (1988). The classical consolidation test has been described in more detail by many authors such as Vickers (1983), Head (1982), Bowles (1992), and Holtz et al (1986).
6.5.3 Hydro-collapse Test

6.5.3.1 Introduction
The hydro-collapsibility properties and behaviour of undisturbed and remoulded samples with particular reference to their response to wetting under different stress levels have been investigated. Soil specimens of 50mm diameter and 19mm thickness were tested in one-dimensional compression tests using an oedometer apparatus. Two methods of proven reliability are currently used to determine the hydro-collapse parameters, the single-oedometer test and the double-oedometer test. Similar results have been obtained using the two techniques (Erol and El-Ruwaih, 1982; Justo et al, 1984; Lawton et al, 1989; and Basma and Tuncer, 1992).

6.5.3.2 The Single Oedometer Test
The single oedometer test consists of loading the specimen incrementally to a specific state of vertical stress and allowing the sample to come to equilibrium under the applied pressure. The sample is then flooded by water, and the deformation is measured. The deformation induced by the addition of water divided by the height of the specimen before flooding, expressed as a percentage, defines the magnitude of the hydro-collapse.

6.5.3.3 The Double Oedometer Test
The double oedometer test involves a pair of nominally identical specimens. One of these specimens is loaded in its natural state and the deformation is measured at each stress level. The second specimen is initially flooded and a similar loading procedure is carried out. The difference between the equilibrium deformations is used to determine the magnitude of hydro-collapse.
6.5.3.4 Apparatus

Two types of standard fixed ring oedometers were used: front lever loading and rear lever loading (Bishop type) devices manufactured by Wykeham Farrance Engineering Limited (see Figure 6.6). Vertical loads were applied to the specimen using dead weights which were added to a counterbalanced lever arm via a loading yoke, which could be adjusted to maintain the lever arm in a level position. In most cases, it was necessary to start the loading process with the lever arm inclined slightly upwards, such that the mean position of the arm was horizontally level. Load was subsequently transferred to the specimen via a pre-greased rounded end of the load shaft, thus ensuring that non-eccentric load was applied to the soil specimen.

For measuring the vertical compression, a dial gauge of 0.002mm per division was used, calibration of this instrument having previously been carried out by the technical staff during routine laboratory maintenance. Smooth polished stainless steel rings of 50mm and 75mm diameters and 19mm thickness were used for testing. They were slightly lubricated by silicon grease to reduce friction between the soil specimens and their inside walls. The rings themselves were held in place by an outer confining ring which was securely clamped into place.

6.5.3.5 Calibration of Apparatus

The elastic deformation characteristics of the apparatus used were determined using a similar method for that described in BS1377 (BSI 1990). A stainless steel disc, of similar dimensions to the soil specimens, was placed into the apparatus in turn between dry filter paper (the same as that used for normal testing). Loads applied corresponded to the stress range used for each soil specimen. The calibration was repeated three times and the average value of compression corresponding to apparatus deformation was determined for each
stress increment and an appropriate correction made to the measured settlements.

6.5.3.6 Test Procedure

Specimen loading was carried out with reference to British Standard BS1377 (BSI 1990). A loading sequence of 5, 10, 25, 50, 100, 200, 400, 800 and 1600 kN/m² was used, the general procedure was to apply increments of load, such that the pressure was doubled at each increment, i.e. with a load ratio of 2. For undisturbed specimens a relatively high initial stress could be used because below this value no significant compression was observed. For the dry state the loading increments were added at two hour intervals, whereas for the saturation state the intervals were four hours. In most cases the load on the specimens at the time of flooding was left for 24 hours before the addition of the next load. For the artificial specimens prepared from silt-clay mixture in which a Wyoming bentonite was employed as a bonding material, the time intervals were 24 hours for the wet state and 48-72 hours (depending on the clay mineral content) at the time of flooding. Generally it was evident that the hydro-collapse of silty soil occurred rapidly (less than 2 hrs) even for relatively high clay fraction (Booth, 1977; Huang, 1980; Lawton et al). Trial tests were conducted to determine the time period required for complete hydro-collapse, and the results indicated that the complete collapse occurred within a period less than one hour for all natural and artificial loess materials except those silt-clay mixtures prepared by mixing the pure silt from crushed sand with Wyoming bentonite.
Fig. 6.1 Rotating Cutter Device for Preliminary Cutting in the Preparation of Undisturbed Specimens

Fig. 6.2 Final Trimming for the Preparation of Accurate Undisturbed Specimens
Fig. 6.3 Funnel-Airfall Method for the Preparation of Reconstituted Specimens of Unstable Structure
Fig. 6.4 Pluviation-Airfall Method for the Preparation of Reconstituted Specimens of Unstable Structure
Fig. 6.5  Cylindrical Steel Mould for the Preparation of Reconstituted Specimens of Metastable Structure
Fig. 6.6 WF 24000 Bishop Type Oedometer (use 75 mm ring)
CHAPTER 7
PRESENTATION OF EXPERIMENTAL RESULTS

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7.4 COMPRESSIBILITY AND HYDROCOLLAPSE BEHAVIOUR OF MODEL (ARTIFICIAL) LOESS
  7.4.1 Introduction
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  7.4.4 Pure Silt from Crushed Sand and Bentonite ($M_{cs-B}$) Mixes
  7.4.5 Pure Silt - Clay Size Quartz ($M_{cs-Qc}$) Mixes
7.1 INTRODUCTION

This chapter aims to present the results obtained from single and double oedometer tests, the testing methods of which have been described in Chapter 6. The compressibility properties and hydrocollapse behaviour of undisturbed and reconstituted soil specimens have been determined for different types of loess, namely natural loess from China, natural loess from Libya and artificial loess (model loess). The test results of the index properties and the SEM micrographs of the materials used have been presented in Chapter 5.

7.2 COMPRESSIBILITY AND HYDROCOLLAPSE BEHAVIOUR OF NATURAL LOESS MATERIALS

7.2.1 Undisturbed Loess Samples

7.2.1.1 Libyan Loess

The hydrocollapse values for Khoms and Gharyan soils (Tripoli Loess) from single and double oedometer tests are summarised in Table 7.1. The general trend of the hydrocollapse behaviour for Tripoli Loess is shown in Figures 7.1 and 7.2. The calculated amount of hydrocollapse (hydrocollapse at 200 kPa) was in the range 12.5-13.5% for both Khoms and Gharyan soils. Based on the
values given by Jennings and Knight (1975), the Libyan Loess can be considered to pose severe collapse problems. Generally for a stress range higher than 25 kPa the measured magnitudes of hydrocollapse of the specimens vary over a range of 1 to 22% for both samples, depending on the level of the vertical pressure applied to the specimens at the time of wetting. Figures 7.1 and 7.2 show that the compression of specimens in the air-dry condition (pre-wetted state) is insignificant as compared to the hydrocollapse settlement upon wetting. According to the test results presented in Figures 7.1 and 7.2 it can be seen that there is a unique post-wetted relationship between void ratio and applied pressure for all specimens that were subjected to wetting at different stress levels. Thus upon wetting at any applied vertical stress the material always collapsed to a value of void ratio prescribed by the unique post-wetted void ratio - applied pressure curve. A further increase in applied pressure resulted in additional reductions in the void ratio following the post-wetted curve.

The curves shown in Figure 7.3a illustrate the effect of wetting pressure on the collapsibility of loess material and indicate that the magnitude of hydrocollapse increases as the applied pressure at wetting increases up to a certain value, at which the maximum collapse will be obtained. In the case of Figure 7.3a the maximum collapse occurred at about 1600 kPa and reached 20 to 22%. Figure 7.3a also clearly shows that the rate of change of volume due to hydrocollapse with applied pressure is large at low levels of applied pressure but reduces considerably at higher levels such that it becomes very small above 800 kPa. It should be noted that the curve, if progressed beyond 1600 kPa, would reach a maximum whereafter the degree of hydrocollapse would reduce since the dry and wet curves will converge. This is due to the large applied pressures overcoming the strength of the loess structure when in the dry state and causing large settlements without wetting. The measured values in the single-
Oedometer tests at different stress levels appear to be in agreement with the values calculated from the double-oedometer test results. The data produce an approximately linear relationship when plotted on a logarithmic scale (Figure 7.3b), although the tendency to an asymptotic void ratio is indicated by the flattening of the curve at high pressures.

7.2.1.2 Chinese Loess

Hydrocollapse test results for undisturbed specimens from Malan loess flooded at pressures of 50-800 kPa are summarised in Table 7.1 and presented in Figure 7.4. The magnitude of hydrocollapse varies over a range of 1.0 to 15.7%, depending on the level of the vertical pressure applied to the specimens at the time of wetting. At a stress level of 200 kPa the degree of hydrocollapse was found to be approximately 8%. It can be seen once again that the compression of the specimens is insignificant in the dry state as compared to the collapse settlements due to flooding. From Figure 7.4 it can be also noticed that all loess samples, regardless of flooding pressure, showed a unique post-wetted relationship between void ratio and applied pressure. This observation again indicates that the loess has a unique post-wetting curve, and always collapsed to the final void ratios prescribed by this curve, independent of the applied stress level at flooding. When the hydrocollapse compressions were complete, the e-log p curve became approximately linear, and a further increase in applied stress resulted in additional reduction in void ratio following the curve.

Figure 7.5 shows the variation in hydrocollapse magnitude due to the effect of different applied pressures at the time of wetting. As for the Tripoli loess, the hydrocollapse value increases as the applied stress increases. It is clear that the rate of volume change due to hydrocollapse with applied pressure is large at low levels of stress but reduces considerably at higher levels. A comparison between the hydrocollapse behaviour of undisturbed specimens from Malan
loess and from Tripoli loess is presented in Figure 7.6 which shows that although the general trend for both materials is similar, the hydrocollapse values at all stress levels for Chinese loess are lower than those for Libyan loess.

7.2.2 Reconstituted Loess Samples

7.2.2.1 Introduction
Rebuilding soil structure experimentally is one of the important approaches which can be used to study the engineering behaviour and specifically collapse behaviour of loess deposits. One of the main purposes of this research is to examine the possibility of producing a reconstituted sample that simulates the collapsible structure of loess deposits, and subsequently to determine a proper method of experimental modelling to represent metastable soil structures. To achieve this goal, reconstituted specimens of natural loess material from Libya and China have been prepared by the airfall method and tested in single and double oedometer tests. The hydrocollapse test results are presented in this section.

7.2.2.2 Libyan Loess
Oedometer test results for reconstituted specimens prepared from natural loess materials are summarised in Table 7.1 and presented in Figures 7.7 and 7.8. The materials were air dried, pulverized, and sieved through 89, 75, and 63 \( \mu \text{m} \) sieves. The soil fractions retained on the 75 and 63 \( \mu \text{m} \) sieves and the fraction that passed through the 63 \( \mu \text{m} \) sieve, were then mixed together to form a homogeneous loess sample. The initial properties (dry density and void ratio) of all remoulded specimens were approximately equal to the corresponding values for the undisturbed specimens prepared from the same materials. The
reconstituted specimens were prepared by using the dry particles-air fall method as described previously.

Figures 7.7 and 7.8 show the results of the single and double oedometer tests performed on initially air-dried remoulded loess specimens prepared from Khoms and Gharyan soils. It is clear that the general shape of the compression curves for the reconstituted specimens is similar to those for the undisturbed specimens of the same materials. The collapse behaviour of the reconstituted and undisturbed specimens is thus qualitatively similar. In the course of loading, both types of sample undergo structural collapse due to the effect of loading and subsequent wetting. The post-wetting compression curve for the unbonded samples is very similar to that of the bonded samples. All specimens that were subjected to wetting at different vertical stress levels showed a unique post-wetted relationship between void ratio and applied pressure. These observations indicate that the metastable structure of collapsible soil can be simulated experimentally in the laboratory.

The magnitudes of the hydrocollapse compression vary over a range of 4.0 to 17.5%, depending on the applied stress level at the time of wetting. Hydrocollapse magnitudes of about 13.5% for Khoms soil and 15.5% for Gharyan soil were determined for a vertical applied stress of 200 kPa, the level of stress used to define the hydrocollapsibility of the material. In the air-dry state both types of specimen indicate a low compressibility response and they showed insignificant compression over a range of applied stress lower than 100 kPa. However, they revealed significant compression in a stress range higher than this pressure when compared with the compression of undisturbed material. In the saturated state the unbonded materials exhibited a higher compressibility response than the bonded materials, particularly in the range of low applied pressure. This structural collapse behaviour of the naturally
bonded and unbonded materials could be predicted and can be attributed to the effect of interparticle bonding.

