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CONTENTS

Chapter 32

Collapsible soils

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Collapsible soils present significant geotechnical and structural engineering challenges the world over. They can be found in many forms – either naturally occurring or formed through human activities. However, an essential prerequisite is that an open metastable structure develops through various bonding mechanisms. Bonds can be generated via capillary forces (suctions) and/or through cementing materials such as clay or salts. Collapse occurs when net stresses (via loading or saturation) exceed the yield strength of these bonding materials. Collapse is most commonly triggered by inundation through a range of different water sources, although the impact varies with different sources yielding different amounts of collapse. To engineer in and mitigate the effects of collapsible soils, it is essential to recognise their existence, which may not be easy, and to gather vital geologic and geomorphologic information. Collapsibility should be confirmed through direct response to wetting/loading tests using laboratory and field methods. The key challenge faced with collapsible soils is the spatial extent and the degree of wetting that will take place. Care is needed to ensure that appropriate and realistic assessments are undertaken. Ultimately, if treated using one of a suite of the possible improvement techniques available, then the potential for collapse can be eliminated effectively.

32.1 Introduction

Collapsible soils are extremely common and can be formed naturally owing to various geologic and geomorphologic processes, or be the result of human activity. These processes, although different in nature, allow the development of an open metastable structure: the essential prerequisite to the formation of a collapsible deposit (Dudley, 1970). Volume changes that occur are more sudden than those experienced through consolidation processes and typically occur in material that is non-plastic or of very low plasticity, confined and initially dry (Houston *et al.*, 2001).

Although collapsible soils are found only in arid regions, arid environments tend to favour their formation. Naturally occurring collapsible soils are formed typically from debris flow (e.g. alluvial fan materials), as wind-blown sediments (e.g. loess), as cemented high salt content metastable soils (e.g. sabkha, see Chapter 29 *Arid soils*), and as tropical residual soil (see Chapter 30 *Tropical soils*). In addition, collapsible soils can be formed artificially through poor compaction control or where compaction is dry of optimum (e.g. non-engineered fill, see Chapter 34 *Non-engineered fills*), or as waste materials (e.g. fly ash beds, Madhyannapu *et al.*, 2006). However formed, common to almost all collapsible soils are both low densities and a relatively stiff and strong state when dry (as illustrated in **Figure 32.1**). Notable exceptions to this are post-glacial sensitive 'quick' clay soils found mainly in Canada, Alaska and Scandinavia, but these are arguably a special case, being geographically centred and in a saturated collapsible state. An additional exception is saturated slide material such as saturated sands that has been allowed to flow on slopes that have experienced liquefaction (Nieuwenhuis and de Groot, 1995).

Collapsible soils can be defined as soils in which the major structural units are initially arranged in an open metastable packing through a suite of different bonding mechanisms. If the soil is loaded beyond the yield strength of the bonding material, collapse will occur – this results in a rearrangement of particles to form a denser stable configuration (see **Figure 32.2**). Thus the collapse itself is controlled both microscopically and macroscopically. An appreciation of both these elements is essential if the true nature of collapse, and therefore its effective remediation, are to be fully understood.

Collapse is often triggered by a combination of increased stress (load) and the addition of water leading to increased

Figure 32.1 Example of loess in its pre-collapsed state exhibiting its relatively stiff soft rock state when dry, allowing the formation of man-made caves, Slovakia

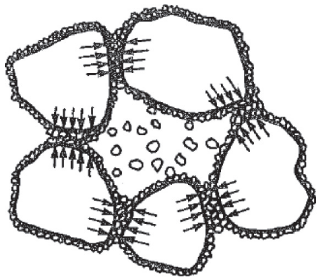

Figure 32.2 An illustration of collapse through inundation
Reproduced from Houston *et al.* (1988) with kind permission from ASCE

degrees of saturation. However, collapse is most often triggered by an increase in water content. There are a variety of sources of water ingress that can initiate collapse, many associated with urban environments. These include landscape irrigation, broken water or sewer pipes, run-off or poor drainage control, groundwater recharge, or water content changes through capillary rise.

An example of the effects of urbanisation and associated landscaping was illustrated by a commercial building in New Mexico. After winning the city's most beautiful lawn and landscaping award, achieved through heavy watering, the building suffered US$ 500 000 foundation damage owing to soil collapse (Houston *et al.*, 2001). The result of infiltration with water results in a relatively sudden volume compression, and is often associated with loss of strength. This can clearly have important geotechnical consequences, including loss of serviceability resulting in expensive remediation or, on occasion, complete failure. This was illustrated by the structural failures caused by damage to foundations on collapsible soils in Egypt (Sakr *et al.*, 2008), and the collapse problems caused by hydrogeological changes associated with the construction of a dam in Brazil (Vilar and Rodrigues, 2011).

Other reported problems have occurred in embankment bases (Thorel *et al.*, 2011), dam embankments (Peterson and Iverson, 1953), road embankments (Knight and Dehlen, 1963) and fill (Charles and Watts, 2001). It can be particularly problematical when collapse causes damage to historic buildings (Herle *et al.*, 2009), e.g. the cracking that developed in the wall of a 15th century pagoda in Lanzhou, China, after the introduction of an irrigation scheme (see **Figure 32.3**).

From this it is clear that, given the correct depositional environment, collapse has the potential to occur in any soil. In fact, given the correct stress environment, a collapsible soil may exhibit expansive behaviour (Derbyshire and Mellors, 1988; Barksdale and Blight, 1997; see also section 32.3.3 below). It is therefore necessary to understand the process of collapse if problems associated with collapsible soils are to be avoided or mitigated. As collapse is for the most part triggered by increases in water content, problems are often

Figure 32.3 Collapse-induced cracking in a 15th century pagoda
Reproduced from Billard *et al.* (2000); John Wiley & Sons, Inc.

encountered in urban and built environments and so the impact occurs where it has potential to cause the most harm. This can be particularly problematical as in many parts of the world collapsible soils exist in areas of high seismic actively (Houston *et al.*, 2003). The consequences can be catastrophic, e.g. the Haiyuan earthquake in which induced loess landslides in 1920 killed over 200 000 people (Derbyshire *et al.*, 2000; Zhang and Wang, 2007). Moreover, the potential for collapse will remain until full collapse has been induced either naturally through flooding and/or loading (earthquakes), or artificially by human activity. Human activity can be either accidental via poor drainage control, or deliberate through ground improvement, e.g. dynamic compaction.

32.2 Where are collapsible soils found?

Rogers (1995), Lin (1995), Bell and de Bruyn (1997) and Houston *et al*. (2001, 2003) discuss in detail the various forms of collapsible soils found worldwide. Collapse occurs because soil has certain inherent properties. Typical features that are found with most collapsible soils include:

- an open metastable structure;
- a high voids ratio and low dry density;
- a high porosity;
- a geologically young or recently altered deposit;
- a deposit of high sensitivity;
- a soil with inherent low interparticle bond strength.

Thus many soils can and will exhibit collapsible behaviour. This is illustrated in **Figure 32.4** along with the various formation processes that can yield a collapsible soil.

Collapsible soils include naturally occurring soil formations, such as tropical residual soils, loess and quick clays, and anthropogenic soils such as uncompacted or poorly compacted fill. However, any compacted soil can exhibit collapse if the confining pressure is high enough (see section 32.3.3 below). Further details of tropical and anthropogenic soil collapse can be found in Chapters 30 *Tropical soils* and 34 *Non-engineered fills* respectively. Additional information on the collapse behaviour of residual soils is also provided by Barksdale and Blight (1997) and Roa and Revansiddappa (2002). Examples include decomposed gneiss (Feda, 1966), decomposed granites (Brink and Kanty, 1961) and granitic gneiss (Pereira and Fredlund, 2000).

Other soils exhibiting collapse include collapsible gravels (Rollins *et al.*, 1994), granitic sands in South Africa (Jennings and Knight, 1975), collapsible sands of the southern African east coast (Rust *et al.*, 2005) and quick clays (Bell, 2000). Loess is probably the most commonly encountered naturally occurring collapsible soil, covering some 10% of the worlds landmass (Jefferson *et al.*, 2001). Many of the practices and engineering approaches used to deal with collapsible soils stem from the treatment of loess soils.

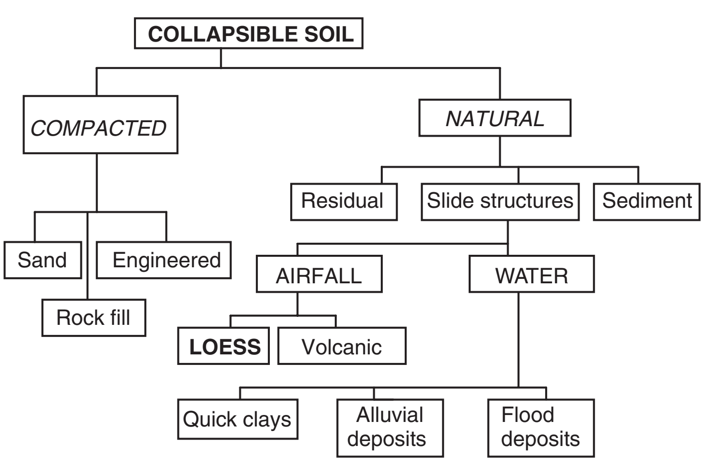

Figure 32.4 Classification of collapsible soils (Rogers, 1995)

32.2.1 Wind-blown soils – collapsible loess soils

Loess consists essentially of silt-sized (typically 20–30 μm) primary quartz particles that form as a result of high energy earth-surface processes such as glacial grinding or cold climate weathering (Rogers *et al.*, 1994). These particles are transported from the source by rivers. Subsequent flooding by these rivers allows the quartz silt particles to be deposited on flood plains (Smalley *et al.*, 2007). On drying out, these particles are detached and transported by the prevailing winds until deposition leeward at distances ranging from tens to hundreds of kilometres. Cementing materials are often added after deposition or dissolved and re-precipitated at particle contacts (Houston *et al.*, 2001). This process has resulted in the almost continuous deposit draped over the landscape from the North China plain to southeast England (where it is generally referred to as 'brickearth') – see **Figure 32.5**.

