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CONTENTS

Chapter 33

Expansive soils

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Expansive soils present significant geotechnical and structural engineering challenges the world over, with costs associated with expansive behaviour estimated to run into several billion pounds annually. Expansive soils are those which experience significant volume changes associated with changes in water content. These volume changes can either be in the form of swell or shrinkage, and are sometimes known as swell–shrink soils. Key aspects that need identification when dealing with expansive soils include soil properties, suction/water conditions, temporal and spatial water content variations that may be generated, for example, by trees, and the geometry/stiffness of foundations and associated structures. Expansive soils can be found both in humid environments where expansive problems occur with soils of high plasticity index, and in arid/semi-arid soils where soils of even moderate expansiveness can cause significant damage. This chapter reviews the nature and extent of expansive soils, highlighting key engineering issues. These include methods to investigate expansive behaviour both in the field and the laboratory, and the associated empirical and analytical tools to evaluate expansive behaviour. Design options for pre- and post-construction are highlighted for both foundations and pavements, together with methods to ameliorate potentially damaging expansive behaviour.

33.1 What is an expansive soil?

Essentially, expansive soil is one that changes in volume in relation to changes in water content. The focus here is on soils that exhibit significant swell potential and, in addition, shrinkage potential. There are a number of cases where expansion can occur because of chemically induced changes (e.g. swelling of lime-treated sulfate soils). However, many soils that exhibit swelling and shrinking behaviour contain expansive clay minerals, such as smectite, that absorb water. The more of this clay a soil contains, the higher its swell potential and the more water it can absorb. As a result, these materials swell and thus increase in volume when they become wet, and shrink when they dry. The more water they absorb, the more their volume increases – for the most expansive clays expansions of 10% are not uncommon (Chen, 1988; Nelson and Miller, 1992). It should be noted that other soils exhibit volume change characteristics with changes in water content, e.g. collapsible soils, and these are dealt with in Chapter 32 *Collapsible soils*.

The amount by which the ground can shrink and/or swell is determined by the water content in the near-surface zone. Significant activity usually occurs to about 3 m depth, unless this zone is extended by the presence of tree roots (Driscoll, 1983; Biddle, 1998). Fine-grained clay-rich soils can absorb large quantities of water after rainfall, becoming sticky and heavy. Conversely, they can also become very hard when dry, resulting in shrinking and cracking of the ground. This hardening and softening is known as 'shrink–swell' behaviour. The effects of significant changes in water content on soils with a high shrink–swell potential can be severe on supporting structures.

Swelling and shrinkage are not fully reversible processes (Holtz and Kovacs, 1981). The process of shrinkage causes cracks which, on re-wetting, do not close up perfectly and hence cause the soil to bulk out slightly, and also allow enhanced access to water for the swelling process. In geological timescales, shrinkage cracks may become in-filled with sediment, thus imparting heterogeneity to the soil. When material falls into cracks, the soil is unable to move back – resulting in enhanced swelling pressures.

The primary problem with expansive soils is that deformations are significantly greater than those that can be predicted using classical elastic and plastic theory. As a result, a number of different approaches have been developed to predict and engineer expansive soils, and these are highlighted throughout this chapter.

33.2 Why are they problematic?

Many towns, cities, transport routes and buildings are founded on clay-rich soils and rocks. The clays within these materials may be a significant hazard to engineering construction due to their ability to shrink or swell with changes in water content. Changing water content may be due to seasonal variations (often related to rainfall and the evapotranspiration of vegetation), or be brought about by local site changes such as leakage from water supply pipes or drains, changes to surface drainage and landscaping (including paving), or following the planting, removal or severe pruning of trees or hedges, as man is unable to supply water to desiccated soil as efficiently as a tree originally extracted it through its root system (Cheney, 1988). During a long dry period or drought, a persistent water deficit may develop causing the soil to dry out to a greater depth than normal, leading to long-term subsidence. This is why expansive problems are often found in arid environments (see Chapter 29 *Arid soils*). As this water deficit dissipates it is possible that long-term heave may occur.

In the UK, the effects of shrinkage and swelling were first recognised by geotechnical specialists following the dry summer of 1947, and since then the cost of damage due to the shrinking and swelling of clay soils in the UK has risen dramatically. After the drought of 1975/76, insurance claims came to over £50 million. In 1991, after the preceding drought, claims peaked at over £500 million. Over the past 10 years the adverse effects of shrink–swell behaviour have cost the economy an estimated £3 billion, making it the most damaging geohazard in Britain today. The Association of British Insurers has estimated that the average cost of shrink–swell related subsidence to the insurance industry stands at over £400 million annually (Driscoll and Crilly, 2000). In the US, the estimated damage to buildings and infrastructure exceeds $15 billion annually. The American Society of Civil Engineers estimates that one in four homes have some damage caused by expansive soils. In a typical year, expansive soils cause a greater financial loss to property owners than earthquakes, floods, hurricanes and tornadoes combined (Nelson and Miller, 1992).