Figures 7.9 and 7.10 show the relationship between the hydrocollapse magnitude and the applied pressure for reconstituted loess samples from Khoms and Gharyan soils. It can be seen that the soil hydrocollapse increases as the wetting pressure increases, as expected, and the rate of change is large at low levels of applied pressure. At higher levels it reduces considerably, such that it becomes very small above 400 kPa for Khoms samples and exhibits no change above that pressure in the case of Gharyan samples.

7.2.2.3 Malan Loess

The compressibility properties and hydrocollapse behaviour for recostituted specimens having different initial voids ratios prepared from natural Malan loess from China are presented in Figure 7.11. From this Figure it can be seen that the dry curves for the specimens having higher voids ratios indicate that they are more compressible than those having lower voids ratio, as would be expected. The saturated curves for both types of specimens are approximately similar. After the hydrocollapse settlements are completed, the e - log p curves become nearly linear with a steeper slope. Figure 7.12 shows the relationship between the percentage hydrocollapse and the wetting pressure for the specimens having higher voids ratios. It is clear that the rate of change in hydrocollapse is very small, the curve being nearly flat and hence the amount of soil hydrocollapse is nearly constant, varying from approximately 14% at 100 kPa vertical stress to approximately 16% at 800 kPa. This behaviour is related to the high initial voids ratios of the specimens. Hydrocollapse values of 6.5, 8.2 and 10.6% have been obtained when the specimens having lower initial voids ratio were tested at 50, 100 and 200 kPa stress levels.
7.3 COMPRESSIBILITY AND HYDRO-COLLAPSE BEHAVIOUR OF PURE SILT

7.3.1 Introduction

Different natural loess materials have been tested and the results presented in Section 7.2. In order to understand the hydrocollapse behaviour of the silt-clay (M-C) mixture, and then to explain the hydrocollapse mechanism of loess deposits, it is very important to determine the compressibility characteristics and hydrocollapse behaviour of the clay-free soil system (pure silt). Three different types of silt particles were tested: spherical glass balls (Ballotini), pure silt from crushed sand and pure silt from Malan loess. Results of these tests are presented in the following sections.

7.3.2 Pure Silt from Malan Loess (China)

Fine materials of silt size have been produced in the laboratory having been extracted from natural Chinese Malan loess. The clay particles that exist in the natural loess were separated from the silt particles and washed out using a sedimentation method. Reconstituted soil specimens have been prepared and tested in one-dimensional oedometer compression tests. The single and double-oedometer test results are shown in Figure 7.13. The effect of stress level at time of wetting on the magnitude of hydrocollapse is presented in Figure 7.14. Based on the results presented in Figures 7.13 and 7.14, the main observations can be summarized as follows:

(1) The magnitude of hydrocollapse is nearly constant, varying from about 3% to 5%.
Although the initial void ratios of the specimens tested \((e_0 = 1.15)\) were very high, the hydrocollapse is relatively very small when compared with results of natural undisturbed loess material.

The general shape of the pre-wetting (dry state) curves is very similar to those curves of the post-wetting (saturation) state. In both conditions the materials exhibit downward curves with a relatively gentle slope in the stress range below 100 kPa and a steeper slope in the higher range. It seems that there is no effect of water on the compressibility properties of the pure silt.

7.3.3 Pure Silt from Crushed Sand

Hydrocollapse test results for reconstituted soil specimens prepared from pure silt are presented in Figure 7.15. Based on the test results, the following conclusions can be drawn:

1. The compressibility behaviour of pure silt from crushed sand is similar to that of pure silt from Malan loess, although the curves are generally flatter indicating a slightly less compressible material. However it should be noted that the initial void ratios were lower than those for the Malan loess.

2. No significant collapse was observed, although the initial void ratios of the specimens tested were relatively high \((e_0 \geq 0.9)\).

3. The general trend of the \(e\)-log \(p\) curves is the same before and after wetting. This behaviour also confirms that there is no effect of water on the compressibility properties of the pure silt.
Ballotini glass balls are ideal solid particles. They are spherical, uniform and very smooth. These specific properties give them the ability to move around very easily. It was observed that it is very difficult to form an open packing of metastable structure by using these particles. They move easily and make a soil structure of relatively close packing and low voids ratio when they are directly dropped into the oedometer ring. Soil specimens of about 0.56 voids ratio were prepared and subjected to a single oedometer test (Figure 7.16) in which the material was shown to have very low compressibility in both dry and wet conditions. It is also very clear that no significant hydrocollapse (<0.5%) was observed. The e-log p curve is approximately linear and similar before and after wetting. It seems that there is no effect of water on the collapse behaviour of this kind of material.
7.4 COMPRESSIBILITY AND HYDROCollapse BEHAVIOUR OF MODEL (ARTIFICIAL) LOESS

7.4.1 Introduction

In an effort to investigate the influence of compositional factors of soil structure such as the geometrical characteristics of the solid particle (shape, size and surface texture), the type and amount of clay minerals and the soil fabric on the compressibility and hydro-collapse behaviour of loess deposits, a laboratory study was conducted using mixtures of different materials, as explained in Chapter 5. Two types of clay minerals were used: low plasticity kaolinite (English china clay) with a plasticity index (PI) of about 30% and very high plasticity sodium montmorillonite (Bentonite) with a plasticity index of about 500%. To investigate the effect of the shape and surface texture of the solid particles, ballontini glass balls (ideal particles) of spherical shape and very smooth surface and pure silt derived from crushed sand of very angular shape and very rough surface were tested. In order to study the effect of the soil fabric, particularly the distribution of the clay particles in between the silt particles two different methods of mixing were used, poor-mixed material (dry mixing method) and well-mixed materials (wet mixing) method.

All the specimens were prepared at the same initial properties (water content, dry density, voids ratio). The initial voids ratio adopted was about 0.90 for all specimens, being close to the average value of voids ratio for most loess deposits in the world, which ranges between 0.80 to 1.00. The other important factor is that an open metastable structure of initial voids ratio equal to 0.90 was found to be achievable for all model specimens of various clay mineral content.
7.4.2 Ballotini Glass Balls-Kaolinite (BGB-K) Mixes

The compressibility features of dry and wet totally ideal soils produced from spherical glass balls and English china clay are shown in Figures 7.17 to 7.25. The curves shown in the figures represent the compression curves for both the pre-wetted (dry) and post-wetted (wet) states. Generally it seems that the BGB-K mixes are highly compressible material in both dry and wet states. The main two aspects of the dry and wet compression curves are the pressure beyond which compression increases considerably, known as the critical pressure \((P_{cr})\) by Phien-Wej et al (1992) or the collapse pressure \((P_{cs})\) by Reginatto and Ferrero (1973) and the slope (the rate of volume change) of the curves. Redolfi (1980) named this critical pressure the yield pressure. For both states the yield pressure increased as the clay mineral content increased, although for the dry state the pressure was higher than that for the wet state. In the dry state, the critical pressure was about 25 kPa for K/BGB ratios of 10/90, 15/85 and 20/80. For corresponding ratios of 25/75, 30/70, 35/65, and 40/60 the critical pressures were approximately 50 kPa, 100 kPa, 100 kPa and 200 kPa respectively. These critical pressures defined the stresses at which the mechanisms that maintain the metastability of the open packing structure of the soil became destroyed and the system starts to collapse. This structural collapse occurred due to the micro-shear failure at the points of the interparticle contacts. In the case of the wet state, the initial flooding weakened and softened the clay minerals, causing a drastic reduction of the critical pressure of all mixes to about 10 kPa. The slope (the rate of volume change) of the dry and wet curves beyond the critical pressure increased as the clay/silt (C/M) ratio increased.

Figures 7.26 and 7.27 show variation of voids ratio \((e)\) with the logarithm of the applied pressure \((\log p)\) for various values of K/BGB ratio. When the clay
mineral content is increased the e-log p curve for both dry and wet states is gradually moved to the right. The compression index (the slope of the straight line part of compression curve) for both dry and wet curves also increased as the clay mineral content increased. It can also be noted that as the applied vertical stress increased, the dry specimen curves tended to decline rapidly after the yielding pressure towards the compression curves of the saturated specimens. This led to the assumption that if more stress had been applied, they would have come closer to each other. This behaviour of both dry and wet materials is mainly related to the shape and surface texture of the primary Ballotini solid glass balls and the physical properties of Kaolinite. The sphericity and smoothness give the Ballotini sufficient mobility to take a more stable position of low void ratio and high density.

The magnitudes of the hydro-collapse settlement vary over a range of 4% to 20% (see Table 7.2), depending on the clay mineral content and the level of the vertical stress applied to the specimens at the time of flooding. The hydro-collapse settlements for specimens prepared by the wet mixing method and measured in the single-oedometer test appear to be in good agreement with the hydro-collapse deformation obtained from the double oedometer test results. This observation indicates that the BGB-K mix has a unique post-wetted e-log p relationship which is independent of the testing method. Thus the hydro-collapse-wetting pressure relationship within a specified stress range can be determined accurately by means of double-oedometer tests. These results show that the compressibility aspects of loess deposits are mainly controlled by the shape and surface texture of the silt particles, the volume of pore spaces between the particles and the clay mineral content. This means that the deformational behaviour of loess deposits depends basically on the soil microstructure (fabric and bonding).
The hydrocollapse values for BGB-K mixes from double oedometer tests are summarised in Table 7.2. The effect of the clay mineral content on the hydrocollapse properties of model loess is illustrated in Figure 7.28. The general trend in this figure suggests that for a particular applied wetting pressure, the hydro-collapse value increases as the clay content increased up to a maximum value, following which the hydro-collapse starts to decline with a further increase in clay mineral content. The optimum clay mineral content (the clay mineral content of maximum collapse) increases as the applied stress increases. These peak values of hydro-collapse were generally found to occur just before the point of critical pressure (yielding stress) after which the influence of applied stress alone had already induced significant structural collapse of dry soil. This observation can be explained by the fact that the clay mineral works as connecting bridges and filling material for the ideal silt particles (glass balls). In the dry condition, the clay particles at the contact points of the silt particles provide some shear strength that can resist deformation up to a certain level of stress, beyond which the soil structure starts to be destroyed and collapse occurs. The higher the clay fraction, the higher this resistance is to densification. In the wet condition, when the soil is allowed free access to water, the clay mineral at the contact points becomes very soft and perhaps even starts to work as a lubricating material. Together with the smoothness of the surface of the glass balls, these two properties make the system unstable and large hydro-collapse occurs.

Figure 7.28 shows that the greatest amounts of hydro-collapse for stress levels between 5 and 50kPa occurs at a relatively low clay mineral content of approximately 17% (i.e. 25/75 K/BGB ratio). These values of hydro-collapse are 15.4%, 15.7%, 18.2% and 19.8% respectively. For 100 kPa, 200 kPa and 400 kPa the greatest amounts of hydro-collapse were observed at higher clay mineral contents corresponding to K/BGB ratios of 30/70, 35/65, and 40/60,
respectively. The measured hydro-collapse values are 19.7%, 19.7%, and 20.1%, respectively. It should be noted that at stress levels of 50kPa and above, the maximum degree of hydrocollapse appears to reach a remarkably consistent value of approximately 20%.

The basic (fundamental) factors that control the movement of the primary solid particles are size, shape and surface texture of the particles, pore spaces in between the soil particles, the amount and type of clay minerals and the stress level. Based on the test results it seems that there is a certain combination of these fundamental factors which combine together to form a metastable structure of maximum hydro-collapse when loaded and wetted. For example, for the BGB-K mix the combination which gives the maximum hydro-collapse at 50kPa was found to be a K/BGB ratio of 25/75 (16.8% clay mineral content).