It is possible to isolate five major loess regions worldwide: North America, South America, Europe (including western Russia), central Asia and China (Smalley *et al.*, 2007). These loess regions often underlie highly populated areas and major infrastructure links, making them vulnerable to soil collapse. The areas of most widespread concern are concentrated in eastern Europe, Russia and, to a growing extent, China (see Derbyshire *et al.*, 1995, 2000), although potentially serious problems of collapse exist wherever loess is found.

Figure 32.5 indicates the approximate distribution of loess/brickearth greater than 0.3 m in thickness in the UK. Significant thicknesses (greater than 1 m) are restricted to north and east Kent (see Fookes and Best, 1969; Derbyshire and Mellors, 1988), south Essex (see Northmore *et al.*, 1996) and the Sussex coastal plains. In Essex, deposits of up to 8 m have been found, although thicknesses of 4 m or so are

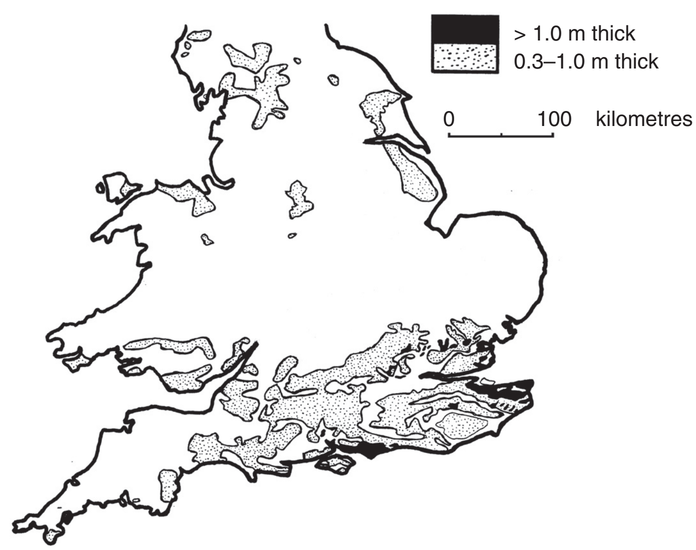

Figure 32.5 Distribution of 'brickearth' (loess) deposits in southern England and Wales
Modified from Catt (1985); Allen & Unwin (as reproduced in Jefferson *et al.*, 2001)

more typical (Northmore *et al.*, 1996). However, substantial deposits can reach tens and even hundreds of metres thickness worldwide (see Smalley *et al.*, 2007 for further details).

Regional trends in loess occur in the type of deposit found around the world and these can be determined through textural and mineralogical distinctions. However, there is generally a progressive decrease in modal size with distance from its source material – suggesting that sorting of loess material occurs with wind direction during deposition (Catt, 1985).

Originally, loessic deposits would have been more extensive but they have been removed by post-depositional erosion, colluviation, deforestation/agriculture and resource stripping activities (Catt, 1977). As a result, modern loess deposits are often only found overlying relatively permeable strata.

32.2.2 Other collapsible deposits

As discussed in sections 32.2 and 32.3, many soils exhibit collapsibility. Apart from those already highlighted above, a number of water-sediment deposits exhibit collapse potential. Two main groups exist: alluvial deposits and quick clays. Alluvial deposits reported to cause collapse problems include alluvial fans, alluvial flood plain deposits and mud flow deposits, with several case histories of collapse problems in alluvial fan deposits being provided by Rollins *et al.* (1994).

By comparison, saturated quick clay has become unstable owing to its post-glacial depositional environments, allowing an open structure to form via slow sedimentation under shallow marine conditions (Rogers, 1995; Locat, 1995). The resulting open fabrics are maintained by small amounts of carbonate cementation of clay minerals, with salt leaching often having occurred. Quick clays typically have liquidity indices greater than 1, with liquid limits often less than 40% (Bell, 2000). When disturbed, the particles in quick clays are remoulded into closely packed configurations. As the water content remains unchanged during the collapse process, quick clays become oversaturated and may flow as a viscous liquid. This can have devastating effects as witnessed by the 1978 landslide in Rissa, Norway, where, after the initial landslide, a series of minor slides developed which eventually covered an area of 3 300 000 m^2; or more recently by the large landslide in Leda clay in Ontario, Canada in 1993 (see Bell (2000) for further discussion).

32.3 What controls collapsible behaviour?

Collapsibility occurs owing to the various geomorphologic and geologic processes as well as human activities during soil formation. The key to this is understanding the nature and processes that take place during provenance (P), transportation (T) and deposition (D) of the soil particles: it is the PTD sequence of a deposit that produces an open metastable collapsible structure of relatively high void ratio (Sun, 2002; Jefferson *et al.*, 2003b; Smalley *et al.*, 2007). The PTD approach conceptualises the processes of soil genesis, subsequent transportation and ultimate disposition, so elucidating how these influence the engineering behaviour of the final deposit.

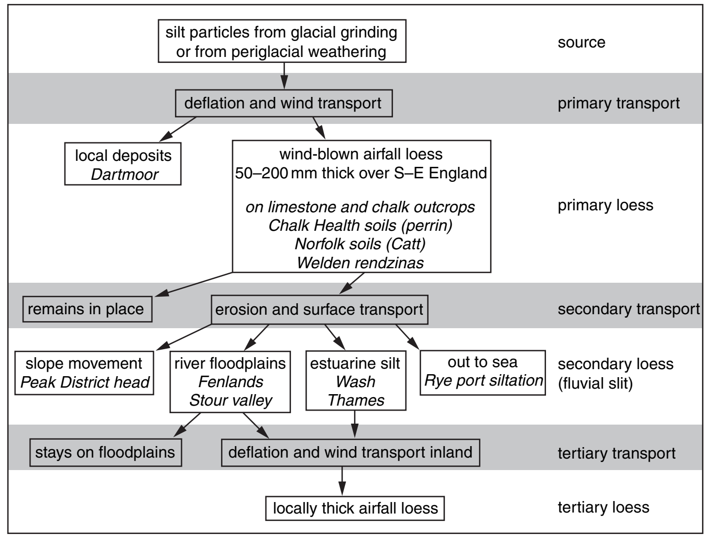

Figure 32.6 A PTD geomorphological model of loess in southern Britain. Note: Norfolk soils (Catt *et al.*, 1971); Chalk Heath soils (Perrin, 1956)
Reproduced from Jefferson *et al.* (2003b); East Midlands Geological Society

However, some deposits have been through secondary or tertiary PTD sequences, often reducing or even eliminating collapse potential. An example to illustrate this is given in **Figure 32.6**, based on a PTD model of loess found in southern Britain, with further details discussed by Pye and Sherwin (1999) and Derbyshire and Meng (2005).

Wind-blown collapsible soils such as loess often consist of different zones of variable collapsibility, both laterally and with depth, as a result of their geomorphology. The prevailing wind direction generates zones of material laterally changing in nature from sandy loess through to clayey loess deposits; see **Figure 32.7**. It should be noted that the terms clayey, silty and sandy loess are defined by particle size analysis (see Holtz and Gibbs, 1952). In turn, the degree of saturation and potential collapsibility often follows similar trends. For example, the loess in Bulgaria (**Figure 32.7**) exhibits an increase in saturation as the modal size reduces (Jefferson *et al.*, 2002). The thicknesses of loess formed often follow the same pattern – with the greatest thicknesses occurring nearest to the source material.

In addition, deposits such as loess have been formed over many thousands of years and during this time climactic conditions have varied considerably. As a result, the loess soil sequence has alternating layers of loess (formed during cold periods) and clay-rich palaeosols (formed during warm periods) – see **Figure 32.8** for illustration.

The alternating nature of loess formation significantly influences the engineering behaviour and ultimately the nature of collapse, and the location (depth) where collapse occurs. This will dictate the nature of the infiltration patterns of water into the soil and as a result can yield collapse in unexpected locations. Moreover, this can influence the

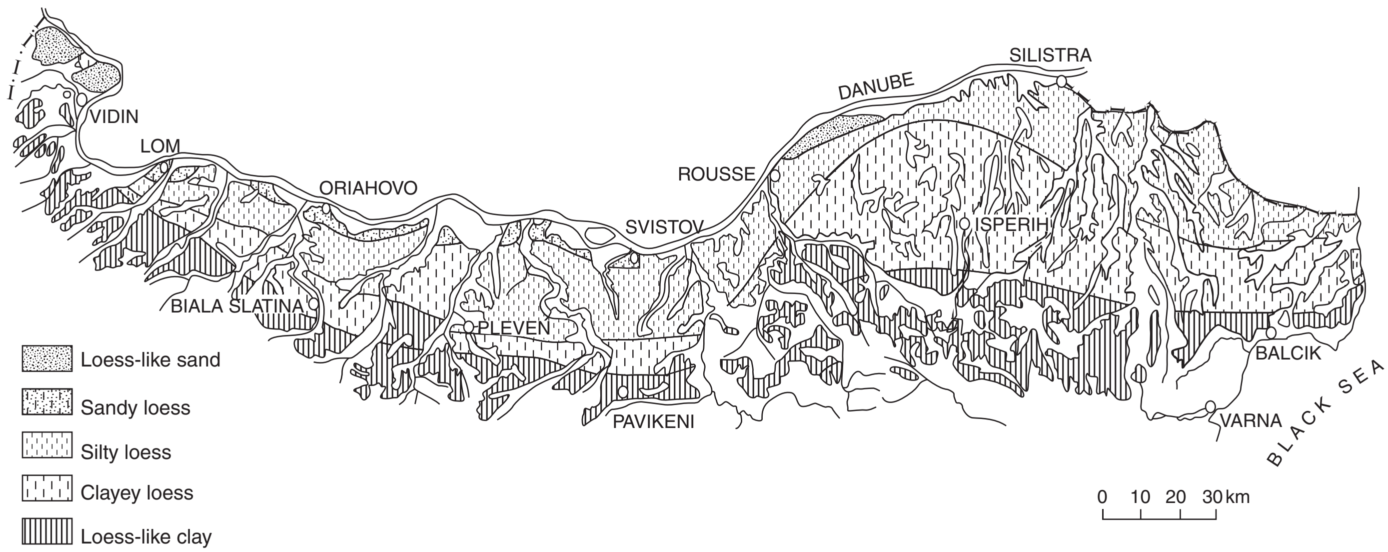

Figure 32.7 Distribution of collapsible loess soils from Danube flood plain in Bulgaria
Reproduced from Minkov (1968); Marin Drinov Academic Publishing House/BAS Press