Swelling pressures can cause heaving, or lifting, of structures whilst shrinkage can cause differential settlement. Failure results when the volume changes are unevenly distributed beneath the foundation. For example, water content changes in the soil around the edge of a building can cause swelling pressure beneath the perimeter of the building, while the water content of the soil beneath the centre remains constant. This results in a failure known as 'end lift' (**Figure 33.1**). The opposite of this is 'centre lift', where swelling is focused beneath the centre of the structure or where shrinkage takes place under the edges.

Figure 33.1 Structural damage to house caused by 'end lift'
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Damage to foundations in expansive soils commonly results from tree growth. This occurs in two principal ways: (i) physical disturbance of the ground, and (ii) shrinkage of the ground by removal of water. Physical disturbance of the ground caused by root growth is often seen as damage to pavements and broken walls. An example of vegetation-induced shrinkage causing differential settlement of building foundations is provided in **Figure 33.2**. Vegetation-induced changes to water profiles can also have a significant impact on other underground features, including utilities. Clayton *et al*. (2010), reporting monitoring data over a two-year period of pipes in London Clay, found significant ground movements (both vertical and horizontal) of the order of 3–6 mm/m length of pipe, which generated significant tensile stresses when in the vicinity of trees. Such tree-induced movement has the potential to be a significant contributor to failure of old pipes located in clay soils near deciduous trees (Clayton *et al*., 2010). Further details are discussed in section 33.5.4.5.

33.3 Where are expansive soils found?

In the UK, towns and cities built on clay-rich soils most susceptible to shrink–swell behaviour are found mainly in the southeast of the country (**Figure 33.3**). In the southeast, many of the clay formations are too young to have been changed into stronger mudstones, leaving them still able to absorb and lose moisture. Clay rocks elsewhere in the country are older and

Figure 33.2 Example of differential settlement due to influence of trees

have been hardened by processes resulting from deep burial and are less able to absorb water. Some areas (e.g. around The Wash, northwest of Peterborough – see **Figure 33.3**) are deeply buried beneath other (superficial) soils that are not susceptible to shrink–swell behaviour. However, other superficial deposits such as alluvium, peat and laminated clays can also be susceptible to soil subsidence and heave (e.g. in the Vale of York, east of Leeds – see **Figure 33.3**).

Expansive soils are found throughout many regions of the world, particularly in arid and semi-arid regions, as well as