Figure 7.29 presents the hydrocollapse-applied pressure relationship for BGB-K mixes having different K/BGB ratios. From the figure it can be seen that the applied stress has a significant effect on the hydro-collapse behaviour, as discussed above. Generally for all the different mixes, the amount of hydro-collapse increased as the applied vertical pressure increased up to the maximum value and then decreased rapidly by further increasing the applied pressure. Two clear inflexions can be seen, the first one at the critical (yielding) pressure (10 kPa) of the saturated compression curves and the second one at the critical pressure of the dry compression curves, which depends on the K/BGB ratio. In the first part of the curves the slope rises gently up to the first point of inflexion, and then rises sharply up to the maximum value of hydro-collapse. All the maximum hydro-collapse values have been obtained at the critical pressure of the dry material for all different mixes. For example, the maximum hydro-collapse for the K/BGB ratio of
10/90 occurred at the yielding stress (critical pressure) of the dry compression curve of this mix which equals 25 kPa. The maximum values of hydro-collapse were found to be related to the K/BGB ratios. For mixes of lower clay mineral content the maximum hydro-collapse magnitudes were observed at lower vertical stress levels.

A comparison between the hydro-collapse values for specimens prepared by different mixing methods is illustrated in Figure 7.30. The curves in this figure indicate that the wet mixing method produces specimens of higher potential to hydro-collapse than those produced by dry mixing method. From this figure it can also be seen that in the range of 15% to 25% clay mineral content the difference in the hydro-collapse values is maximum and the curves start to converge at lower or higher than this range of clay content. These results show that the method of mixing has a significant effect on the hydro-collapse behaviour of loess deposits.

7.4.3 Pure Silt From Crushed Sand-Kaolinite (Mcs-K) Mixes

The single and double oedometer test results for pure silt-kaolinite (Mcs-K) mixtures of different clay/silt (C/M) ratios are presented in Figures 7.31 to 7.42. These tests were carried out in order to observe the hydro-collapse behaviour of initially identical soil specimens which were then flooded at different stress levels. In each figure, one specimen was soaked at a very low applied stress of 5 kPa, two specimens were flooded at a vertical stress of 200 kPa and the other one was saturated at a very high applied pressure of 1600 kPa. The e-log p curves corresponding to 5 and 1600 kPa wetting pressures resemble the wet and dry compression curves of the double oedometer testing condition respectively. The curves corresponding to 200 kPa wetting pressure
represent the single oedometer testing condition for both the dry and wet mixing methods.

In the light of the results, it can be seen that at a low stress level the dry specimens are much less compressible than the wet specimens. Generally all mixes showed similar characteristics during testing and have trends similar to those of the BGB-K mixes. For both dry and wet conditions the specimens are less compressible in the stress range lower than the yielding pressure (critical pressure) and they are at stresses greater than the yielding pressure. The critical values of vertical pressure were found to be related to the clay/silt (C/M) ratio. The critical pressure (yielding stress) increases as the clay content increases, and for the dry specimens the critical stress is higher than that of wet specimens. For dry specimens of clay/silt ratio range between 15/85 and 30/70 the critical pressure is 200 kPa. For ratios of 35/65 and 40/60 the yielding stress is 400 kPa, and for ratios of 45/55, 50/50 the yielding pressure is 800 kPa. For the wet specimens the critical stress is approximately 10 kPa, which is equal to that of the wet specimen of the BGB-K mixes.

It can also be seen from Figures 7.31 to 7.42 that a considerable amount of hydrocollapse could occur at a low stress level. This amount of hydrocollapse was found to increase as the clay mineral content increased up to a certain clay/silt (C/M) ratio and then decreased. Although the general trend of the hydrocollapse behaviour for both BGB-K and M_{cs}-K mixes is similar, in terms of the magnitude of hydrocollapse they are different. In general the hydrocollapse values for BGB-K mixes are much higher than those for M_{cs}-K mixes, particularly at low level of stress. This difference seems to be mainly due to the differences in the geometrical properties of the primary silt particles for both mixes. The angularity (very sharp edges) and the rough surfaces of the silt particles from crushed sand make the M_{cs}-K mixes more stable and
reduce the hydrocollapse. Another important observation from Figures 7.31 to 7.42 is that they all return to a unique saturated compression curve on flooding. This behaviour appears to be independent of the value of applied stress at which soaking was initiated.

It can also be noted that the maximum amount of hydrocollapse occurs at a wetting pressure just below or equal to the yielding stress. The reduction in hydrocollapse at higher stress levels is caused by the considerable densification of the dry material.

A summary of hydrocollapse values for Mcs-K mixes is presented in Table 7.3. The hydrocollapse-clay mineral content relationship for the Mcs-C mixtures is illustrated by Figure 7.43. The behaviour of the Mcs-C mixture is similar to that of BGB-K mixture. The hydrocollapse increases as the clay mineral content increases up to a certain amount (optimum clay content) at which the hydrocollapse reaches the peak value, and thereafter decreases as the proportion of clay in the mixture increases further. The optimum clay content was found to increase as the applied vertical stress increased. From Figure 7.43 it can be noticed that the optimum clay content for stress levels of 5 to 100 kPa is 16.8%, for stress levels of 100 to 400 kPa it was found to be equal to 26.8% (C/M = 40/6), and for 800 kPa it was observed as 30.2% (C/M = 45/55).

The influence of the applied vertical stress at the time of wetting on the hydrocollapse properties of Mcs-K mixtures was determined from the double oedometer test results as shown in Figure 7.44. The results show a similar trend to that of BGB-K mixtures. It can be seen that the hydrocollapse value increases with the pressure at wetting until it reaches a peak, thereafter it again decreases. For the same clay/silt (C/M) ratio, an increase in applied
pressure below the critical stress of the dry material led to a larger hydrocollapse. However, beyond the critical pressure (yielding stress) at which the effect of applied stress alone had already induced significant soil collapse, the hydrocollapse starts to decrease with a further increase in stress. The wetting pressure of maximum hydrocollapse was found to be related to the clay mineral content as illustrated in Figure 7.44. It increased as the clay content increased. For 15/85, 20/80, 25/75 and 30/70 C/M ratios the maximum hydrocollapse values were obtained at a wetting pressure of 200kPa, for 35/65, 40/60 and 10/90 C/M ratios they occurred at 400 kPa and for 45/55 and 50/50 C/M ratios they occurred at a wetting pressure of 800 kPa. All these wetting pressures at which maximum hydrocollapse values for all clay/silt (C/M) ratio were obtained were found to be equal to the critical pressures (yielding stresses) for the dry materials of different Mcs-K mixtures.

The effect of mixing method on the degree of hydro-collapse of Mcs-K mixture is shown in Figure 7.45. This figure shows that the general trend of the curves for both dry and wet mixing methods is similar to that for BGB-K mix. The main feature of this figure is that the hydro-collapse values for the specimens prepared by the wet mixing method are much higher than those for the specimens prepared by the dry mixing method. This result is attributed to the degree of mixing and the distribution of the clay minerals in between the silt particles, the wet mixing method being capable of better mixing. It can also be seen that the curves for the two mixing methods converge as the percentage of clay gets lower or higher and diverge at 15%-30% range of clay mineral content.
7.4.4 Pure Silt from Crushed Sand and Bentonite (Mcs-B) Mixes

Hydro-collapse test results for identical specimens of the same initial properties (dry density, void ratio, and water content) prepared by mixing pure silt from crushed sand with Bentonite to produce silt-clay (Mcs-B) mixtures of different clay/silt (C/M) ratios are shown in Figures 7.46 to 7.51. Six specimen sets (four specimens for each set) of different clay/silt (C/M) ratio (5/95, 7.5/92.5, 10/90, 12.5/87.5, 15/85 and 20/80) were prepared and tested using the airfall technique and hydrocollapse test methods respectively. Specimens prepared by dry mixing method were also tested and flooded at 200 kPa (one specimen for each C/M ratio). To avoid swelling, the specimens were not flooded at the lower range of pressures.

The hydro-collapse model of the Mcs-B mix was found to be generally similar to those for Mcs-K and BGB-K mixes, but the deformational behaviour of Mcs-B mix was different in many features. From Figure 7.54 it can be seen that the whole picture of the hydro-collapse behaviour (to determine the optimum clay content) can be obtained within a small range of clay content variation (5% - 15%) for Mcs-B mixes, in comparison to the 10% - 40% clay mineral content range which was observed during the testing of Mcs-K and BGB-K mixes. It may be seen that the potential of hydro-collapse varies significantly with very small increases (2.5%) in clay content for bentonite, but that the variation in fundamental behaviour (although not the magnitude of hydrocollapse, which is generally greater) is less dramatic in the case of kaolinite. This difference in the collapse behaviour between these mixtures seems to be related mainly to the difference in the plasticity properties of the low plasticity (PI = 20-30%) kaolinite and the high plasticity (PI = 400-500%) bentonite. The optimum clay content for the Mcs-B mix at stress levels between 50 and 400 kPa seems to be in the range 6% - 10% clay content,
whereas for the Mcs-K and BGB-K mixes it is 15% - 30% clay content at the same range of stress levels. It is clear that the optimum clay content decreases as the plasticity of the clay increases. Above the optimum clay content a point is reached where expansive behaviour is exhibited, although this depends on the applied stress level (Dudley, 1970; Rollins et al, 1994).

One of the most interesting features of the collapse behaviour of Mcs-B mixes is that the magnitude of hydro-collapse is generally much less than that for Mcs-K mixes. For example the maximum hydro-collapse at an applied stress of 200 kPa for Mcs-B mix is about 3%, which was obtained at about 9% clay mineral content (10/90 C/M ratio), whereas the maximum hydro-collapse for the Mcs-K mix is about 13.5% which was obtained at about 25% clay mineral content (40/60 C/M ratio).

The compressibility of Mcs-B mixes is generally less compared to that of Mcs-K mixes, as can be proved by the higher values of the yielding pressure for both the dry and wet e-log p curves of the specimens tested in the oedometer (see Figures 7.52, 7.53). The yielding pressure for the Mcs-B mix of 7.5/92.5 C/M ratio is about 800 kPa for dry specimens and about 100 kPa for the wet material. For the Mcs-B mixes of 12.5/87.5 and 15/85 C/M ratios the yielding stress is more than 1600 kPa for dry material and about 200 kPa for wet material.

Figure 7.54 shows the effect of clay mineral content (Bentonite content) on the hydro-collapse magnitude of the pure silt-Bentonite (Mcs-B) mix at different stress levels (see Table 7.4). The general trend of most curves in Figure 7.54 indicated that the amount of hydro-collapse increased by increasing the clay mineral content up to a maximum value and then decreased as the proportion of clay material increased further. These peak values of
hydro-collapse correspond to the optimum clay mineral content. Also from these curves, it could be noticed that the optimum clay mineral content at which a maximum hydro-collapse would be obtained depends on the level of applied normal stress. For example at 50 and 100 kPa stress levels, the maximum hydro-collapse values of 1.16 and 1.85% were respectively obtained at about 7% clay mineral content, whereas for stress levels of 200 and 400 kPa, the peak values of hydro-collapse of 3.03% and 4.80% were respectively observed at approximately 9% content of clay material. For 800 kPa and 1600 kPa stress level, no peak value of hydro-collapse was reached. This means that the optimum clay mineral content is higher than 18.4%, the maximum amount of clay content which was used in Mcs-B mixes. It is clear from Figure 7.56 that the hydro-collapse increases rapidly with a small increase in clay content (4.5%) and then rises and falls gently around the peak value, whereafter it decreases sharply by further increasing in the clay content. All these changes in the hydro-collapse behaviour of Mcs-B mixes occur in a small range 4.5-14.0% of clay mineral content.

The relationship between the hydro-collapse magnitude and wetting pressure for Mcs-B mixes is shown in Figure 7.55. From this figure it can be noticed that the general shape of the curves is different from those curves for BGB-K and Mcs-K mixes. The main differences seem to be that in the case of Mcs-B mixes there are no maximum hydro-collapse values at certain applied pressures and at stresses between 50 kPa and 200 kPa the curves are very close to each other (i.e. the variation in the magnitude of hydro-collapse is small in comparison with those results for the other mixes). Figure 7.56 also shows that the gradient (the rate of change) of the curves increases as the clay/silt (C/M) ratio increased, and the variation in hydro-collapse increases with increasing applied stress.
The effect of mixing method on the hydro-collapse behaviour of loess deposits is shown in Figure 7.56. The main feature of this figure is that the hydro-collapse values for the specimens prepared by wet mixing method are much higher than those for the specimens prepared by dry mixing method. This result is attributed to the degree of mixing and the distribution of the clay minerals in between the silt particles, the wet mixing method being capable of better mixing. It can also be seen that the curves for the two mixing methods converge as the percentage of clay gets higher and the maximum hydro-collapse for the dry mixing specimens occurred at a C/M ratio higher than that for the wet mixing specimens.