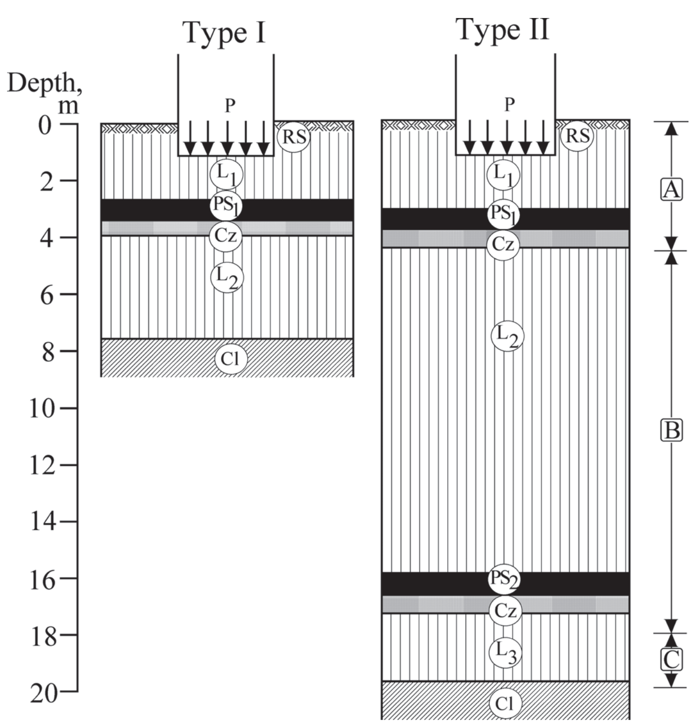

Figure 32.8 Types of collapsible loess soils
Reproduced from Jefferson *et al.* (2005), with permission from Elsevier

effectiveness of any ground improvement approach used to remove collapsibility.

As a result of the various geomorphologic processes involved in the formation of loess, these deposits typically have three zones of relative collapsibility:

- Zone 1: a zone at depth of collapsed material due to overburden pressure.
- Zone 2: a collapsible zone.
- Zone 3: a surface crust (which requires additional load to cause collapse). This has led countries with extensive loess deposits (including Eastern Europe and the former Soviet Union) to develop classification schemes related to the collapse of loaded foundations (see **Figure 32.8**). Two types of collapsibility occur in such schemes:

Type I mainly loaded collapsibility (collapse deformation under overburden pressure of $\delta_n < 5$ cm); and
Type II mainly unloaded collapsibility ($\delta_n > 5$ cm, where δ_n is the collapse deformation).

Type I loess is usually of small thickness (shown in **Figure 32.8**) and contains one or two palaeosols (PS), together with an associated carbonate zone (Cz). Collapse occurs after the foundation stress exceeds a certain critical stress, which can be determined by laboratory tests or tests in the field (see section 20.4 below).

Type II loess has greater thickness – up to 50 m or more. **Figure 32.8** shows a typical case of a Danubian terrace with a deposition of loess of about 20 m. In this case, three loess horizons (L_1, L_2 and L_3) are separated by two palaeosols (PS_1 and PS_2). In the Type II loess, three zones can be distinguished:

(i) upper zone A – no unloaded collapsibility but with potential loaded collapsibility;
(ii) middle zone B – unloaded collapsibility; and
(iii) lower zone C – uncollapsible (or collapsed).

Loess in zone C has previously collapsed under overburden pressure. Loess in zone A has had no unloaded collapsibility, since here the overburden pressure is small (i.e. it will not collapse under self-weight). However, it can be collapsible under additional load. In zone B, unloaded collapsibility occurs – it often contains thicker loess horizons with lower density and higher porosity, n (i.e. $n > 50\%$), and it has a higher silt content than in zone C.

Further details of the different geomorphologic processes that generate collapsibility are given in Chapters 30 *Tropical soils* and 29 *Arid soils*, and an excellent treatment of the broader subject is provided by Fookes *et al.* (2005).

32.3.1 Bonding mechanisms and fabric

For collapse to occur, an open structure with relatively large voids must exist together with a source of strength to hold soil particles in position, resisting shearing forces associated with the current stress environment. To achieve this, bonding between particles or grains of sufficient strength must occur which, when weakened by the addition of water and/or an additional load, allows particles to slide over one another – resulting in collapse. There are generally considered to be three main bonding mechanisms present in collapsible soils (Barden *et al.*, 1973; Clemence and Finbarr, 1981; Rogers, 1995), namely:

(i) capillary or matric suction forces (see **Figure 32.9(a)**);

(ii) clay and silt particles at coarser particle contacts (see **Figure 32.9(b-d)**);

(iii) cementing agents, such as carbonates or oxides (see **Figure 32.9(e)**).

The fabric of a collapsible soil takes the form of a loose skeleton built of grains (generally quartz in the case of loess) and micro-aggregations (in the case of loess assemblages of clay or clay and silty-clay particles, see **Figure 32.10**). In some soils, such as loess, carbonate diagenesis may strengthen the meniscus clay bridges between silt grains and further influence collapse potential. Recent observations by Milodowski *et al.* (2012) suggest that there are three variants of the clay bridges: (1) simple clay meniscus films, (2) clay films developed on a scaffold of an earlier meniscus of fibrous calcite and (3) clay films permeated or encrusted by microcrystalline calcite and/or dolomite. Hence the strength can vary both laterally and vertically over relatively short distances. This, together with the durability of the cementing material, needs to be taken into account for engineering purposes.

It should be noted that the microfabric of loess soils that have experienced reworking (cf. **Figure 32.6**) are often anisotropic and as a result exhibit much reduced collapsibility (Pye and Sherwin, 1999). In addition, as with other collapsible soils, domains of pelletised material can form and be arranged in a loose framework. Further details are given in Klukanova and Frankovska (1995), Jefferson *et al*. (2003b) and Milodowski *et al*. (2012). Derbyshire and Meng (2005) provide further discussion of fabrics associated with loess soils in China.

Other collapsible soils, such as quick clay, have essentially the same open metastable fabric, but are generated under different geomorphologic conditions. Further details are provided by Locat (1995), Bentley and Roberts (1995) and Bell (2000); see also section 32.2.2 above. Further details related to other collapsible deposits are given elsewhere in this manual (see Chapters 34 *Non-engineered fills* and 30 *Tropical soils*).

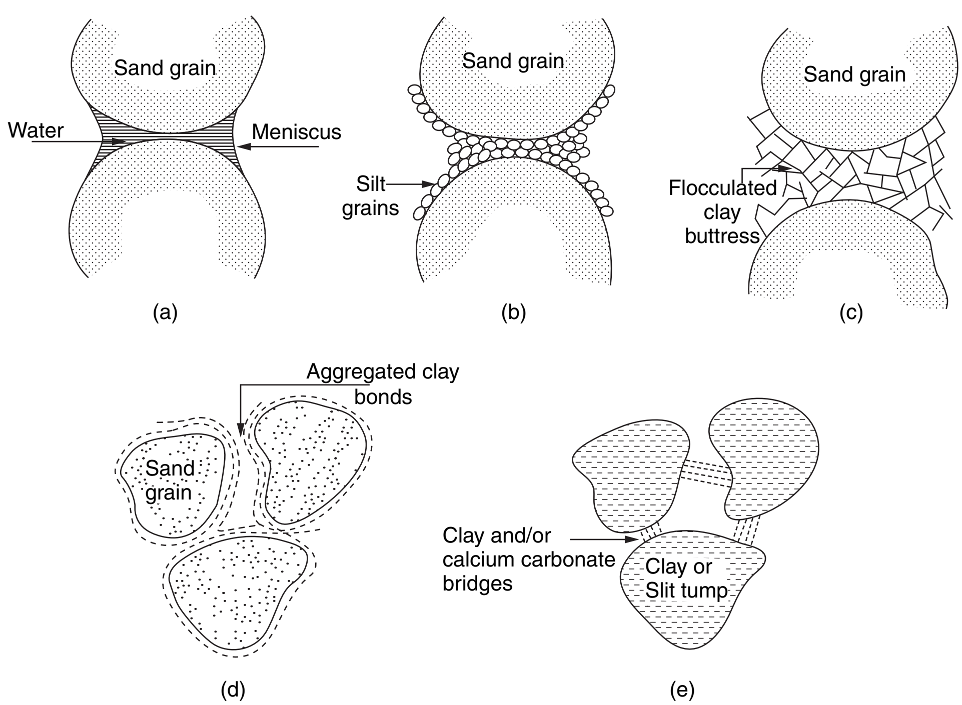

Figure 32.9 Typical bonding arrangement formed in collapsible soils
Reproduced from Popescu (1986), with permission from Elsevier

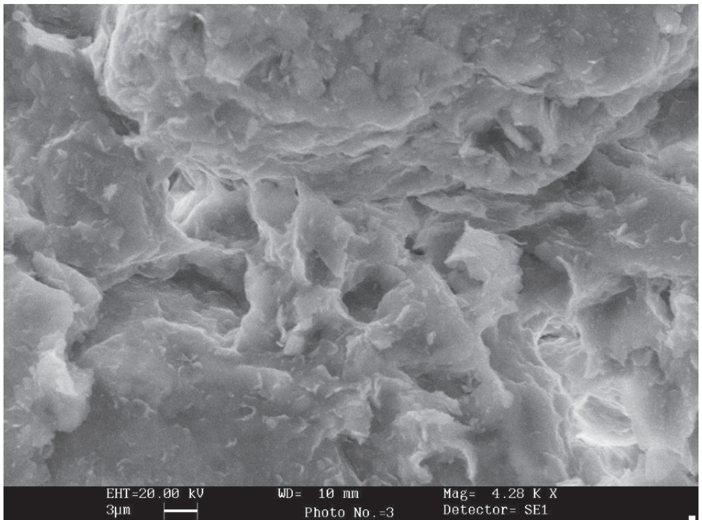

Figure 32.10 Scanning electron micrograph of bonded quartz particles in Brickearth (loess) from Ockley brickworks, Sittingbourne, Kent

Reproduced from Jefferson *et al.* (2001); all rights reserved

It should be noted that although fabric is widely recognised as important in explaining collapse behaviour, it lacks a simple quantitative descriptor (Alonso, 1993 as cited by Pereira and Fredlund, 2000).