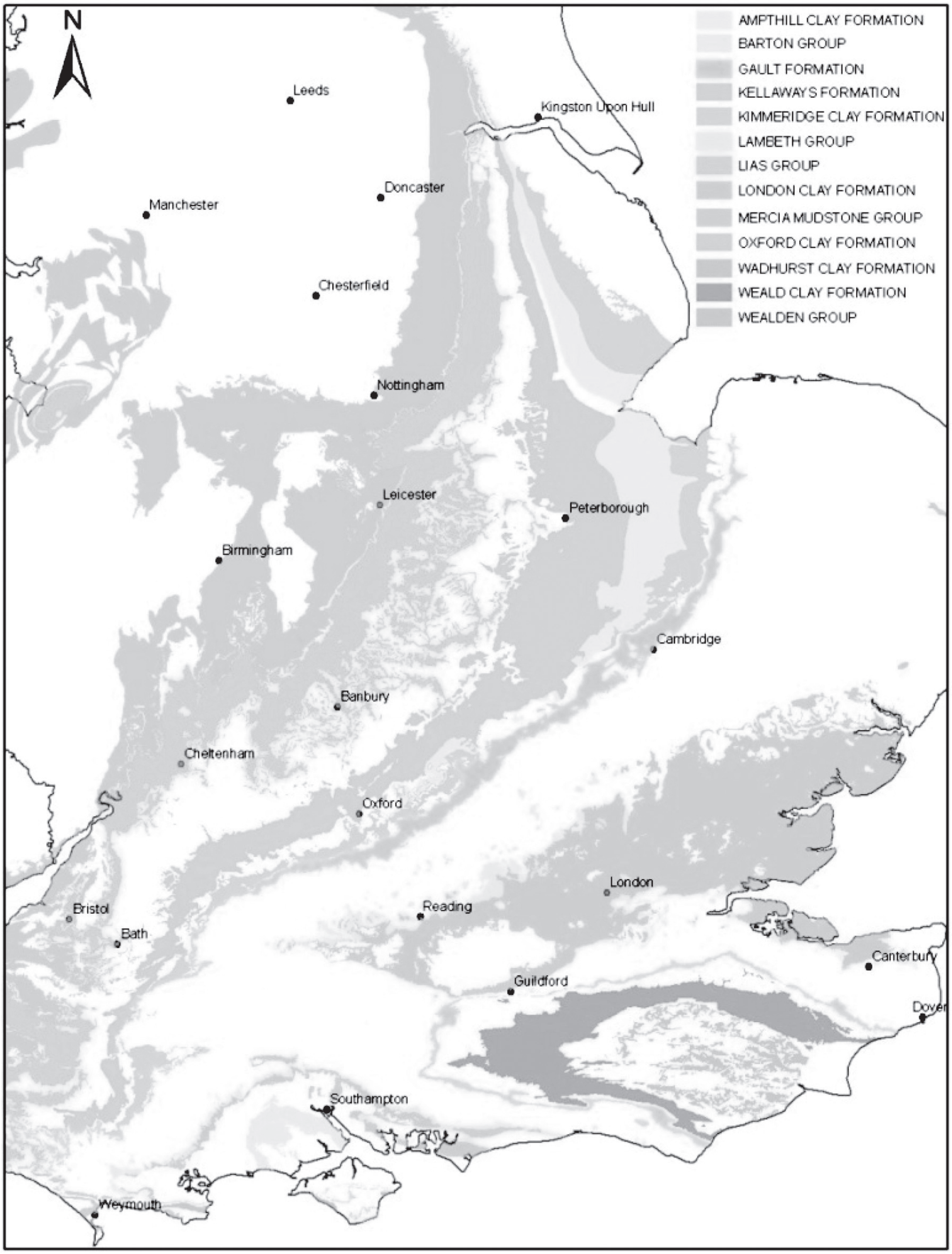

Figure 33.3 Distribution of UK clay-rich soil formations. A colour version of this figure is available online

those where wet conditions occur after prolonged periods of drought. Their distribution is dependent on geology (parent material), climate, hydrology, geomorphology and vegetation.

The literature is full of studies from all over the world, concerned with problems associated with expansive clays (e.g. Simmons, 1991; Fredlund and Rahardjo, 1993; Stavridakis, 2006; Hyndman and Hyndman, 2009). Expansive soils incur major construction costs around the world, with notable examples found in the USA, Australia, India and South Africa, to name but a few. In these countries, or significant areas of them, the evaporation rate is higher than the annual rainfall so there is usually a moisture deficiency in the soil. Subsequently, when it rains, the ground swells and so increases the potential for heave to occur. In semi-arid regions a pattern of short periods of rainfall followed by long dry periods (drought) can develop, resulting in seasonal cycles of swelling and shrinkage.

Due to the global distribution of expansive soils, many different ways to tackle the problem have been developed and these can vary considerably (Radevsky, 2001). The methods to deal with the problem of expansive soils differ in many ways and depend not only on technical developments, but also on the legal framework and regulations of a country, insurance policies and the attitude of insurers, experience of the engineers and other specialists dealing with the problem, and importantly the sensitivity of the owner of the property affected. In the UK in particular, there is high sensitivity to relative small cracks (see section 33.5.3, below). A summary of these issues is provided by Radevsky (2001) in his review of how different countries deal with expansive soil problems, and a detailed informative study from Arizona, USA has more recently been presented by Houston *et al*. (2011). The latter study demonstrated how the source of problems from expansive soils often stems from poor drainage, construction problems, homeowner activity and its adverse effects, and landscaping through the use of vegetation, or a combination of these. These aspects may cause more expansive soil problems than landscape type itself.

Overall, in humid climates, problems with expansive soils tend to be limited to those soils containing higher plasticity index (I_p) clays. However, in arid/semi-arid climates, soils that exhibit even moderate expansiveness can cause distress to residential property. This stems directly from their relatively high suction and the larger changes in water content that result when water levels change.

33.4 Shrink–swell behaviour

Excluding deep underground excavations (e.g. tunnels), shrinkage and swelling effects are restricted to the near-surface zone. Significant activity usually occurs to about 3 m depth, but this can vary depending on climatic conditions. The shrink–swell potential of expansive soils is determined by its initial water content, void ratio, internal structure and vertical stresses, as well as the type and amount of clay minerals in the soil (Bell and Culshaw, 2001). These minerals determine the natural expansiveness of the soil, and include smectite, montmorillonite, nontronite, vermiculite, illite and chlorite. Generally, the more of these minerals that are present in the soil, the greater the expansive potential. However, these expansive effects may become diluted by the presence of other non-swelling minerals such as quartz and carbonate (Kemp *et al*., 2005).