7.4.5 Pure Silt - Clay Size Quartz (McS-Qc) Mixes

The hydro-collapse models for loess mixtures prepared by mixing clay-free silt particles with crushed quartz particles of clay size at different C/M ratios (10/90, 20/80 and 30/70) are similar to those models obtained by testing the McS-B and BGB-K mixes. The results are presented in Figure 7.57 which show that the collapsibility increases as the proportion of clay size particles increased. The effect of the percentage of the clay size material on the hydro-collapse magnitude of the mixture is shown in Figure 7.58, which indicates that the hydro-collapsibility of loess material is related to the clay-sized particle content, as observed from the other loess mixes. It can be seen that the hydro-collapse value increased from about 1.67% for 10/90 C/M ratio to about 7.95% for 40/60 C/M ratio. From the results it is apparent that fine material of clay size is important and has a significant effect on the hydro-collapse properties of loess deposits, since clay-sized quartz can produce collapse when it is used instead of clay mineral powder to create soil specimens of metastable structure.
Table 7.1 Hydrocollapse Values for Different Natural Loess Materials

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Und. Spec. = Undisturbed Specimens
Rec. Spec. = Reconstituted Specimens
Table 7.2 Hydrocollapse Values for Ballotini Glass Balls - Kaolinite (BGB-K) Mixes

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<thead>
<tr>
<th>Stress Level (kPa)</th>
<th>Hydrocollapse Values (%)</th>
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<tr>
<td>10 kPa</td>
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<td>11.82</td>
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Table 7.3 Hydrocollapse Values for Pure Silt from Crushed Sand- Kaolinite($M_{CS-K}$) Mixes

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CHAPTER 8
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CHAPTER 8
DISCUSSION OF RESULTS

8.1 INTRODUCTION

In this chapter the results of the experimental work are discussed and explained. Comparisons of the measured properties of the natural undisturbed loess with reconstituted model loess have been made in order to evaluate the airfall model. These properties were also compared with those reported in the literature for the other types of loess worldwide, allowing a reassessment of this published research. The hydrocollapse mechanism of loess deposits has thus been clarified. The soil classification system and the phenomenon of fragipan formation have also been discussed in the light of the results obtained in this research. The results of the index properties and the scanning electron microscope analyses of the natural and model loess materials have been discussed in Chapter 5.

8.2 THE VALIDITY OF THE AIR-FALL MODEL

The oedometer machine is normally used to demonstrate the hydrocollapse process of collapsing soil, usually with respect to carefully prepared specimens of undisturbed material. However the use of undisturbed specimens means that many important variables are difficult and sometimes impossible to investigate properly. One of the primary aims of the research was to examine whether realistic oedometer specimens could be prepared by direct airfall
sedimentation since this would allow a whole range of variables to be tested, including particle size, shape, nature and size distribution, voids ratio and packing density, types of fluid present, and nature and amount of clay minerals. The aim of this discussion is consequently to demonstrate that a simple airfall method for specimen preparation produces satisfactory samples of collapsing loess by comparing the oedometer test results for natural undisturbed and artificially reconstituted specimens prepared from Khoms and Gharyan loess in Libya and from Malan loess in China.

The results from the Libyan loess indicated that similar behaviour can be observed for both soils (see Figures 8.1 and 8.2). In the air-dry state both undisturbed and remoulded specimens exhibited insignificant compression over a range of applied stress lower than 100 kPa. However, the reconstituted specimens revealed significantly greater compression at stresses higher than 100 kPa when compared with the undisturbed material. In the saturated state the reconstituted materials exhibited a higher compressibility response than the undisturbed materials, particularly in the range of low applied pressure. It is apparent from Figures 8.1 and 8.2, therefore, that the two air-dry curves for both soils started to diverge at a vertical stress level of approximately 100 kPa, whereas the two saturated curves became convergent at this pressure and the distance between them decreased as the applied pressure increased. It seems likely the curves would coincide at a vertical pressure higher than 1600 kPa. This structural collapse behaviour of the naturally bonded and unbonded materials may be attributed to the effect of the interparticle bonding developed at the contacts of the silt particles.

The hydrocollapse-applied pressure relationships for undisturbed and reconstituted Libyan loess specimens are presented in Figure 8.3. From this figure it can be observed that the interparticle bonding materials have a
significant effect on the hydrocollapse of these soils. The degree of hydrocollapse for the unbonded materials is higher than that for bonded ones at lower levels of applied stress, and lower in the case of higher applied stresses. Therefore at a certain level of applied pressure both materials have the same degree of hydrocollapse. For Khoms soil this pressure was found to be approximately 340 kPa and for Gharyan soil approximately 520 kPa.

The results of the tests on undisturbed and reconstituted Malan loess are shown in Figures 8.4 and 8.5. It can be seen from Figure 8.4 that for the specimens of higher initial voids ratio the artificial (reconstituted) samples undergo greater compression under increased stress prior to wetting than the undisturbed samples. In contrast the reconstituted specimens of voids ratio approximately similar to the undisturbed material exhibit almost identical compression behaviour to that of the undisturbed specimens, as shown in Figure 8.5. This difference in compressibility is clearly a result of the difference in the initial voids ratio. As observed for Libyan loess, both reconstituted samples undergo greater collapse on wetting (or hydroconsolidation) at lower stress levels. This is due to both stress and environmental histories, the natural undisturbed specimens having been subjected to greater levels of vertical stress in their history and long-term bonding has occurred. Of perhaps greater interest is the relationship between the post-wetted curves, those for the undisturbed specimens having a somewhat steeper gradient. This implies that the bonding in undisturbed samples only partially breaks down immediately on wetting at lower stresses, but that a lower ultimate void ratio will be reached if stresses are increased to high levels.

According to these results, it seems that the hydrocollapse model of naturally deposited loess materials can generally be simulated in the laboratory utilising the airfall technique, and a good approximation to behaviour in situ is possible.
Using artificial materials prepared by mixing pure silt from crushed sand or Ballotini glass balls with clay powder to create airfall loess specimens does not completely reflect the hydrocollapse process of materials *in situ* but still provides a proper way to investigate the effect of compositional factors such as the geometrical properties of the silt particles and the type and amount of clay minerals on the collapse behaviour of loess material. Therefore soil specimens of metastable structure can be produced using the airfall technique, and repeatable results have been obtained in single and double oedometer tests. This in turn leads to the primary premise that if it can be shown that the airfall technique models a real loess deposit, which it evidently does, it will therefore allow the introduction of a whole variety of variables. It will be possible to test a wide range of clay contents, and to start from a wide range of voids ratios. This will considerably advance the subjects of formation and collapse behaviour of loess deposits. Roscoe (1970) suggested certain general research objectives for soil mechanics, the first one being to deal with soils in their simplest possible states so that their engineering properties can be defined by the minimum possible number of parameters. This approach has been followed in this research and is discussed hereafter.

8.3 COMPRESSION - TIME RELATIONSHIP

The test results of structural deformation during the hydrocollapse processes for different loess materials are plotted against the logarithm of time in Figures 8.6 and 8.7. The general shape of the curves indicates that the hydrocollapse process may not be a uniform one, with respect to time. It seems to be similar to the deformation-log time curve for a given load increment obtained from consolidation tests on fine-grained soil. Three regions of collapse processes
during flooding can be noticed from curves. A time lapse occurs initially as water flows through the dry porous stones and filtration papers and enters the specimens (region 1). As water moves and distributes throughout the soil specimen, which results in a softening of clay bridges and aggregates, the majority of hydrocollapse takes place until it is essentially complete (major hydrocollapse compression, region 2). Some deformation continues slowly to occur at constant effective stress (post-hydrocollapse compression, region 3).

The effects of clay minerals (type and content) and applied stress level on the rate of hydrocollapse were investigated from single-oedometer tests on specimens containing varying proportions of different silt particles under nominally identical conditions. The compression - log time curves for both BGB-K and Mcs-K mixes indicate that the densification processes are relatively very rapid (see Figure 8.6). More than 95% of the total compression (hydrocollapse settlement) occurred within 20-30 minutes. For the silt-bentonite (Mcs-B) mix the process of hydrocollapse takes much longer (more than 24 hours, see Figure 8.7. This is because of the high plasticity and low permeability of the bentonite.

It is apparent that the level of applied vertical stress had no significant effect on the rate of hydrocollapse for all mixes having kaolinite as a clay mineral (see Figures 8.8 and 8.9). This may be due to a balance effect between the stress level, which would be expected to increase the rate of deformation, and the relatively low voids ratio and hence permeability, which would tend to decrease the rate of compression. Figure 8.10 shows that the stress level had a significant effect on the hydrocollapse rate of loess specimens having bentonite as a clay mineral. This is because of the high susceptibility to the volume change of the bentonite.
Other investigators have found similar trends. For example, Mellors (1977) found that most of the settlement took place within 20 minutes. Booth (1977) found that 90% collapse for different samples of silty soils tested under a variety of moisture, density, and stress conditions generally occurred in less than 25 minutes. The total compression measured after flooding a loess specimen under a certain vertical stress was divided into two components by Tahir (1970) and Browzin (1981):

(a) Subsidence (hydrocollapse), the compression which occurs rapidly at an accelerating rate of deformation within a very short period of time typically (0.5 - 10 mins) before the uniform decelerating rate of deformation is achieved.

(b) Post-subsidence consolidation, the residual deformation after the completion of subsidence occurs at a uniform decelerating rate under the same stress condition.

According to the observations obtained from this research it is possible to criticise the hydrocollapse (subsidence) definition introduced by Tahir (1970). Based on the Tahir definition the complete hydrocollapse process occurs in less than two minutes for most loessial soils. He also used the speed of collapse as a base for his interpretation. However, the accelerating rate of compression seems to be related to the deformation of the "very large and large" pores distributed inside the soil mass; prior to deformation of these pores the soil permeability is very high and thus the hydrocollapse speed will be high. Similarly, the decelerating rate of compression seems to be due to a reduction in the voids ratio of the material and hence its permeability. Therefore the gradual decrease in permeability resulting from hydrocollapse is considered the likely reason for the overall lower ratios of settlement observed during flooding. Air bubbles were seen to come out from the specimens immediately after the addition of water and continue for about 10 to 20
minutes. The same observation was recorded by Mellors (1978). It is probable that some air could be trapped within the specimens, and this may cause the compression to continue at a decreased rate. However, the interrelationship of the propagative nature of the hydrocollapse process and the changing permeability is considered to be the main reason for the non-uniform hydrocollapse.

The decrease in the rate of compression cannot therefore be considered to be an evidence of a consolidation process, but an indication of structural deformation and solid particle rearrangement leading to higher density, lower voids ratio, lower permeability, and thus a lower rate of hydrocollapse. Another important observation is that the timing of the hydrocollapse process is controlled by the time required for all the air to come out from the soil specimens.

It can be concluded that the variation in the rate of hydrocollapse is mainly related to the type and amount of clay minerals in the soil, particularly for loess materials having clay minerals of low susceptibility to the volume change. In the case of low plasticity clay such as kaolinite, the permeability of the mixture is very high and the water is allowed to flow very quickly inside the sample. As a result of rapid softening of the clay bridges between the particles, the hydrocollapse occurs rapidly. In the case of high plasticity clay such as bentonite, complete softening of the macropeds and distortion of the clay bridges requires infiltration of water throughout the whole soil, which occurs more slowly because of the relatively low permeability of the bentonite. The hydrocollapse thus takes longer to complete. Nevertheless, regardless of the clay mineral, the test results indicate that hydrocollapse of loess takes a very short period of time to complete. The normal consolidation process of fine-grained clayey soils in situ can take many months or years to be complete.
whereas the hydrocollapse settlement of loessial soil occurs typically within days or weeks.