32.3.2 Mechanisms of collapse

Collapse in cemented soils typically involves the destruction of all three bonding types. In contrast, collapse, in uncemented dry soils is solely due to the destruction of capillary forces. The strength derived from suction and cementing can be characterised in similar ways. However, on wetting suction will be lost, whereas chemical bonding is likely to be less affected by a change in suction. However, salt and clay bonds that occur at particle contacts will tend to be removed or weakened after inundation and hence collapse occurs.

Petrographic evidence indicates that collapse in cemented loess soils occurs in three stages after inundation (Klukanova and Frankovska, 1995; Milodowski *et al.*, 2012):

Stage 1 Dispersion and disruption of clay bridges or buttresses between loosely packed silt grains, leading to initial rapid collapse of inter-ped matrix.

Stage 2 Load taken up via contact between adjacent compact silt peds, which rearrange into a closer packing.

Stage 3 With increased loading, progressive deformation and shearing of peds occur, resulting in further collapse as peds disaggregate and silt particles collapse into now-unsupported inter-ped areas.

Similar observations concerning the collapse process in compacted soils have been made by Pereira and Fredlund (2000), who suggest:

Phase 1 (Pre-collapse): High matric suction generates a metastable structure that suffers small volume deformations with decreased suction. No particle slippage occurs and structure remains intact.

Phase 2 (Collapse): Intermediate matric suction and a significant decrease in volume occur, altering the structure through bond breakage.

Phase 3 (Post-collapse): Saturation is approached and no further decrease in volume (or reductions in matric suction) will occur.

Pereira and Fredlund (2000) further observed that as the net confining pressure increases, the wetting-induced collapse becomes greater and the matric suction associated with phases 1 and 3 will be higher.

During the collapse process, the nature of pores also changes. Studies on loess have shown that most of the macropores (100–500 μm) are typically destroyed, leaving smaller intergrain and interaggregate pores (8–100 μm) (Osopov and Sokolov, 1995). Similar observations with respect to collapse mechanisms were made by Klukanova and Frankovska (1995) and Feda (1995), and more recently on residual soils by Roa and Revansiddappa (2002).

Cerato *et al.* (2009) observed that compacted soils with a greater number of smaller clods showed greater collapse, with collapse largely dependent on the interaggregate and intra-aggregate pore distribution. Moreover, when clods are in a drier than optimum water content, stronger state, higher yield stresses result and the overall soil structure is less prone to collapse. In some fill materials the parent material may also lose some strength, or aggregates within the fill may soften as its water content increases – resulting in a possible collapse (Lawton *et al.*, 1992; Charles and Watts, 2001; Charles and Skinner, 2001).

For uncemented soil, collapse is related to the destruction of capillary (matric suction) forces, with water infiltration producing wetting fronts. The volume change associated with collapse is confined to the wetted zone (Fredlund and Gan, 1995). As matric suction can be visualised as isochrones (akin to excess pore water pressures seen in consolidation) analysis can be undertaken in much the same way. Further details of this, including experimental observations, are provided by Fredlund and Gan (1995). Collapse deformations that occur owing to suction reductions have been found to depend mainly on soil density and the stress state under which collapse occurs (Sun *et al.*, 2007).

Pereira and Fredlund (2000) highlighted key features in the collapse of compacted soils:

- For any type of soil compacted dry of optimum water content, collapse can occur.
- High microforces of shear strength exist through bonding, chiefly via capillary action.
- Compressibility gradually increases and shear strength gradually decreases in collapsible soils during saturation.

- Soil collapse progresses with increasing degree of saturation. However, above a critical degree of saturation, no further collapse occurs.
- Collapse is associated with localised shear failure (see further discussion in section 32.4.5 below).
- During wetting-induced collapse under constant load and anisotropic oedometer conditions, horizontal stresses increase.
- For a given mean normal total stress under triaxial conditions, the magnitude of axial collapse increases and radial collapse decreases with increased stress ratio.

Fill collapse potential is thus controlled by placement conditions, water content history and stress history. Further discussion on this subject and of other poorly compacted materials is provided by Charles and Skinner (2001), Charles and Watts (2001), and elsewhere in this manual; see Chapter 34 *Non-engineered fills*.

32.3.3 Modelling approaches – collapse prediction

Most collapsible soils exist in a partially saturated state. The suctions that develop are made up of two components: matric suction and osmotic suction, the sum of which is known as total suction. Discussion of these aspects and their implications is presented in Chapter 30 *Tropical soils* and a detailed treatment is also provided by Fredlund and Rahardjo (1993) and Fredlund (2006).

This approach has important limitations and a more complete model has been developed by Alonso *et al.* (1990). Their approach extended the Modified Cam Clay model to unsaturated soils and introduced the loading–collapse (LC) surface to define yielding due to either external loading (total stress) or saturation (loss of suction). Experimental evidence for this has been presented by a number of authors, e.g. Jotisankasa *et al.* (2009). The LC model further demonstrated the stress path dependency of collapse and explains why, under lower net stress, water inundation may cause swelling and at higher net stresses, collapse occurs. Therefore any modelling approaches used should treat soils as potentially expansive or collapsible in the same framework. However, it should be noted that collapse, unlike swelling, is an irreversible process.

The effects of bonding and bond yield strength are important aspects in many collapsible soils. Further details are discussed by Leroueil and Vaughan (1990), Maâtouk *et al*. (1995), Malandraki and Toll (1996) and Cuccovillo and Coop (1999).

Reviews of the constitutive models used to assess partially saturated soils are provided by D'Onza *et al.* (2011) and Sheng (2011), with discussion on their limitations provided by Zhang and Li (2011). Overall, constitutive models for partially saturated soils deal with the mechanical stress–strain and hydraulic suction–saturation relationships.

Constitutive models have allowed a number of numerical approaches to be developed. However, these often need a range of parameters for analysis and so may not be cost-effective for many projects. However, this can be mitigated to some degree using approaches developed by Nobar and Duncan (1972) and Farias *et al*. (1998) that utilise, for example, stress–strain curves for dry and saturated behaviour. A similar approach has been advocated by Charles and Skinner (2001) and Skinner (2001) when dealing with fill. In addition, discrete element methods (DEMs) are allowing micromechanical aspects of collapsible soils to be examined (see Liu and Sun, 2002, for further details).

32.4 Investigation and assessment

In order to provide an economical and efficient engineering solution four basic steps must be undertaken when dealing with collapsible soils (after Popescu, 1986):

(i) identification – determine whether a collapsible soils exists;

(ii) classification – if a collapsible soils exists, how significant is it?

(iii) quantification – assess the degree of collapse that will occur;

(iv) evaluation – assess the design options.

However, one of the greatest problems with collapsible soils is that their existence and the extent of their collapse potential are often not recognised prior to construction (Houston *et al.*, 2001). Thus it is essential to first identify a collapsible soil and then to estimate its collapse potential, particularly (but not exclusively) on sites containing water-sensitive soils.

Engineers often mistake, or simply do not recognise the presence of, a collapsible soil. Current standards relating to soil field descriptions, used by engineers, tend to group all fine materials together (e.g. silts and clays) under a common descriptor, which does not help in this regard. Whilst there are practical reasons for this, such groupings potentially reduce the ability of engineers to identify and assess whether a soil is collapsible. Moreover, even though a considerable database of knowledge exists globally, much of this work tends to be lost owing to use of formats and terms unfamiliar to engineers or simply suffers from language barriers (Jefferson *et al.*, 2003a).

Popescu (1986), Houston and Houston (1997) and Houston *et al*. (2001) provide excellent overviews of the key aspects associated with the identification and characterisation of collapsible soils. When characterising a collapsible soil, Houston *et al.* (2001) suggest the following stages be undertaken:

(i) reconnaissance;

(ii) use of indirect correlations;

(iii) laboratory testing;

(iv) field testing.

These aspects are, in general, common to the investigation of expansive soils and reference to discussions in Chapter 33 *Expansive soils* would be useful.

32.4.1 Reconnaissance

Using reconnaissance to gather useful geologic and geomorphologic information can be useful in anticipating collapse by providing clues on what to look out for. The first step is to understand geologic and geomorphologic settings (see section 32.3 above). For example, Lin (1995) found that there was a strong correlation between geomorphologic information and collapsibility. It may be that the underlying assumption should be that a deposit is collapsible until confirmed otherwise – as in the case of Beckwith (1995), who recommends that alluvial fans should all be assumed to be collapsible. Charles and Watts (2001) make a similar suggestion when dealing with partially saturated fill – until there is adequate evidence to the contrary. Further clues to the likelihood of collapsible soils can be gained from prior history and environmental factors.

Lin (1995) highlighted how collapse of loess soils was influence by age, overburden pressure, and the degree of saturation and suction, illustrating the difficulties associated with determining whether or not a soil is collapsible. Popescu (1992) highlighted other aspects that need to be considered when assessing the collapsibility of soils. These include:

(i) Internal factors

- mineralogy of particles;
- shape and distribution of particles;
- nature of interparticle bonding/cementing;
- soil structure;
- initial dry density (which is often low);
- initial water content.

(ii) External factors

- availability and nature of water;
- applied pressure;
- time permitted for water percolation to occur;
- stress history;
- climate.

Thus a detailed programme of classification and quantification is necessary to fully assess collapse potential across a site.

Details of reconnaissance and its role in site investigations are dealt with elsewhere in the manual – see Chapter 45 *Geophysical exploration and remote sensing*. Further information can also be found in Fookes *et al*. (2005).

32.4.2 Indirect correlations

Various collapse coefficients related to loess have been produced, which include ever more parameters, e.g. Basma and Tuncer (1992) and Fujun *et al*. (1998). However, these are unnecessarily complex and the traditional collapse potential, for example the one described by Gibbs and Bara (1962), is better owing to its relative simplicity.