8.4 INTERPARTICLE BONDING-HYDROCOLLAPSE RELATIONSHIP

The clay content, clay type and distribution of clay particles inside the soil fabric are the main bonding features influencing the hydrocollapse behaviour of loess. According to the results of this investigation, it was proved that the presence of clay is essential for collapse to occur and that there was an optimum clay content varying with the clay minerals and stress level (see Figures 7.15, 7.28, 7.43 and 7.55). The results presented in Figure 7.15 show that pure silt (i.e. clay-free soil system) does not exhibit any significant hydrocollapse and that the behaviour of the material is similar before and after wetting. These findings confirm the hypothesis proposed by other researchers based on experimental studies of natural undisturbed loess materials. Lobdell (1981) concluded that a key factor in evaluating the collapse potential of a loessial soil appears to lie in the mineralogy of the soil, specifically that of the clay minerals which comprise the majority of the bonding material in loess deposits. Clemence and Finnbarr (1981) and Mitchell and Coutinho (1991) indicated that the existence of clay particles as a bonding material is essential (fundamental) in the formation of collapsible soil. Mellors (1995) found that the hydrocollapse phenomenon of loess deposits appears to be directly influenced by the behaviour of the clay fraction in the soil. Barksdale and Blight (1997) reported that the blanket of clayey sand which covers extensive areas of Southern Africa was originally deposited with a loose fabric and it has a collapsible structure because of the presence of the clay (Knight 1961).
Dudley (1970) stated that, although the presence of clay is not essential, a small amount contributes to the magnitude of the total collapse with the optimum amount varying with the type of clay minerals and the size fractions of the silt particles. He also indicated that maximum collapse occurs when the clay content is about 12%; below 5% there is little collapse and above 30% the soil swells. He did not specify the type of clay, but generally these findings seem to agree with the findings obtained in this research.

Bull (1964), who was cited by Clemence (1981), showed that maximum hydrocollapse occurs when the quantity of clay is around 12%. Tahir (1970) found that loess samples of clay content higher than 13% tend to give low subsidence (hydrocollapse), whereas those of clay content less than 13% are generally more susceptible to subsidence. These findings agree with the results of the pure silt-bentonite (Mcs-B) mixtures obtained in this research. Tahir concluded that the clay content has a significant influence on hydrocollapse (subsidence) of loess materials. Lawton et al (1992) reported that results of soil specimens prepared from various sand-silt-clay mixtures at the same initial dry density and water content indicated that collapse potential reaches a maximum value at a clay fraction between 10% to 40% by weight.

Handy (1973) concluded that a preliminary estimation of collapsibility of Iowa loess may be made on the basis of clay content:

a) Loess containing less than 16% of clay size material (2 μm) has a high probability of collapse upon wetting.

b) Loess containing 16 to 24% of clay size material is probably collapsible.

c) Loess containing 24 to 32% clay size material has less than a 50% chance of collapse, but still shows some likelihood of collapsibility.

d) Loess containing more than 32% of clay size material is usually safe from collapse, but not from other types of failure.
Handy also observed that no collapse or very small collapse will occur at very low clay content. These findings were made particularly for Iowa loess which has a very high voids ratio (0.9 - 1.25), and all specimens having a voids ratio less than one have no collapse. Handy in his study did not examine the type of clay mineral, classified the clay content based on the grain-size criterion, and did not identify the stress level at which the soil was flooded to determine the collapsibility of the material. According to the Handy results it seems that Iowa loess contains clay minerals composed mainly of montmorillonite (bentonite) as reported by Lobdell (1981). The results obtained in this research give support to the observations made by Handy and vice-versa, since it was found that using bentonite as a clay mineral in the model loess gives generally small collapse, the maximum values (2-3%) being obtained at around 6-10% clay content depending on the stress levels (see Figure 8.11).

In the light of this research, it is therefore possible to suggest that the hydrocollapsibility of loess in the United States occurs at relatively very low clay content (less than 13%), because the mineralogy of the clay material consists of illite and montmorillonite (medium to high plasticity clay). It was found that in earth dams, the soils that collapsed varied from silty sand to medium plasticity clay. This shows that soils containing high plasticity clay do not tend to collapse, supporting the results obtained in this research that soil containing bentonite (a high plasticity clay) as a bonding material exhibits relatively little collapse compared with that containing kaolinite (a low plasticity clay).

Results of hydroconsolidation tests conducted by Lobdell (1981) indicated that Palouse loess undergoes very little structural collapse due to inundation. This suggests that Palouse loess is not a typical loessial soil like those of the midwest. He stated that when typical loess is saturated, the clay cement adsorbs
water and a loss of strength occurs. If the applied stress is sufficient the weakened clay will fail in shear at points of contact between grains. Destruction of the intergranular supports allows the grains to move into void spaces yielding a net decrease in volume and a permanent settlement. The loss in strength of a clay-bonded (cemented) soil was thus attributed to the nature of the clay mineral which holds the silt particles together. He also stated that the major cementing agent in most loess deposits of the United States is montmorillonite whereas the chief cementing agent of Palouse loess is illite, with only a trace of montmorillonite. Different clay minerals vary in their ability to adsorb hydrogen cations and thus in their ability to adsorb water. He stated that as montmorillonite has the greater ability "to adsorb hydrogen cations (water) and since Palouse loess is cemented primarily by illite, it displays little or no softening when the moisture content is increased and thus possesses a low collapse potential.

This interpretation does not agree with the findings of the author and other researchers however. In this research M-K mixes exhibited much greater collapse than M-B mixes (see Figure 8.11), soil mixes having bentonite as a cementing agent exhibiting a peak value of collapse of about 3%. In addition the dominant clay mineral in Malan loess is illite (Jia-Mao, 1985), there being only a small content of kaolinite and montmorillonite, and it exhibits significant collapse. It can be concluded, therefore, that the main reason why Palouse loess is not collapsible is the relatively high dry density (1.52 - 1.56% Mg/m³), as observed by Lobdell himself, perhaps combined with the fact that illite has a relatively high plasticity (LL ~ 100, PI ~ 40) compared to kaolinite (LL = 60, PI = 25). The Malan loess has significant collapse because of its open structure and high voids ratio (e ≥ 1).
Tahir (1970) reported three types of interparticle bonding: calcareous bonds, clayey bonds, and combined calcareous and clayey bonds. He stated that calcareous bonding could not be the dominant means of bonding in loess because subsidence (hydrocollapse) would not occur, calcium and magnesium carbonates having no, or very little, solubility in water. His comment that both the quantity and quality of clay present in loess significantly affects subsidence is supported by this work. It was equally supported by Berg (1967), who stated that those samples from Tashkent loess which have montmorillonite as a clay mineral swell easily and this closes the pores in loess. The corresponding soils were thus classified as non-subsiding. Conversely, those samples having kaolinite as the dominant clay mineral were stated to have a tendency to subside. From Rumania, Lehr (1967) also concluded that loess containing montmorillonite has a much lower tendency to subsidence than loess containing kaolinite.

Tahir (1970) also suggested that the interparticle bonding of loess structure results from the combined effect of clay and carbonates. He noticed that loess samples with a combination of a low clay content (less than 13%) and a high carbonate content (greater than 6.2%) gave a higher degree of subsidence (hydrocollapse). He suggested that samples with a higher calcium carbonate (CaCO₃) content showed a greater sensitivity to subsidence. However, it was proved that the calcium carbonate alone does not produce significant amounts of loess hydrocollapse (see Figure 8.12). Carbonates have been shown to work as a fine material which alters the grain size distribution features of the soil, particularly the filling fraction (2-10µm) as defined by Browzin (1984). This may have a slight effect on the hydrocollapse properties. However, calcium carbonate cannot be considered as necessary for loess hydrocollapse.
This theme was taken up by Mellors (1995), who reported that the calcium carbonate does not play a significant part in the strength of the interparticle bonds. This conclusion was derived from the observation that the Pegwell Bay loess, which contained 12-20% calcium carbonate, underwent a significant degree of hydrocollapse when it had been flooded by calcium chloride solution. He stated that "if the calcium carbonate occurred as a cement in the soil, the addition of calcium chloride solution would have inhibited the breakdown of any calcium carbonate cementation". He also confirmed this observation by a SEM study which indicated that there was no readily discernible concentration of calcium cations (Ca$^{2+}$) at points of contacts which could suggest a carbonate cement.

The effect of soil fabric on the hydrocollapse behaviour of loess has been studied only qualitatively until now, quantitative study being one of the important points of this work. In this research the effect of the fabric was investigated through the testing of soil specimens prepared by different methods of mixing: poor (dry) and good (wet) mixing. These different methods produced loess specimens of different fabrics, mainly due to the position and distribution of the clay particles in between and around the primary silt particles. In the dry mixing methods it is very difficult, and probably impossible, to distribute uniformly the clay mineral around the silt particles, resulting in some concentration of clay materials and equally some areas in which there is a relatively high percentage of direct contact points (grain to grain contact) with no clay between the silt particles. In the case of wet mixing the clay particles are relatively well distributed and a lower percentage of direct contact points exist.

As shown in Figure 8.13, it is clear that the hydrocollapse values of specimens prepared by wet mixing are consistently higher than those of the dry mixed
material. This is attributed to the degree of distribution of the clay particles and their position. This interpretation can be clarified by considering the hydrocollapse values at low and high clay contents. In the case of very small clay content, the curves are generally closer to each other. This is because the efficiency of mixing has very little overall effect: most of the contact points are of grain-to-grain nature (free of clay) and the high friction between the particles will resist their movement. Relatively small and similar amounts of hydrocollapse will occur.

In the case of high clay content, the primary silt particles will be floating in the medium of clay material, the mixing method again having no significant influence and relatively small and similar hydrocollapse occurring. In between these limiting states, it is noticeable that the difference between the curves (i.e. the difference in the degree of hydrocollapse) is greatest as the optimum clay content is approached. This is illustrated in Figure 8.14.

8.5 THE EFFECT OF THE GEOMETRICAL CHARACTERISTICS OF SILT PARTICLES ON THE HYDROCollapse BEHAVIOUR OF LOESS

As introduced in Section 4.4.3, Rogers et al (1994a) indicated that the collapse properties of loess deposits depend in some part on the properties of the individual particles which form these soil formations. Figure 8.15 shows the relationship between the clay mineral content and the degree of hydrocollapse for BGB-K and McS-K mixes at low to medium stress levels (10-200 kPa). Three main features can be extracted from this figure: the general shapes and trends of the curves are similar for both mixes, the applied stress has a significant influence on the degree of hydrocollapse and the Ballotini glass
balls-kaolinite (BGB-K) mixes undergo greater degrees of hydrocollapse (particularly at low normal stress). Figure 8.16 presents the equivalent data for higher stress levels. The main feature shown by this figure is that the curves for both mixes are generally parallel and they are much closer to each other, particularly at very small clay contents.

These important and interesting features can also be derived from Figures 8.17 and 8.18, in which the relationship between the applied stress and the hydrocollapse for both mixes at low and high clay contents are illustrated, respectively. In Figure 8.17 it is very clear that the differences between the curves are much larger at stresses lower than 100 kPa than at higher levels of applied stress. Indeed the curves for the two materials cross each other for the lowest clay content at very high stress. The curves in Figure 8.18 are approximately parallel and close to each other, specifically at stresses higher than 400 kPa.

These observations are further illustrated by considering the differences in magnitude of hydrocollapse in Figures 8.19 and 8.20. From these two figures it can be noticed that there is a certain range of clay mineral content and stress level at which the difference in hydrocollapse values of the two mixes is maximum. These ranges are 13 - 20% for clay mineral content (i.e. 20/80 - 30/70 as kaolinite/silt ratio) and stresses of 10-50 kPa. It is clear that the range of clay content for maximum difference is similar to the range of the optimum clay content for maximum hydrocollapse. However, in general the difference in hydrocollapse values decreases as both the clay amount and applied stress increased. From Figure 8.20 it can be seen that at low stress levels (10-50 kPa) the differences in hydrocollapse values are generally much higher than those at high stress levels (400 kPa - 1600 kPa). Maximum differences were observed at 13% clay mineral content (20/80 K/M ratio) for 10-50 kPa, whereas for all
other stress levels the difference peaks occurred at approximately 20% clay mineral content (30% kaolinite powder content). The hydrocollapse differences also decreased as the amount of clay mineral increased. Also at the higher range of stress it is clear from both figures that the clay content has no significant effect on the difference of hydrocollapse. However, Figure 8.20 shows that maximum differences for 10/90, 15/85, 20/80 and 25/75 kaolinite/silt (K/M) ratios were obtained at 25 kPa stress level, but for all other ratios they occurred at 50 kPa.

This behaviour can be explained by the observation that the geometrical characteristics of the primary silt particles work most effectively at a certain clay mineral content, above which the clay reduces this effect by forcing the primary particles apart. This reaches a limiting state where there is more than enough clay to fill the minimum voids of maximum dry density. In this case the soil will behave as classical clayey soil.

At high stress levels (400 kPa - 1600 kPa) the effect of the clay mineral content on the difference in hydrocollapse is much reduced. For example at 400 kPa the difference varies from 2 to 4% and 1600 kPa it varies from 1 to 2%. In a similar way the effect of applied stress on the hydrocollapse difference was much reduced at higher clay contents.

Alwail (1990) suggested that the shape of silt particles greatly affects the collapse potential of silt-sand mixtures. Ottawa sand mixed with an angular silt containing small (< 10%) amounts of clay had significantly higher collapse potential compared to the same sand mixed with a rounded silt with negligible clay content. Based on the observations of this research, it seems that the difference in the collapsibility of the silt-sand mixtures is primarily due to the variation in the clay content, and not the shape of the particles, although it was
proved that the roundness of the silt particles tends to increase collapse and the angularity of the particles helps to decrease it.

In summary the roundness and smoothness of the glass balls and the angularity and roughness of the silt particles derived from crushed sand are the main solid particle features that have a significant effect on the hydrocollapse behaviour of loess material. At higher clay mineral content and high stress level this difference was reduced because of the reduction of the effect of the shape factors. Thus it can be concluded that at high clay mineral content both BGB-K and Mcs-K mixes become geotechnically similar materials with similar behaviour (see Figure 8.18). This is because of the effect of the shape factors was masked and all other compositional properties are the same.

In addition to the shape factors, the main reason that the difference in hydrocollapse is relatively small at high stress level is the much steeper dry compression curves (upper curves) for the BGB-K mix causing rapid convergence with the saturated compression curves (lower curves), as shown in Figures 8.21 and 8.22. The effect of this is to produce similar degrees of hydrocollapse at the very high stress levels. Another important observation is that the maximum differences occurred at the yield stresses of the dry compression curves of the this mix.

8.6 THE MECHANISM OF HYDROCOLLAPSE

The open structural arrangement of the primary silt particles and the clay bonds which connected them are the key factors controlling the hydro-collapse phenomenon. Soil structural models shown in Figures 2.13a and 4.2 and Plates
5.7, 5.10 and 5.11 demonstrate that a proportion of the clay materials exist as bonds for the silt grains. These bonds undergo local compression in the small gaps between adjacent silt particles resulting in the development of strength. In the case of dry conditions the structure of the soil specimens is in equilibrium under the action of the relatively low applied stress. When the applied stress is increased the structure remains evidently unchanged, the clay mineral compressing slightly without any large relative movement of the soil particles. As long as the specimens remain dry, the local micro-shear forces at the silt particle interfaces, resulting from applied loading, are resisted without any appreciable structural distress. However, when the loaded specimen gets wet, the clay bridges soften such that they can no longer resist the forces (i.e. the shear strength of the clay is exceeded) and structural hydro-collapse occurs, resulting in considerable volume reduction, as shown in Figure 2.13b. The hydro-collapse process involves sliding, rotational, and translational rearrangement of soil particles.

One of the most important points is the inter-relationship between the matric suction (negative pore water pressure) and the hydro-collapse phenomenon. Matric suction is one of the factors that affect the hydro-collapse behaviour of unsaturated soil but not the primary factor, according to this research. Tadepalli and Fredland (1991), however, state that the collapse phenomenon is primarily related to the reduction of the matric suction during inundation, and indeed conclude that matric suction is one of the two stress-state variables that control the behaviour of an unsaturated soil. The explanation for this might be in the fact that the authors here used the term collapsible soil in a general sense and did not identify which type of soil the observation was applicable to. These findings are not compatible with the findings of this research, some of which are as follows:
1. Many collapsible soils have no significant collapse at low stress level when they are wetted, but they have high hydrocollapsibility when they are flooded at applied stress $\geq 100$ kPa. If the primary bonding mechanism were suction, this would not be the case.

2. Unsaturated pure silt soil has no significant collapse even at relatively high pressure. The material behaviour dominates the response, therefore, i.e. the same type of soil can give different amounts of hydrocollapse because of its different fabric. For example, soils having a different distribution of clay particles in between the silt grains have a different collapsibility.

3. Soils of the same compositional and initial properties have different collapse potential because only of a difference in the shape of their silt particles.

One of the key questions is: why does a certain loess material undergo a completely different hydrocollapse from a second loess material at the same stress levels, even though the loess materials have the same initial basic properties (density, void ratio and water content), and also have the same clay type and content? The only difference is the shape and surface texture of the silt particles. The other important question is: why does the same type of soil give different hydrocollapse values at different stress levels? This is apparent when the hydrocollapse results of the $M_{CS}$-K mix are compared with the results of the BGB-K mix (see Figures 8.15 and 8.16), the hydrocollapse values for BGB-K being much higher than those for the $M_{CS}$-K mix at the same stress levels. In addition, the results of the two samples containing bentonite (i.e. $M_{CS}$-B and BGB-B) gave the same trend (see Figure 8.23) This difference in in hydrocollapse behaviour can only be attributed to the difference in the shape and surface texture of the solid silt particles. The same trend in behaviour was
observed when different types of clay (see Figure 8.24) of the same content were used.

Thus the key factors which control the compressibility and hydrocollapse behaviour of loess deposits are the geometrical (shape, size and grain-size distribution) and surface texture characteristics of the silt particles, and clay type and clay content.

Matric suction can thus only explain the hydrocollapse behaviour of uncemented soil containing no clay particles (fine sand, sand-silt mix and pure silt). In addition, if the matric suction is the main factor that controls the process of collapse, then the magnitude of collapse must be approximately the same at all stress levels. This was observed in tests in which oedometer specimens of pure silt derived from different materials were tested under different stress levels, only small variations in the magnitude of collapse being observed. This variation is related to three factors:

(a) Bonding due to the very fine particles particularly the fraction of less than 5μm size. This factor has a relatively significant effect at low stress levels, the small bonding force of this factor being sufficient to resist the small applied stress, causing the collapse at low stress levels to be slightly smaller than that at higher levels of stress.

(b) Bond due to particle interlocking. This effect also decreases as the applied pressure increased.

(c) The effect of wetting: the compressibility of wet material is slightly higher than that of the dry material.

To confirm the conclusion on the type of soil most affected by matric suction, Fredlund and Gan (1995) performed laboratory tests on a collapsible soil. They concluded that the collapse of an uncemented, dry collapsible soil is due
to the loss in the normal stress between soil particles leading to shear failure as a result of a reduction of matric suction from wetting. In so doing, they classified an uncemented soil as one that is free of clay. Their study was conducted on silty sand which contained 62% sand, 32% silt and 6% clay, and the type of clay was not determined. This conclusion thus agreed with the findings of this research.

8.6.1 Clay Content Based - Soil Models

Based on the test results obtained the hydrocollapse mechanism can be explained. For simplicity two micro-structural models for loess soil can be demonstrated: small clay content - soil model, and high clay content - soil model.

8.6.1.1 Small Clay - Soil Model

In this model, there are two specified ranges of clay content: very small and optimum clay contents. Consider first the very small clay content condition. Addition of a very small amount of clay caused more compression when compared with specimens which were free of clay (i.e. pure silt). This compression increased as the clay content increased. The clay, in this case, worked as a lubricant material at the contact points between the silt particles and hence facilitated their sliding against each other.

The main feature of this model is that the clay particles are very small, and most of the contacts between the primary silt particles (loessial fraction) are free from clay bonds, i.e. only a small number of contact points have been provided with clay minerals. This particular bonding system can be used to explain the mechanism of structural deformation due to wetting
(hydrocollapse). Initially, at the highest void ratio conditions, the open structure is maintained mainly by small bonded contacts, and this structure collapses significantly when it gets wet under very low stress levels (i.e. 5 kPa, see Figures 7.28 and 7.43). Even in the dry condition, the BGB-K mix deforms significantly, which means the yielding pressure of the dry curve is relatively low (see Figures 7.17 to 7.20). After this stage the rate of deformation will be smaller particularly at relatively high stresses. This behaviour is because the stresses at the contact points are high enough to break down the clay bonds even though the applied stress is very low. The soil is thus initially deformed at low stresses because of the limited bonded contact points, after which a large percentage of direct grain to grain contacts will be produced which can resist the shear stress more effectively. This is most evident in the case of the crushed sand.

The second model is the optimum clay content condition. For the silt-clay (M-C) mixtures containing kaolinite, the optimum clay content (i.e. that at which maximum collapse occurred) falls in the range 15-25% for intermediate stress levels (50, 100, 200, 400 kPa). The soil structure at this stage consists of relatively large silt particles separated at their contact points by clay packets or clay macropeds. Before flooding, the structure was able to resist loads by the internal resistance resulting from the bonds caused by the clay bridges. As the soil was soaked the structure collapsed. Upon saturation the clay worked as a lubricant between the silt particles at the contact points and caused sliding of silt grains against each other. The sliding continued until the structure reached equilibrium and regained its stability. The structure after equilibrium could be assumed to consist of silt particles in direct contact with voids full of saturated clay aggregates. By increasing the clay content further, the swelling of the clay macropeds started to give an effect opposite to the compression caused by sliding of silt grains and distortion of the clay packets. This opposite effect
caused a decrease in the compression as the clay content increased. In this
model the combination between the primary silt particles and the clay particles
allows the soil to achieve the highest degree of densification (highest degree of
collapse). In this case there is a good balance between the following three
parameters:

1. particle movement is facilitated by a maximum number of bonded
   contacts (more surface area of the primary particles covered with the
   clay minerals).
2. the amount of fine material, which includes the clay particles, must be
   just enough to fill the voids without forcing the silt particles apart and
   hence to achieve the maximum density.
3. The effect of the swelling must be a minimum.

The hydrocollapse behaviour of this model is shown by Figures 7.28, 7.43 and
8.15. It is clear that maximum collapse occurred at very low stress (5 kPa)
when the clay/silt ratios were 20/80 and 25/75, because at these ratios the
amount of clay is great enough to produce a good lubrication system, and small
enough to make the swelling effect minimum.

8.6.1.2 High clay - soil model

In this model, the amount of clay is more than that needed to fill the voids, and
hence sufficient to force the silt particles apart and make them 'float' in a
medium of clay particles. In this case the swelling factor has a significant
effect on the deformation process and the hydrocollapse is greatly reduced.

As the clay mineral content of the kaolinite mixes increased to more than 30%,
specimens of relatively high voids ratio (0.90) started to show a significant
reduction in the degree of hydrocollapse under a relatively small stress (< 25
kPa), and even swelling potential was observed for specimens of very high clay
content (> 40%) at this stress level. At higher stresses, however, they still
exhibited relatively high compression: specimens subject to pressures as high as 400 kPa, 800 kPa and 1600 kPa experienced hydrocollapse of approximately 14%, 16% and 15% respectively.

In this state it can be said that the open structure having a high percentage of pore spaces and a high clay content consists of silt grains embedded in the clay medium, and therefore the swelling of the clay material under small stresses predominated causing a significant reduction in the compression. Under high pressures the clay packets experienced compression, although the swelling of the clay reduced this compression to some extent. Too much clay material simply turns the system into a normal plastic soil having long-range bonding. This model of the soil system appears to be the situation in New Zealand, for example where the highly clayey loess tends not to exhibit hydrocollapse.

In summary, the mechanism of collapse of loess deposits is fundamentally governed by the ease with which the primary silt particles can move against each other. It was found that loess material of a certain percentage of clay mineral exhibited maximum collapse, the degree of collapse reducing with less or more clay content. It was observed that loess material manufactured from smooth, round silt particles collapsed more than that made from rough, angular and sharp edged silt particles. Mellors (1995) proved that the amount of collapse increased as the degree of dispersion of the clay material increased. All these results confirm the concept that the collapse mechanism is basically controlled by the ability of the silt particles to move against each other.
8.7 HYDROCOLLAPSE AND SOIL CLASSIFICATION

The soil classification system is the arrangement of different soils with similar properties into groups and subgroups. This system provides a common language amongst engineers to express briefly the general characteristics of soils. Most of the soil classification systems that have been developed for engineering purposes are based on simple index properties such as particle size distribution and plasticity. The current classification systems are thus more applicable for remoulded soils than for those in their natural undisturbed state. The most commonly used engineering soil classification systems were mainly developed for earthworks, highways, earth dams or airport work (Carter and Bentley, 1991).