Many criteria and correlations have been proposed in the assessment of collapse potential based on soil properties such as natural water content, void ratio or index properties (see Rogers *et al*., 1994; Northmore *et al*., 1996; Bell, 2000). However, these can be misleading as they are often based on remoulded and approximate soil properties; therefore, inappropriate evaluation can thus occur (Northmore *et al*., 1996). Many of the correlations available have met with only moderate success owing to their weak correlations and considerable scatter (Houston *et al*., 2001). Thus, it is more efficient and economical to use either laboratory or field tests when assessing collapse potential. This has the added advantage of providing not only identification, but also assessment data.

32.4.3 Laboratory testing

The most effective method to access collapsibility is through collapse tests. The actual collapse potential is traditionally measured using double and single oedometer tests (Jennings and Knight, 1975), which have been subsequently modified by Houston *et al.* (1988). The amount of collapse strain produced when the test specimen is flooded under a given pressure indicates a sample's susceptibility to collapse.

Figure 32.11 illustrates a typical response (Houston *et al.*, 1988), where a seating stress of 5 kPa has been used to establish an initial state. Any compression under this stress is attributed to sample disturbance. The initial compression curve A–B–C represents the response of the soil at its *in situ* water content. Pressure is applied until the stress on the sample is equal to (or greater than) that expected in the field. At this point, the sample is inundated and compression measured (C–D in **Figure 32.11**), after which further loading is undertaken, corresponding to line D–E. This equates to the three phases of collapse described by Pereira and Fredlund (2000).

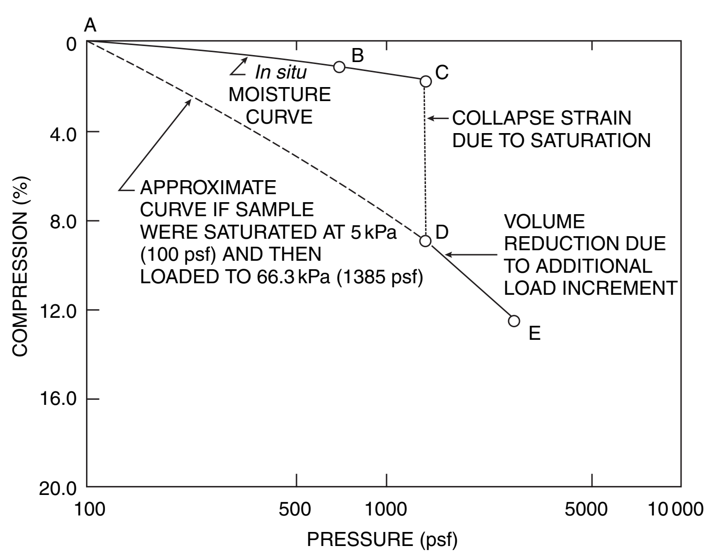

Figure 32.11 Compression curves for modified Jennings and Knight oedometer test (N.B. 1 pound per square foot (psf) = 0.0479 kPa)
Reproduced from Houston *et al.* (1988), with kind permission from ASCE

The amount of collapse of a layer is found by multiplying the thickness of the layer by the collapse strain, using values corresponding to the final stress at the midpoint of the zone in question. Further details are provided by Houston *et al.* (1988), who note that collapse strains vary both laterally and with depth, so requiring integration of strains along a vertical column of zones to estimate surface settlements.

The double oedometer test uses two near-identical specimens of soil and incremental stress increases are applied to one specimen in its natural state and to the other specimen that has been immediately saturated at under a seating stress of typically 5 kPa. Based on the double oedometer test, the degree of collapsibility can be assessed and used to provide an indication of the potential severity of collapse. **Table 32.1** provides details presented by Jennings and Knight (1975), which indicate a 1% collapse can be regarded as metastable. However, this cut-off varies across the world, with values of 1.5% taken in China (Lin and Wang, 1988) and values in excess of 2% in the USA used to indicate soils susceptible to collapse (Lutenegger and Hallberg, 1988).

Collapsible fill identification is often best achieved by testing samples at various water contents and dry densities over the range of stress levels expected (Houston *et al.*, 2001; Charles and Skinner, 2001).

Although only approximate, double oedometer tests do give a repeatable and reproducible qualitative indication of collapse. Often it is the collapsibility risk that is more important to assess than the actual amount of collapse that will occur. However, traditional oedometer tests suffer from sample disturbance effects and often reach saturations not commonly encountered in the field (Rust *et al.*, 2005). The extent of sampling effects has been debated, as have the relative merits of the use of block and tube sampling when testing collapsible soils (see Houston *et al.*, 1988; Day, 1990; Houston and El-Ehwany, 1991; Neely, 2010). Northmore *et al.* (1996) observed that, upon flooding in the oedometer, certain specimens of brickearth became saturated almost instantaneously with a rapid intake of water into the pore space.

Thus, at best, traditional oedometer collapse tests should be considered as index tests; for full collapse evaluation, a field trial should be conducted. When estimating collapse settlements, significant suctions may remain after wetting and soil will remain only partially collapsed (see section 32.5.1 for further discussion). This must therefore be accounted for in any assessments.

Other test methods have been employed to characterise collapsibility; these include suction-monitored oedometers (e.g. Dineen and Burland, 1995; Jotisankasa *et al.*, 2007; Vilar and Rodrigues, 2011), Rowe cells (Blanchfield and Anderson, 2000) and triaxial collapse tests (e.g. Lawton *et al.*, 1991; Pereira and Fredlund, 2000; Rust *et al.*, 2005). Further details are provided by Fredlund and Rahardjo (1993) and Rampino *et al.* (2000). Tarantino *et al.* (2011) provide a review of techniques used for measuring and controlling suctions, whilst Fredlund and Houston (2009) discuss protocols for the assessment of unsaturated soil properties in geotechnical practice. Vilar and Rodrigues (2011) provide a useful example of suction measurement in the assessment of collapse. Further details are presented in Chapter 30 *Tropical soils*.

Collapse (%)	Severity of problem
0–1	No problem
1–5	Moderate trouble
5–10	Trouble
10–20	Severe trouble
>20	Very severe trouble

Table 32.1 Collapse percentage (defined as $\Delta e/(1+e)$, where e is the void ratio) as an indication of potential severity
Data taken from Jennings and Knight (1975)

32.4.4 Field testing

Field methods have traditionally used plate loading tests (Reznik, 1991, 1995; Rollins *et al.*, 1994) and more recently, pressuremeter tests to determine collapse potential (Smith and Rollins, 1997; Schnaid *et al.*, 2004). Francisca (2007) provides details of the use of standard penetration tests (SPTs) to evaluate the constrained modulus and collapsibility of loess in Argentina, with higher N values being recorded in soils of a lower collapse potential. However, care is needed to ensure uniformity of stress state in the collapse region. This is often the main disadvantage with *in situ* collapse tests and has led a number of researchers to develop more sensitive test methodologies.

For example, Handy (1995) devised a stepped blade method to evaluate lateral stress changes. Methods to determine response to wetting were developed by Houston *et al.* (1995b) (the downhole plate test) and Mahmoud *et al.* (1995) (the box plate load test). A brief but useful overview of interpretation and comparison of collapse measurement techniques, including their relative merits, is provided by Houston *et al.* (1995a). These include minimal sample disturbance, large soil volumes tested and degree of wetting likely to be similar to the prototype.

Houston *et al.* (2001) present field investigations of the collapse potential of a low plasticity silt using an *in situ* test apparatus (see **Figure 32.12(a)**). Boxes of concrete or steel were lowered onto a concrete pad and filled with soil, the base of the foundation was inundated with water and movements were recorded (see also Mahmoud *et al.*, 1995). The relationship between partial collapse, matrix suction and degree of saturation (**Figure 32.12(b)**) highlights the importance of understanding the likely changes in water content of the soil surrounding a structure – something which Houston *et al.* (2001) suggests can be very difficult to predict accurately.

Houston *et al.* (2001) also provide a comparison between collapse predicted for a soil investigated using *in situ* methods (not the same tests as presented in **Figure 32.12**) and laboratory methods (**Figure 32.13**), and suggest that there are difficulties involved with the *in situ* approach. The load applied can

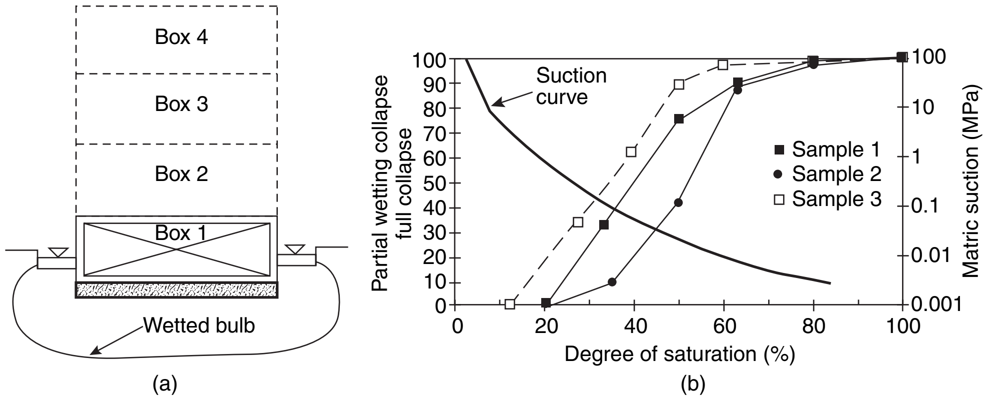

Figure 32.12 (a) Experimental layout to measure collapse of low plasticity silt; (b) the relationship between partial wetting collapse, matrix suction and degree of saturation (sample 1, 2 and 3 refer to three different silt soils of low plasticity)
Reproduced from Houston *et al.* (2001) with kind permission from Springer Science+Business Media

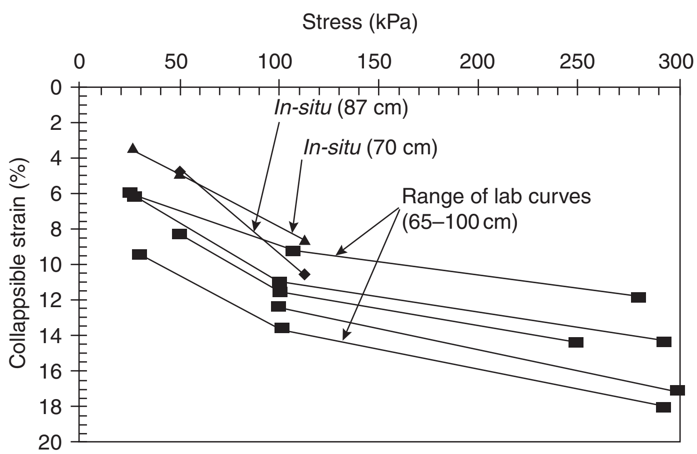

Figure 32.13 Comparison of collapsible strains for a soil investigated *in situ* and in the laboratory , using plate sizes of 70 cm and 87 cm in the plate bearing tests
Reproduced from Houston *et al.* (2001) with kind permission from Springer Science+Business Media

be controlled but the region affected by wetting and the degree of wetting can be very difficult to control.

Smith and Rollins (1997) investigated the use of a borehole pressuremeter to investigate the collapse potential of an arid soil. Once at the desired depth within the borehole, the pressuremeter expands radially to apply a pressure to the annulus of the borehole and measures the dry modulus of the soil (E_D). A set volume of water is discharged through the pressuremeter into the surrounding soil; the modulus during collapse (E_C) and wet modulus (E_W) post-collapse are then measured (see **Figure 32.14**). Smith and Rollins (1997) suggest that the moduli ratios E_C/E_D and E_W/E_D can be used to predict the collapsibility of a potentially collapsible soil (**Table 32.2**).

With recent improvements in technology, geophysical approaches have been advanced as a method to determine collapse potential (Evans *et al.*, 2004; Rodrigues *et al.*, 2006). The power of geophysical approaches to assess when a collapsible zone is present and its extent across a site was illustrated by Northmore *et al.* (2008), see **Figure 32.15**.

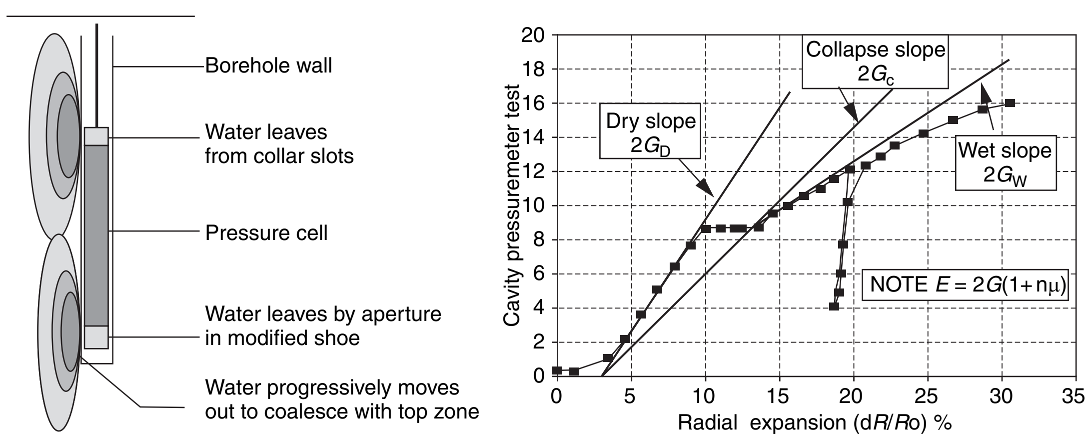

Figure 32.14 Illustration of pressuremeter (water only illustrated on left hand for clarity) and determination of the dry, collapsible and wet moduli, where *G* is shear modulus and μ is Poisson ratio
Reproduced from Smith and Rollins (1997); ASTM

Northmore *et al*. (2008) illustrated how depth and lateral extent of both collapsible and a non-collapsible loessic brick-earth could be ascertained using a suite of different geophysical approaches, including electromagnetic (EM31 and EM34), electrical resistivity and shear wave profiles. Further details are provided by Gunn *et al.* (2006) for shear wave measurements and Jackson *et al.* (2006) for resistivity measurements. However, it is vital that adequate calibration is undertaken through laboratory testing. For this, traditional double oedometer and Panda probe profiling are used, where the Panda probe is a lightweight dynamic cone penetrometer which allows detailed physical soundings down to around 5 m to be taken (Langton, 1999). Once achieved, a full assessment of the presence of a potentially collapsible soil across the site can be made.

An important aspect with any field evaluation is the number of tests needed to adequately characterise the collapse potential of a loess soil. Houston *et al.* (2001) discuss statistical approaches developed to evaluate the minimum number of tests required to satisfactorily characterise a site and its collapse potential.

Criteria			Collapse potential
E_C/E_D		E_W/E_D	
≈1	and	≈1	Not collapsible
<1	and	≈1	Moderate collapse
≈1	and	<1	Moderate collapse
<1	and	<1	Possible severe collapse

Table 32.2 Use of modulus ratios to predict collapsibility of potentially collapsible soil
Data taken from Smith and Rollins (1997)

32.4.5 Assessment of wetting

The most challenging task for improving collapsible soils is the assessment of wetting extent and degree of potential future wetting. This is particularly true in arid or semi-arid environments where collapsible soils have not been wetted to any significant depth. However, even with collapsible deposits found in more humid environments, the deposits are often situated where significant wetting at depth has not occurred, or only part saturation has taken place, rendering the deposit still partially collapsible (Northmore *et al.*, 1996; Charles and Watts, 2001).

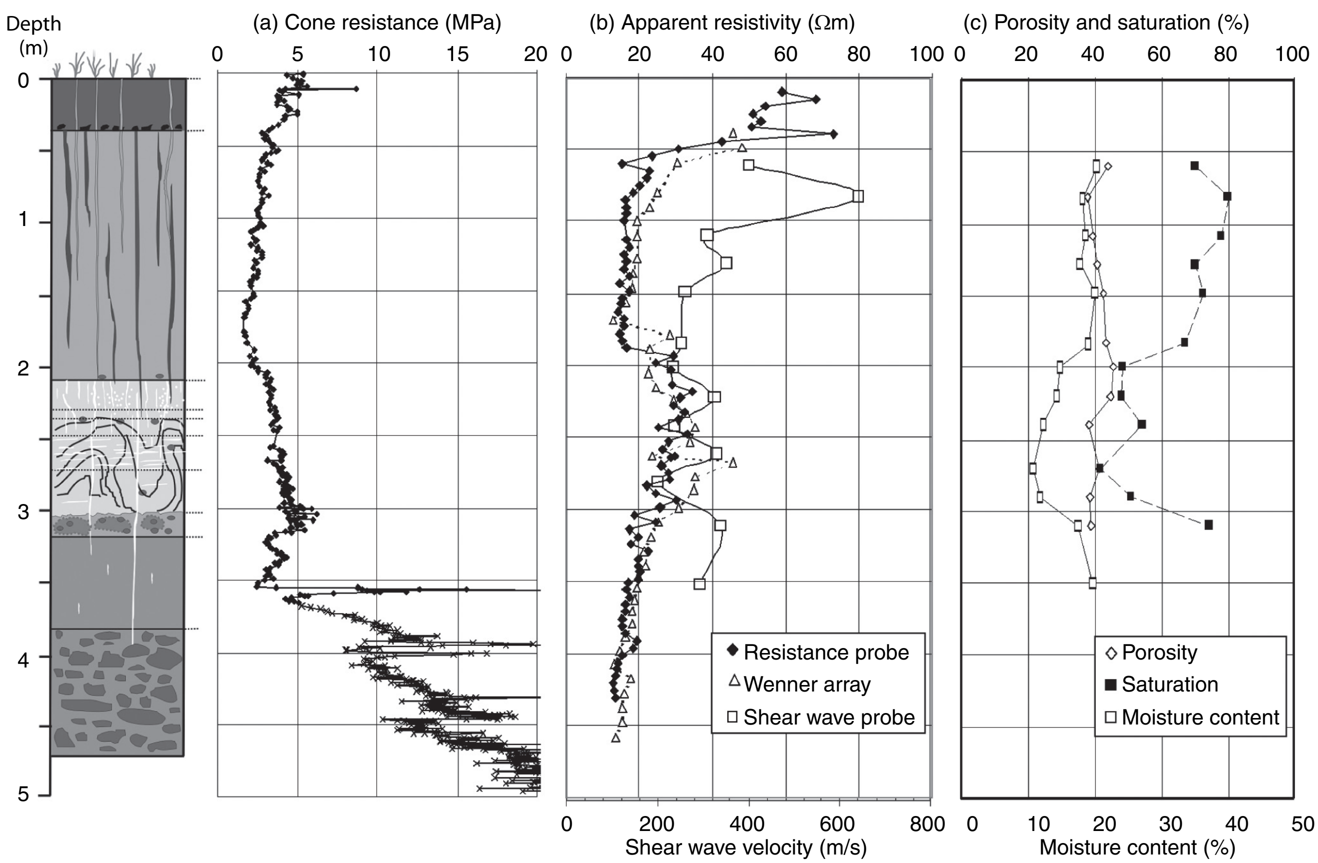

Figure 32.15 Profile through collapsible (~2–3 m) and non-collapsible (~0.5–2 m) brickearth – with collapsible deposits existing between 2 to 3 m, correlating to a drop in the degree of saturation
Reproduced from Northmore *et al.* (2008) all rights reserved

In general, detrimental effects of collapse occur in the zones both under foundations and within the probable wetting front (Houston *et al.*, 1988). Ponding tests may be used which can give an indication of depth and lateral extent of water migration, from which the best foundation option can be determined. El-Ehwany and Houston (1990) present results from laboratory infiltration wetting fronts and, by comparing these with observed rates, predict the depth of wetting versus time. They then describe how this information can be used to predict collapse settlements, taking account of partial wetting.

Clearly an assessment of the extent and degree of wetting is essential to determine collapse potential and the scope and requirements for any treatment processes. Many practitioners tend to be conservative and assume the degree of wetting equates to 100%, particularly if the collapsible zone is near to the surface and does not extend too deeply (Houston and Houston, 1997). Full wetting of a collapsible soil would only be expected with rising ground water. However, this is not often the case, and saturation only usually reaches between 35 and 60%, particularly when downward infiltration occurs. Hence the additional costs associated with such a conservative assumption may not be warranted (Houston *et al.*, 2001). El-Ehwany and Houston (1990) found that 50% saturation produces 85% of full collapse, agreeing approximately with the observations presented in **Figure 32.12 (b)**. They suggest that full collapse is achieved at between 65 and 70% saturation. Lawton *et al.* (1992) and others have shown that partial saturation will first trigger partial collapse, with full collapse occurring at saturation values as low as 60% – a figure that Bally (1988) agrees with. However, Osopov and Sokolov (1995) considered that full collapsibility would be realised only when saturation exceeded 80%.