The identification of collapsible soils and the prediction of the amount of collapse that may occur due to a change in moisture content, loading system or both is very important in preventing damage to structures and foundations. The engineering properties and behaviour of collapsible soil is not well predicted by the current soil classification systems such as BS, Unified or the AASHO system alone, because the collapse behaviour of these types of soil are mainly related to the initial structural properties of the soil which are not included in the classification system processes. Consequently, a number of other criteria for the identification of collapsing soils have been proposed, and they are summarised in Chapter 2. Rogers et. al. (1993) reported that the classification of arid soils (one of the main groups of collapsible soils) is shown to be unsatisfactory using existing classification systems. They suggested that a comprehensive development of the current systems is required to provide a single, universal system, which is thus equally applicable to all types of collapsible soil, for example including arid soils, tropical soils, and residual soils.
It is known that the most widely used and most generally applicable classification is the Unified Soil Classification system (Bentley and Carter, 1991). Rogers et. al. (1993) suggested that this method can be used as a base for the new system. They indicated that the properties which must be taken into account in a classification that includes collapsible soils can be categorised into three groups and suggestions were made for their measurement. For example mineralogy is of prime importance, but its direct measurement is not generally feasible. Thus simple field or laboratory tests that indicate the effects of different types of mineral are required. The primary additional considerations for collapsing soils concern the structure of the soil and its response to loading and wetting.

Wesley and Irfan (1997) proposed a practical system for classifying residual soils (one of the main types of collapsing soils) based on mineralogical composition and structural characteristics. They stated that there are specific features of residual soils that are not adequately covered by conventional methods of soil classification such as the Unified Soil Classification system. These characteristics which distinguish the residual soils from other types can generally be attributed to the presence of specific clay minerals and particular structural properties. On the basis of mineralogical composition, they basically divided the soil into three main groups. Based on the structural effects, each mineralogical group was divided into three sub-groups. The main disadvantage of this classification system is its use of mineralogical composition as a starting point for establishing the three principal groups.

The results obtained in this research emphasise the great importance of the geometrical characteristics (shape, size and size distribution and surface texture) of the solid particles, composition, structure, and stress state as
determining factors for the behaviour of metastable soils (see Figures 8.13 to 8.20).

Collapsible soil derives its volume change properties from both its special composition and its metastable structure that is easily changed as a result of moisture content and stress changes. It is thus clear that successful geotechnical engineering works require as much knowledge about the properties of the constituents and the structure of the materials as about the mechanics of such a soil.

Therefore, in order to make a powerful soil classification and one that is more useful for design, particularly for silty soil, some additional basic tests must be taken into consideration, such as void ratio, dry density, clay type and content, and the oedometer test on undisturbed and reconstituted specimens. However, all current soil classification systems are based on very fundamental and simple tests: plastic limit, liquid limit and grain size analysis. These tests give insufficient information to know whether the soil is collapsible, exhibits swelling or produces other problematic behaviour. For example, as shown in Figures 8.13 and 8.14 silty soils of identical index properties gave different values of hydrocollapse because of their different fabrics. Since there is currently no method of testing to measure quantitatively the soil fabric, an indirect testing method (for example the oedometer test) must be used to determine the effect of the soil fabric on the collapsibility of soil.

Based on the work of the previous investigators, which has been supported and confirmed by the results of this research, a system of multi-stage testing for classification is suggested as follows:
Stage 1 - Basic Classification Stage. This consists of the procedures used currently and gives only a general idea about the soil: it can be compositionally identified, it gives it its geotechnical name, and it provides indications (backed up by experience) of its likely behaviour if it occurs in a naturally dense (non-collapsible) form. The main soil tests required are:

a. plastic limit
b. liquid limit
c. grain size distribution analysis

Stage 2 Advance Classification Stage. The main aim of this stage is to determine whether the soil is a normal soil, or whether it is a collapsible, swelling or other type of problematic soil. The main soil tests required in addition to the Basic Classification Stage tests are:

a. Dry density and specific gravity to determine the voids ratio.
b. Type and content of clay material, the soil fraction of particle size less than 2 μm.
c. The geometrical properties of the solid particles which form the skeletal frame of the soil structure (silt and sand fractions).
d. Hydrocollapse test using the oedometer to determine the coefficient of collapsibility. Also by using the oedometer the coefficient of swelling can be determined.
8.8 HYDROCOLLAPSE PHENOMENON AND FRAGIPAN FORMATION

Fragipans consist of a slightly hard and brittle layer found generally in loess soils at a depth of about 0.75 to 1.50m. Fragipans are very important pedological features that occur in Europe, New Zealand, the USA, and other parts of the world (Smeck and Ciolkosz, 1989). From an agricultural perspective, fragipans are undesirable because they restrict rooting depth and retard removal of excess water that results in decreased productivity.

The phenomenon of fragipan formation is still a matter for investigation and discussion. Bryant (1989) suggested that the hydroconsolidation process may contribute to the formation of this phenomenon. Experimental results of the hydrocollapse behaviour of loess materials are presented and discussed here in order to test the Bryant hypothesis. The proposed theory is that fragipan formation occurs when some soils are loaded and wetted. Bryant (1989) stated that "self-weight collapsed sediments suggest an alternative hypothesis for physical densification with particular focus for application to the formation of fragipans developed in loess deposits in the lower Mississippi valley; although it may certainly have much wider application". Bryant cited the work of Bull (1964) on the alluvial fans and near-surface subsidence in western Fresno County in California, in which he proposed that a clay content of 5 to 30% was necessary for collapse to occur. Investigations of fragipans by Petersen et al (1970) showed that this particle size range was typically found in fragipans. This perceptive observation by Bryant makes the link between collapsing soils and fragipan formation. The question to be considered is whether the observed features of fragipan horizons can be explained by the hydrocollapse mechanism of soil.
In the fifty years since Guy Smith established the term a considerable literature has developed and many aspects of fragipan formation have been investigated (Winters and Simonson, 1951; Grossman and Carlisle, 1969; Smalley and Davin, 1982; and Smeck and Ciolkosz, 1989). Grossman and Carlisle (1969) made a useful list of 18 points that have to be considered. Those which seem particularly significant are the requirements of: a net downward water movement; a high proportion of silt (particularly) and fine sand; clay ("probably the chief bonding agent") being distributed in an appropriate manner relative to the silt and sand grains; and the material having a gross polygonal structure. Another key factor is the more or less constant depth at which fragipans occur. Grossman and Carlisle require the depth to be sufficient to be subject to weak physical disturbance. It is difficult to be definite about the depth criterion, but it has been suggested that "the upper boundary of most fragipans is about 0.5 to 1.0m below the surface if the soil is not eroded..." (Soil Survey Staff, 1975).

Franzmeier et al (1989) studied fragipan formation in loess in Midwestern USA. They suggested that loess provides the simplest system in which to study the processes of soil formation and concluded that the following conditions are most conducive to fragipan formation:
1. A parent material consisting of loess, with the underlying material including palaeosol horizons that contain less than 30% clay.
2. A topography consisting of slopes that range from horizontal to approximately 12%.
3. A moist climate with winter surplus and summer deficit of precipitation relative to potential evapotranspiration.
4. Vegetation consisting of Dicot species, such as oak, that actively exclude silicon (Si) while adsorbing water.
5. A time for formation of at least several thousand years.
A series of tests with varying clay mineral content introduced randomly into the model loess specimens showed that collapse occurred over a range of approximately 5-40% kaolinite, and also that collapse could occur at low stress levels (see Figure 8.15). The most interesting results, in terms of the fragipan problem, are that samples prepared with no clay mineral content (i.e. a pure silt model loess) did not exhibit significant collapse (i.e. < 1%) when loaded and wetted (see Figure 7.15). When small amounts of clay mineral material were added significant collapse did occur (see Figure 8.15). As the clay mineral content was increased the collapse capacity increased up to a certain point and then decreased, the maximum collapse occurring at a kaolinite content in the range of approximately 15-25% depending on the level of applied stress as shown in Figure 8.15. As shown earlier, it was also found that the clay content required for maximum collapse depends also on the mineralogy of the clay (see Figure 8.11).

Figures 8.25 and 8.26 show the hydrocollapse behaviour at stress levels that are likely to occur in fragipan systems (i.e. between 5 and 20kPa) from the results of single and double oedometer tests. It can be seen that at stress levels of 5, 10, and 15 kPa the degree of hydrocollapse increased significantly when the clay mineral content increased from 6.7% to 13.4%. Figures 7.19 and 7.20 show the collapse situation in a totally ideal soil constructed from spherical particles and kaolinite clay powder. These figures illustrate how hydrocollapse can occur in a structure with an open packing that has been generated as a direct result of the correct amount of clay mineral materials modifying the contact points. Of particular note is the large amount of collapse that occurs in this ideal airfall system, and the low stresses at which it occurs.

A loess with a small (less than 30%) clay mineral content placed in a region which acquires a sufficiently moist climate regime would appear to be the ideal
situation for a fragipan to form in, as suggested by Franzmeier et al (1989). The mechanical and physical requirements that relate to the development of sub-surface hydroconsolidation apparently exist in many of the places where fragipan horizons are encountered. The Bryant hypothesis can be developed by showing that three conditions co-exist in the ideal loess which can lead to the formation of an horizon with a dense collapsed structure. These are a correct small clay content, the necessary effective stress via overburden pressure, and the required water via the moist climate region producing a net downflow of water in the soil, which combine together to produce the required effects.

The oedometer tests carried out on the model loesses show that at kaolinite contents over about 40% collapse is inhibited. They also show that a clay-free loess does not collapse. Adopting the soil mechanics terminology used by Jefferson and Smalley (1995 and 1997), the kaolinites (like illites) are 'inactive' clays and only modest contributors of clayey properties in soil engineering situations. The 'inactive' clays have a less dramatic effect than smectites, i.e. a greater quantity of these minerals is required to achieve maximum collapse. This seems to be the situation in most fragipans. Knox (1957) stated that "illite is responsible for a major part of the strength and rigidity of the Albia and Wurtsboro fragipans". In another paper in the same sequence Grossman and Cline (1957) observed that "the rather low median clay percentage emphasises the high efficiency attainable by silicate clay in the role of a bonding agent". Perhaps the low median clay content is more an indicator of the operation of a 'small clay' hydroconsolidation process in the formation of the fragipan. Recent studies showed that a bonding agent other than clay materials was not necessary for fragipan formation (Lamotte et al, 1997). The Bryant hydroconsolidation mechanism allows clay-bridging to contribute to strength, and adds an extra factor via the localised structural collapse.
Therefore, it can be concluded that loess is the obvious soil material for the fragipan to form in because it has a conveniently open structure. There are two soil properties are necessary to define the requirements of the Bryant Hypothesis:

1- Soil system capable of some structural collapse, and

2- The soil system must have a small clay mineral content.
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CHAPTER 9
CONCLUSIONS AND RECOMMENDATIONS
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CONCLUSIONS AND RECOMMENDATIONS FOR FURTHER RESEARCH

9.1 INTRODUCTION

This research has focused on the metastable structure and hydrocollapse behaviour of loess deposits. The hydrocollapse phenomenon of loess deposits has been reviewed, investigated and discussed. Special attention was paid to the influence of the compositional elements and structural factors of loess formations on this phenomenon, particularly the properties of the primary silt particles, the soil fabric and the nature of the interparticle bonds. Two different experimental approaches have been employed to investigate the collapse behaviour of metastable structures: testing of natural undisturbed (real) and artificial reconstituted (model) soil samples. A good understanding of the hydro-collapse characteristics of real and model soil samples would certainly help the study of collapsible soils and provide some light on the hydro-collapse phenomenon of metastable structure of loess deposits. The main conclusions derived from this research and recommendations for further research are described in the following sections.
9.2 CONCLUSIONS

9.2.1 Experimental Modelling of Real Loess Deposits
(Model Loess and Airfall Technique)

* Naturally deposited loess materials have successfully been simulated experimentally in the laboratory. The modelling process included the structural factors and the compositional elements of the real loess deposits. Structurally, soil specimens having a metastable structure have been created by using the airfall technique. The results of natural undisturbed and reconstituted loess specimens proved the validation of this technique. Compositionally, artificial loess materials have been prepared from different silt particles mixed with different clay minerals. Ideal silt particles (glass spheres) and pure silt from crushed sand (a simulation of the natural glacial grinding process) were used. These latter particles have geometrical properties similar to those for the typical natural loess materials. They are angular, blade shaped and have sharp edges. Specimens to examine the small clay-loess model behaviour were produced and tested. The results indicated that this model is valid.