This has implications for the prediction of collapse settlement, based on laboratory tests, which overestimate collapse strain and generally produce a greater degree of saturation than is achieved in the field; estimates put the overestimation at around 10% (El-Ehwany and Houston, 1990). However, this is not particularly large given the nature and accuracy of settlement predictions in general.

Wetting effects can be modelled using unsaturated stress state variables: net normal stress ($\sigma - u_a$) and matric suction ($u_a - u_w$) – further details can be found in Fredlund and Rahardjo (1993). The matric suction changes during wetting can be indicated by soil–water characteristic curves (SWCCs); further details are discussed in Chapter 30 *Tropical soils*. Houston *et al.* (2001) provide a range of SWCCs for collapsible loess soils from around the world. Further details of the evaluation of wetting using suction measurements have been discussed by Walsh *et al.* (1993).

All collapsible soils will experience partial collapse under partial wetting conditions. However, the shape and position of the partial collapse curve depend on the soil type, fines content and type of bonding present. Overall, it is clear that assessment of the extent and degree of wetting is the most difficult part of collapsibility evaluation.

32.5 Key engineering issues

32.5.1 Foundation options

Four basic approaches exist when dealing with design solutions in collapsible soils (Popescu, 1992):

(i) Use very stiff raft foundations and a rigid superstructure to minimise the effects of differential settlements (e.g. **Figure 32.16**). This tends to be expensive and not universally successful.

(ii) Ensure sufficient flexibility of the foundation and superstructure to accommodate ground movements without damage. This approach may be more applicable to smaller, lower cost buildings. Alternatively, a split rigid building can be flexibly connected.

(iii) Bypass the collapsible layer by use of piles.

(iv) Control or alter ground conditions through one or more of the various improvement techniques available (see section 32.5.5 and Chapter 25 *The role of ground improvement*).

Where the thickness of collapsible soil is relatively small, foundation recommendations are straightforward: the foundation level should be set below the collapsible soil layer. If this is not the case, then some form of pre-treatment is necessary to remove collapse potential.

If the collapsible layer is deep or is of significant thickness below the surface, pile foundations are used. However, Grigoryan (1997) has reported several cases where piles used in collapsible soils have experienced loss of bearing capacity and excessive settlements immediately after inundation through negative skin friction effects. In addition, the presence of a collapsible layer may adversely affect the performance of piles during the life of a building.

Kakoli (2011) provides one of the few detailed reviews and assessments of piles used in collapsible soils, drawing mainly on the work of Grigoryan and Grigoryan (1975). They suggested that in collapsible soils, negative skin friction exists for a few hours but disappears after pile settlement. Chen *et al.* (2008) present load tests for piles in collapsible soils subjected to inundation. They measured negative skin friction across five sites in China, finding values of negative skin friction between

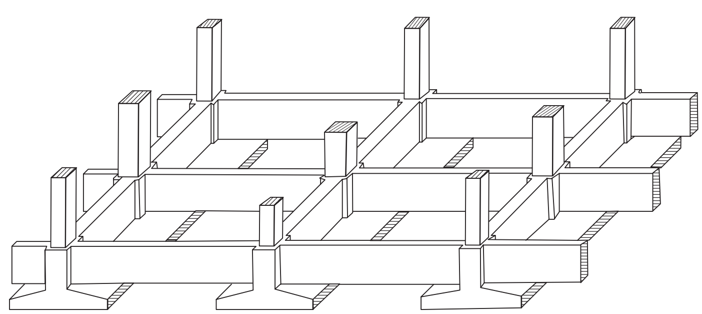

Figure 32.16 Continuous footing design for collapsible soils
Reproduced from Zeevaert (1972); Van Nostrand Reinhold Co.

around 20 and nearly 60 kPa. Kakoli (2011) presents numerical works (using PLAXIS) in the assessment of the effects of inundation on the performance of piles in collapsible soils, achieving results comparable with those presented by Chen *et al.* (2008). He further demonstrates how the effects of inundation increase collapse potential. Further details of the general behaviour of piles in collapsible soils are provided by Redolfi and Mazo (1992).

Often less expensive foundation options can be employed: Bally (1988) and Poposecu (1992) provide some examples of foundation practice in collapsible soils and Jefferson *et al.* (2005) illustrate this with a case study from Bulgaria.

32.5.2 Transport and utility infrastructure

As with foundations, transport infrastructure (roads and railways) can experience problems due to collapse. Particular problems for roads are the non-uniform collapse and non-uniform wetting of sub-grades that occur along the length of a road. These cause rough, wavy surfaces and have the potential to result in many miles of extensive damage to road structures (Houston, 1988). The potential damage to railways is more severe owing to their intolerance of longitudinal differential settlement, and the same is true for pipelines unless they are sufficiently flexible to permit longitudinal differential or lateral movements. Moreover if water pipelines or sewers suffer fracture due to differential movement, water leakage can exacerbate the collapse phenomena. Damage can also occur to associated structural and geotechnical assets, including bridges, slopes and cuttings.

Stress applied to sub-grade from highways has two components: overburden stress and imposed traffic loading (determined using common pavement analytical tools). It is likely that once wetting has occurred, short duration loading from heavy lorries would be sufficient to cause full collapse. It should be noted that collapse strains can occur at any depth at which significant wetting occurs (see section 32.4.5, above), regardless of the relative balance between overburden and imposed loads. Clearly changes in soil type along the length of a highway are important and routine non-destructive tests such as the falling weight deflectometer are useful here (Houston *et al*., 2002).

Drainage management is of particular concern, as most collapsible soils have higher permeabilities and suctions than road foundation materials. In addition, the pavement interrupts evaporation and changes water content regimes relative to the surrounding land. Hence significant risks of wetting collapsible soil sub-grades exist; the extent and depth of wetting is controlled by the sources of water, for example rising groundwater or surface ponding from poor drainage.

The approaches discussed above in section 32.4 and below in section 32.5.5 essentially apply to roads built on collapsible soils. Further details specific to roads are provided by Houston (1988) and Houston *et al.* (2002). Examples of parameters used in pavement design associated with collapsible soils are provided by Cameron and Nuntasarn (2006) and Roohnavaz *et al*. (2011).

32.5.3 Dynamic behaviour

Collapsible soils are particularly susceptible to liquefaction and dynamic settlement owing to their highly contractive nature during shearing. This can result in devastating consequences, with the Haiyuan landslide being a prime example (see Zhang and Wang, 2007). However, detection of the liquefaction and dynamic settlement potential of collapsible soils is difficult as they often have sufficient cementing when dry to prevent significant deformations during dynamic or earthquake loading (Houston *et al*., 2001). If, however, they become wetted, these bonds weaken and their liquefaction and dynamic settlement potential can significantly increase. Post-wetting behaviour is therefore of particular importance in earthquake-prone regions, especially if collapse is triggered by rising groundwater (Houston *et al.*, 2003). Moreover, these soils often have insufficient fines to render them non-liquefiable if saturated. Hence compressions induced upon wetted collapse may be inadequate to mitigate liquefaction and dynamic settlement potential (Houston *et al.*, 2001).

Houston *et al.* (2003) provide a brief overview of research that examines dynamic behaviour of collapsible soils. Their results show how cyclic stress ratios (CSRs) are strongly dependent on the degree of saturation (see **Figure 32.17**). Here, failure is taken to occur at CSR causing 10% strain. Cyclic stress ratios can be defined as the ratio of maximum shear stress (related to the cyclic shear stress amplitude of the earthquake) to vertical effective stress.

Liquefaction in loess soils is complicated by their microstructural aspects and so the process is generally less well understood than liquefaction in sands (see Hwang *et al.*, 2000; Wang *et al.*, 2004). However, improvement approaches that have proved successful in reducing liquefaction potential include dynamic compaction, use of compaction piles and methods that alter the soil's structure such as grouting (Wang *et al.*, 2004) – see section 32.5.5 for more details.

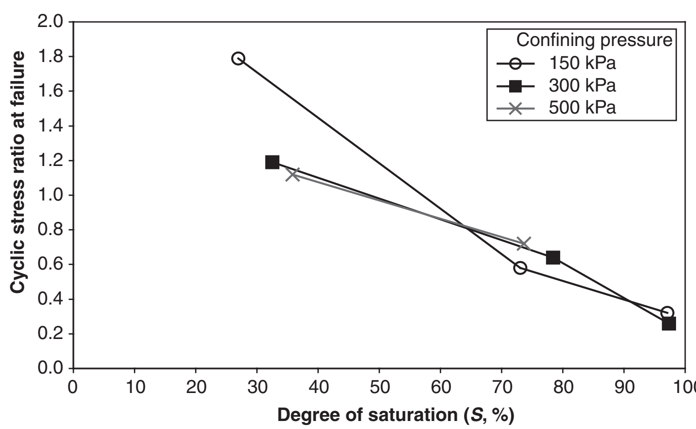

Figure 32.17 Variation of cyclic stress ratio at failure with degree of saturation
Reproduced from Houston *et al.* (2003)

Wang *et al.* (1998, 2004) provide details of lessons learnt when dealing with earthquake-induced problems in loess soils through the use of dynamic triaxial tests and resonant column tests. Key earthquake-induced problems are seismically induced landslides (e.g. the 1920 Haiyuan landslide – Zhang and Wang, 2007); seismically induced subsidence, and liquefaction. Similar problems can occur in other collapsible soils due to earthquakes, for instance seismically induced landslides triggered in quick clays (Stark and Contreras, 1998).

32.5.4 Slope stability

Slope stability issues related to collapsible soils are a problem associated extensively with loess soils, although other collapsible deposits such as quick clay can exhibit significant problems, e.g. the Rissa landslide (see section 32.2.2 above), with further details discussed by Bell (2000).

However loess regions, such as the loess mantled mountainous region of Gansu in China, have suffered more than 40 000 large scale landslides over the last century (Meng and Derbyshire, 1998). Landslides in this region are caused by seismic shocks and severe summer monsoonal rains. As a consequence of this rainfall, loess karst and sinkholes features are additional hazards.

Landslides in loess are very diverse owing to a broad range of conditions in which they occur (Derbyshire and Meng, 2005). The principal types of landslide in loess are shown in **Figure 32.18** with detailed descriptions given in Meng and Derbyshire (1998) and Meng *et al.* (2000b). It should be noted that Tan-ta are small adjustment failures, generally less than 10 m in diameter and at a depth of a few metres, occurring predominately on steep slopes in loess. After initiation of the slide, rapid disintegration takes place, often with high sliding velocities (Meng *et al.*, 2000b).

The considerable experience from China had led to success with a number of control measures. These include:

- landscaping, e.g. stepped slopes;
- retaining structures, e.g. retaining walls, underground drainage trenches and retaining piles.

Using these methods, most shallow and moderately deep landslides can be treated. Often a combination of approaches has proved to be best; for instance, using a combined structure of retaining walls and drainage ditches. However, terrace landslides can prove difficult to handle and often require a horizontal drainage borehole to alleviate groundwater pressures. Because of these pressures loess liquefaction, induced by slight slope displacement, can threaten whole slope stability.

Further details are provided by Meng and Derbyshire (1998) and Meng *et al.* (2000a), including details of successful treatment for landslide control. In addition, Dijkstra *et al.* (2000a) provide a detailed account of laboratory and *in situ* strength measurements with respect to slope stability in loess; details of modelling of landslides are presented by Dijkstra *et al.* (2000b).

32.5.5 Improvement and remediation

A wide variety of improvement processes exist for collapsible soils. Some of the more exotic ones have only been tried at an experimental stage. Evstatiev (1988), Houston *et al.* (2001)

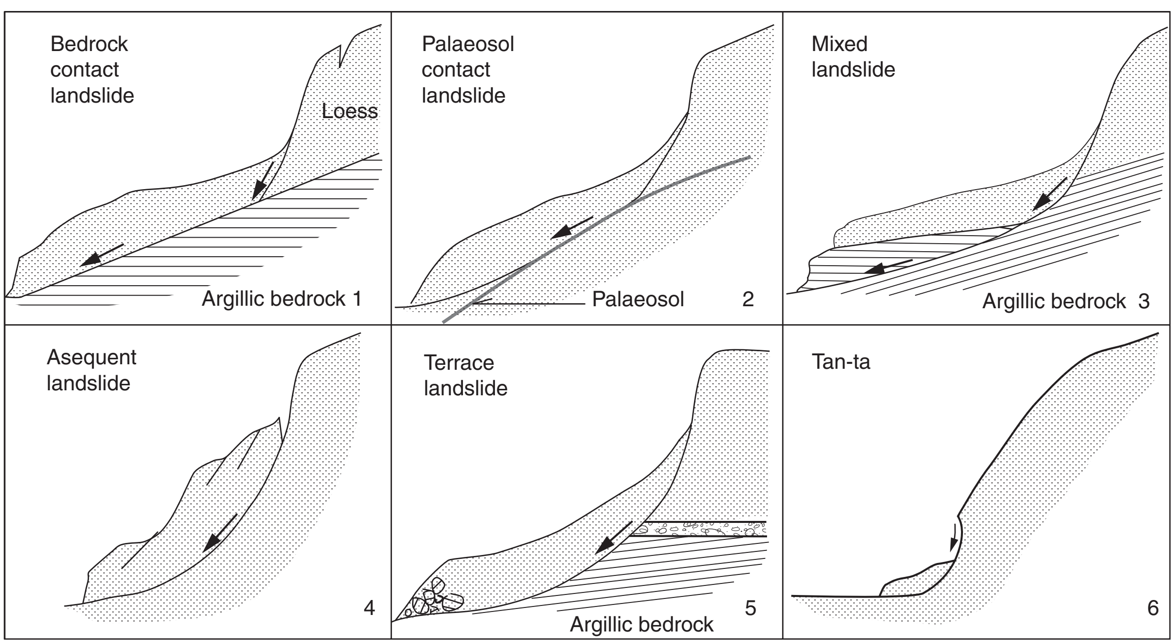

Figure 32.18 Principal types of loess landslides
Reproduced from Meng and Derbyshire (1998) © The Geological Society of London

and Jefferson *et al*. (2005) provide excellent overviews of a range of possible treatment techniques used to improve collapsible loess soils.

Ultimately the best technique depends on several factors (after Houston *et al.*, 2001):

(i) when the collapsible loess soil was discovered;

(ii) how stress is to be applied to the soil;

(iii) depth and extent of the collapsible zone;

(iv) sources of wetting;

(v) costs.

Details are provided throughout the literature, in particular the experience from Eastern Europe, especially Russia, e.g. Abelev (1975), Lutenegger (1986), Ryzhov (1989), Evstatiev (1995) and Evstatiev *et al.* (2002). Deng (1991), Wang (1991), Zhong (1991), Zhai *et al.* (1991), Fujun *et al.* (1998) and Gao *et al*. (2004), amongst others, allow insights into treatment techniques commonly employed in China, while researchers such as Clemence and Finbarr (1981), Rollins and Rogers (1994), Rollins and Kim (1994), Pengelly *et al.* (1997), Houston and Houston (1997), Rollins *et al*. (1998), Houston *et al.* (2001) and Rollins and Kim (2011) provide reviews of ground improvement approaches used to treat collapsible soils in North America. Rollins and Rogers (1994) provide an overview of the various advantages and limitations of a number of different possible treatment methods to reduce/eliminate collapsibility.

Table 32.3 provides an overview of techniques that can be used to treat loess ground and reduce/remove its collapse potential.

It is very common to find bands of clayey material (palaeosol) within some collapsible soils, such as loess deposits. However, it still uncertain what role these bands play in the overall behaviour of the loess mass. It is likely that they will act as planes of weakness. What is certain, though, is that the presence of these bands will influence the effectiveness of any method used to treat a collapsible soil. These bands will affect the transmission of compaction energy and the route taken by stabilising fluids through the soil.

Post-construction treatment typically involves some form of chemical stabilisation – typically grouting – or alternatively some form of underpinning, both of which can prove expensive. The following methods have been used in cases where collapsing loess has damaged existing buildings: silicate grout injection (silicatisation), jet-grouting, underpinning by root piles (pali radici), squeeze grouting (injection by 'tube à manchettes') and stabilisation by in-depth heating.

An alternative proposed by Houston *et al*. (2001) involves controlled differential wetting via separately controllable trenches built around the foundation slab. This approach has been used to tilt a structure in a controlled way. Initial trials demonstrated that, firstly, it was possible to re-level the foundation thereby eliminating any future collapse potential, and, secondly, its control was relatively straightforward by allowing site owners to control flow rates from each trench. However,

Depth (m)	Treatment method	Comments
0–1.5	Surface compaction via vibratory rollers, light tampers	Economical, but requires careful site control, e.g. limits on water content
	Pre-wetting (inundation)	Can effectively treat thicker deposits but needs large volumes of water and time
	Vibrofloatation	Needs careful site control
1.5–10	Vibrocompaction (stone columns, concrete columns, encased stone columns)	Cheaper than conventional piles but requires careful site control and assessment. If uncased, stone columns may fail with loss of lateral support on collapse
	Dynamic compaction; rapid impact compaction	Simple and easily understood but requires care with water content and vibrations produced
	Explosions	Safety issues need to be addressed
	Compaction pile	Needs careful site control
	Grouting	Flexible but may adversely affect the environment
	Ponding/inundation/pre-wetting	Difficult to control effectiveness of compression produced
	Soil mixing lime/cement	Convenient and gains strength with time. Various environmental and safety aspects, and the chemical controls on reactions need to be assessed
	Heat treatment	Expensive
	Chemical methods	Flexible; relatively expensive
>10	As for 1.5–10 m, some techniques may have a limited effect	(See above)
	Pile foundations	High bearing capacity but expensive

Table 32.3 Methods of treating collapsible loess ground
Reproduced from Jefferson *et al.* (2005), with permission from Elsevier

as yet there are few directly relevant precedents, which limit its take-up.

A number of case studies illustrate how collapsibility has been successfully treated and details of these are provided by Evans and Bell (1981), Jefferson *et al*. (2005) and Rollins and Kim (2011).

32.6 Concluding remarks

Collapsible soils are found throughout the world and are formed through various geomorphologic and geologic processes. These can be natural (through fluvial or aeolian processes) or man-made (via poor compaction). Whatever the processes involved, the key prerequisite is that an open metastable structure develops through bonding mechanisms generated via capillary forces (suctions) and/or through cementing materials such as clay or salts. Collapse occurs when net stresses (via loading or saturation) exceed the yield strength of the bonding material. Inundation is by far the most common cause of collapse and can be triggered through a range of different water sources. Different sources yield different amounts of collapse.

Thus a detailed knowledge of macroscopic and microscopic characteristics is vital to engineer these materials effectively and safely. Failure to recognise and deal with collapsible soils can have a significant impact on the built and urban environments – with catastrophic effects and potential loss of life. To engineer effectively in collapsible soils it is essential to recognise their existence, for which key geologic and geomorphologic information is vital. However, collapsibility should be confirmed through direct response to wetting/loading tests using laboratory and field methods. The key challenge with collapsible soils is to predict the extent and degree of wetting that will take place. Care is needed to ensure that appropriate and realistic assessments are undertaken, followed by treatment through a suite of the possible improvement techniques available. The collapsible potential can then be eliminated effectively.

32.7 References

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32.7.1 Further reading

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32.7.2 Useful websites

British Geological Survey; www.bgs.ac.uk

It is recommended this chapter is read in conjunction with

- Chapter 7 *Geotechnical risks and their context for the whole project*
- Chapter 14 *Soils as particulate materials*
- Chapter 40 *The ground as a hazard*
- Chapter 58 *Building on fills*

All chapters in this book rely on the guidance in Sections 1 *Context* and 2 *Fundamental principles.* A sound knowledge of ground investigation is required for all geotechnical works, as set in out Section 4 *Site investigation.*