* It was proved that the approach of using reconstituted specimens prepared from artificial loess material and employing the airfall technique can be used successfully to study the hydrocollapse phenomenon of loess deposits. This approach highlighted new paths for investigation and development, and allowed the whole range of parameters related to this problem to be thoroughly studied.
9.2.2 Hydrocollapse Behaviour of Model Loess

The hydro-collapse behaviour of the reconstituted specimens prepared from natural loess material seems to be qualitatively similar to that of the undisturbed specimens obtained from the same material. Both types of specimen undergo similar structural hydro-collapse due to the effect of loading and subsequent wetting. All specimens that were subjected to wetting at different stress levels showed a unique post-wetted relationship between void ratio and applied pressure.

The hydro-collapse values for the reformed specimens are higher than those for naturally deposited ones in the range of low applied stress (typically less than 100 kpa) and are lower in the case of a high level of applied stress. However, both materials have similar degrees of hydrocollapse according to the standard definition (the degree of hydrocollapse at 200 kPa).

9.2.3 Interparticle Bonding and Hydrocollapse

Testing of pure silt from different materials provided some insight into the hydro-collapse phenomenon of loess deposits, particularly the effect of soil structure parameters such as particle packing and interparticle bonding on the behaviour of collapsible material. Very little hydro-collapse was observed, although the initial void ratios of the specimens tested were relatively high ($e_0 \geq 0.9$). Whatever the level of applied stress the magnitude of hydro-collapse is nearly constant. In addition, the compressibility of pure silt from crushed sand is similar to that of pure (i.e. washed) silt from Malan loess and that of silt-sized glass balls.
The general trend of the e-logp curves prior to wetting is similar to those curves after wetting. It therefore is apparent that there is no effect of wetting on the compressibility and hydro-collapse behaviour of pure silt.

* The importance of the bonding materials, particularly the clay minerals, on the formation of metastable structures and the behaviour of collapsible soil has been proved experimentally. The presence of clay minerals and clay-sized particles at the points of contact is apparently essential for the establishment of a metastable structure and thus for hydrocollapse to occur.

* The hydro-collapse of silt-clay mixtures increases as the clay/silt (C/M) ratio increases up to a certain value, at which a maximum amount of hydro-collapse will be obtained. If the clay content increases further, the degree of hydro-collapse reduces until the material takes on the characteristic behaviour of a clay. The effect of C/M ratio on the behaviour of silt-clay mixtures depends on the type of clay mineral. For each type there is an optimum clay/silt (C/M) ratio at which a maximum value of hydro-collapse is obtained.

* The small clay-soil model of collapsible structure is valid. The clay content exercises a direct control on hydrocollapse behaviour of loess material. For any particular clay mineral there is an optimum clay content at which hydrocollapse is most apparent. For example in the case of kaolinite, at stress levels lower than 400 kPa maximum hydrocollapse occurred with 15-25% clay mineral content. For bentonite, the range and amounts are much reduced: at stress levels between 50-400 kPa the optimum clay mineral content was 6-10%. Generally the clay mineral content promoting maximum collapse is
influenced by the level of applied stress, the clay content rising with stress level.

* Not only are the type and amount of bonding materials important for producing collapsible soil structure, but the position and distribution of these materials inside the soil structure are also very important. Different methods of mixing clay minerals with silt particles give different collapse results. Specimens of wet-mixing always have higher hydro-collapse values than those of dry-mixing.

9.2.4 Geometrical Characteristics of Silt Particles and Hydrocollapse

* An ideal model of loess material can be constructed using glass spheres as the major silt-sized constituent. The soil system prepared by mixing ballotini glass balls with kaolinite has been shown to collapse in the same way as the true silt/clay systems. The results from the ballotini experiments show that the geometrical characteristics of the primary solid particles are very effective variables in the hydrocollapse process. A large degree of interlocking takes place with real silt particles having high angularity and sharp edges, whereas the roundness and smoothness of ballotini leads to greater degree of hydrocollapse.

* It is very important to establish that ballotini systems give results similar to those of true silt particles since this offers an opportunity to make the connection to mathematical structural models, which tend to use spherical particles. Therefore, a very good link between theoretical and experimental research can be established. This allows for integrated
research to be carried out to study the hydrocollapse problem of loess deposits.

9.2.5 Hydrocollapse Behaviour of Libyan Loess

- A hydro-collapse magnitude of about 13% for a wetting pressure of 200 kPa was obtained for the Tripoli loess. Based on the identification of the severity of collapsibility given by Jennings and Knight (1975), the Tripoli loess is considered to pose severe collapse problems, having a high susceptibility to structural hydro-collapse in the same manner as other better known loess deposits.

- The amount of hydro-collapse is dependent on the magnitude of vertical stress acting on the soil specimens at the time of wetting. The hydro-collapse increases as the applied stress increases up to a certain level, at which the maximum collapse is obtained. For the loess samples tested herein the maximum hydro-collapse value is about 22%.

- The hydro-collapse values measured in the single-oedometer test at different stress levels appear to be in agreement with the values predicted from the double-oedometer test results.

- A unique post-wetted void ratio-applied stress relationship was observed for Tripoli loess. Flooding of specimens at any level of applied stress caused the material to collapse to a certain value of void ratio prescribed by the unique post-wetted void ratio-applied stress curve.
9.2.6 Hydrocollapse Behaviour of Chinese Loess

The hydro-collapse values vary, depending on the stress level acting on the specimens at the time of wetting. For a wetting pressure of 200 kPa, a hydrocollapse magnitude of about 8% was obtained. The Malan loess can be considered to pose collapse problems according to the classification of Jennings and Knight (1975).

Similar to the Tripoli loess and other types of loess reported in the literature, Malan loess also has a unique post-wetted relationship between void ratio and applied stress. This observation indicates that the loess material anywhere in the world has a unique post-wetting curve and always collapses to the final voids ratio prescribed by this curve, independent of the applied stress level.

The hydro-collapse values of Malan loess vary as the applied stress increases, the rate of hydro-collapse being large at low levels of applied stress but reducing considerably at higher levels. The general trend of hydro-collapse-applied stress relationships for both loess materials from Libya and from China is similar but the hydro-collapse values at all stress levels for Chinese loess are lower than those for Libyan loess.

9.2.7 Index Properties and Geometrical Characteristics of Loess Materials

The specific gravity, plasticity properties (Atterberg Limits) and the grain-size characteristics of Libyan loess were found to be very similar to those of loess deposits from many parts of the world.
Based on the plasticity and particle-size test results, and according to the definition of the loess types given by Holtz and Gibbs (1951) and Gibbs and Holland (1960), the Libyan loess material can be classified into two types: silty loess in the Tripoli region and clayey loess in the Ghat area.

The average value of the specific gravity of Mal an loess is 2.72 which is slightly higher than the average value (2.70) for those from many parts of the world. The Atterberg Limits of Malan loess are similar to those of silty loess as defined by Gibbs and Holland (1960). The grain-size characteristics of Malan loess were found to be typical also. The average dry density of Malan loess is 1.35 Mg/m³ which is close to the value (1.40 Mg/m³) of typical loess.

The pure silt derived from crushed sand is a uniformly-graded coarse silt, with more than 75% of the particles falling into the size range 10-50 μm and a silt mode size of approximately 25 μm. The grain-size distribution is thus typical of the silt fraction of typical loess as reported in the literature.

The SEM pictures demonstrate that the silt particles of Libyan and Chinese loess materials have similar shape factors, most being blade shaped and angular or semi-angular. The SEM pictures also show the silt particles derived from crushed sand to have similar geometrical characteristics.
9.3 RECOMMENDATIONS FOR FURTHER RESEARCH

The work done in this research provides a step towards the goal of understanding the problem of loess hydrocollapse, and it is a good basis for further investigation in this area.

9.3.1 An Open Structure and Computer Modelling

Results of preliminary investigations illustrated that a highly porous unstable structure of loess deposits can be constructed geometrically by utilizing a simple Monte Carlo method (see Assallay et al 1997). A range of pore sizes and sub-structural types can be generated and collapse processes can be identified. This geometrical construction suggests that the time-zero aeolian deposition event forms a very open packing, creating an unstable structure containing giant pore spaces. These spaces give the unstable structure a very high void ratio ($e \geq 1.5$). Similar results ($e \geq 1.4$) have been obtained experimentally in modelling the time-zero unstable structure using real loess particles deposited by the airfall technique. In this case large pores were produced which collapse almost immediately. It is proposed that the giant pores collapse very early in the history of the loess deposit (while deposition is still proceeding) to transform the unstable structure into a metastable one which can remain in place until the right combination of loading and wetting produces further hydro-collapse and subsidence. The results also show that the typically observed vertical pores may be formed. This is a very interesting observation which gives an indication that the characteristic pore structure of loess deposits, which is dominated by vertical pores, might be due to the airfall mode of sedimentation and geometrical (size and shape) characteristics of silt particles r
rather than the effect of vegetation, as is widely believed. One of the most important findings deduced from this preliminary study is that the size, shape and orientation of the solid particles are fundamental factors that govern the main features of the metastable structure of loess soils.

In order to investigate the effect of these factors on the main aspects of the collapsible structure, in particular the shape, size and distribution of pores, the sub-structural types and the nature of the interparticle contacts, a computer program for soil structure simulation is important to make the calculations easier and more efficient. Therefore developing a computer program for modelling unstable and metastable structures of loess deposits will be one of the most important targets for further research.

9.3.2 Experimental Modelling of an Open Metastable Structure

It was believed that the correct way to study the phenomenon of hydrocollapse was to start by using ideal materials such as ballotini glass balls and simple methods of specimen preparation and testing. Hence the dry particles-airfall technique was used. The wetting-drying cycle method has been attempted by Russian investigators and the problems they encountered were explained in Chapter 6. It seems that this method is similar to the natural process, particularly in the semi-arid environment. Therefore, it was decided that the next stage for studying this problem was to use the wetting-drying method for specimen preparation and to find ways to solve the associated problems and testing methods suggested in Chapter 6.4.3.
9.3.3 Space and Solid Properties of Loess Structure and Hydrocollapse

* In this investigation the voids ratio was generally taken as a constant parameter of approximately 0.9. The effect of this factor on the collapse degree of loess deposits was investigated by several researchers, who found that the magnitude of collapse increases as the voids ratio decreases. Most of the previous research, if not all, was carried out on natural undisturbed loess materials and was restricted to a certain type of soil material. In addition, the study of the influence of the amount and type of clay minerals and the shape factor of the silt particles was not possible. By using the approach developed in this research it is now possible to evaluate the role of all variables. The behaviour of hydrocollapse in general, and particularly the optimum clay content (clay content of maximum hydrocollapse) for low and high plasticity clay was determined at a certain level of voids ratio. The question is: Will these results be the same if the value of voids ratio is increased or decreased? This would be a good starting point for further work.

* As stated earlier, it was proved that the size and grain-size distribution of the silt particles have a significant effect on the loess hydrocollapse, particularly the soil fraction finer than 20μm and the filling fraction (2-10μm) as defined by Browzin (1985). Few tests have been carried out in this research and further investigation is required.

* The effect of clay minerals on the hydrocollapse behaviour of loess deposits was studied in this investigation using kaolinite (low plasticity clay) and bentonite (very high plasticity clay). To complete the basic study of this area, further series of tests are required for loess specimens.
containing illite (medium plasticity clay) and very fine (≤ 2μm) quartz particles of clay size (none plastic material). Based on the results of this research, it can be predicted that the degree of hydrocollapse for loess material having illite as a clay mineral would fall in between the range of values between those for bentonite loess and kaolinite loess, and the optimum clay content would be in the range of 10-15%, depending on the applied stress level.

* It is believed that there is a common misunderstanding by several researchers that the calcium carbonate has a dominant effect on the collapse behaviour of loess deposits, and some of them reported that the calcium carbonate was essential for soil collapse to occur (Tahir, 1970). This misunderstanding came from the fact that the calcium carbonate is often accompanied by the clay material in loess. The results of a few tests conducted by the author illustrated that calcium carbonate has no significant effect on the hydrocollapse phenomenon of loess deposits, and the loess hydrocollapse is mainly due to the existence of clay minerals in the soil. This point of research has not been investigated thoroughly and more investigation is necessary to clarify this argument.
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