The key aspect of expansive soils behaviour is the soil vulnerability of water-induced volume change. When soils with a high expansive potential are present, they will usually not cause a problem as long as their water content remains relatively constant. This is largely controlled by (Houston *et al*., 2011):

- soil properties, e.g. mineralogy;
- suction and water conditions;
- water content variations, both temporally and spatially;
- geometry and stiffness of a structure, in particular its foundation.

In a partially saturated soil, changes in water content, or suction (increasing strength of the soil due to negative pore water pressures), significantly increase the chances of damage occurring. Changes in soil suction occur due to water movement through the soil due to evaporation, transpiration or recharge, which are often significantly influenced by interaction with trees through response to dry/wet periods of weather (Biddle, 2001). In a fully saturated soil, the shrink–swell behaviour is controlled by the clay mineralogy.

33.4.1 Mineralogical aspect of expansive soils

Clay particles are very small and their shape is determined by the arrangement of the thin crystal lattice layers that they form, along with many other elements which can become incorporated into the clay mineral structure (hydrogen, sodium, calcium, magnesium, sulfur). The presence and abundance of these dissolved ions can have a large impact on the behaviour of the clay minerals. In an expansive clay, the molecular structure and arrangement of these clay crystal sheets have a particular affinity to attract and hold water molecules between the crystalline layers in a strongly bonded 'sandwich'. Because of the electrical dipole structure of water molecules, they have an electro-chemical attraction to the microscopic clay sheets. The mechanism by which these molecules become attached to each other is called adsorption. The clay mineral montmorillonite, part of the smectite family, can adsorb very large amounts of water molecules between its clay sheets, and therefore has a large shrink–swell potential. For further details of mineralogy of clay minerals and their influence of engineering properties of soils, see Mitchell and Soga (2005).

When potentially expansive soils become saturated, more water molecules are absorbed between the clay sheets, causing the bulk volume of the soil to increase, or swell. This same process weakens the inter-clay bonds and causes a reduction in the strength of the soil. When water is removed, by evaporation or gravitational forces, the water between the clay sheets is released, causing the overall volume of the soil to decrease, or shrink. As this occurs, features such as voids or desiccation cracks can develop.

Potentially expansive soils are initially identified by undertaking particle size analyses to determine the percentage of fine particles in a sample. Clay-sized particles are considered to be less than 2 μm (although this value varies slightly throughout the world), but the difference between clays and silts is more to do with origin and particle shape. Silt particles (generally comprising quartz particles) are products of mechanical erosion, whereas clay particles are products of chemical weathering and are characterised by their sheet structure and composition.

33.4.2 Changes to effective stress and role of suctions

Following any reduction in *total* stress, deformations will take place in the ground. A distinction can be made between (i) an immediate, but time-dependent elastic rebound, and (ii) swelling due to effective stress changes. In soils, as in rocks, rebound can be an important deformation process which encourages stress relief fractures and zones of secondary permeability which can localise delayed swelling. The amount of deformation depends on the undrained stiffness of the soil, which is equivalent to the modulus of elasticity for the soil, as reflected by its Young's modulus and Poisson's ratio. Subsequent swelling requires an *effective* stress decrease, and a movement of fluid into a geological formation or soil. The magnitude of strains associated with these processes depends on the drained stiffness, the extent of the stress change, the resulting water pressures in the soil or rock, and the new boundary conditions. The rate of volume change depends on the compressibility, expansibility and hydraulic conductivity of the sediment and surrounding materials. In stiff homogeneous materials with a low hydraulic conductivity, several decades may be necessary to complete the process.

Accurate laboratory measurements of the controlling elastic properties at small strains in both rebound and swelling (i.e. before yield takes place) are difficult, largely because of sampling disturbance (Burland, 1989). Further discussion of these difficulties, states of stress, and the other important concepts of consolidation/swelling in soils are treated in detail by many standard soil engineering texts (Powrie, 2004; Atkinson, 2007) – see also Section 2 *Fundamental principles* of this manual.

Shrinkage by evaporation is similarly accompanied by a reduction in water pressure and development of negative capillary pressures. Deformation follows the same principles of effective stress. However Bishop *et al*. (1975) have shown by laboratory studies that the degree of saturation of unconfined dried clay samples at a given water content was less than for a similar sample consolidated in a triaxial test to the same water content, i.e. there was some intake of air which affected both the modulus and strength of the soil. This process leads to a void ratio which is higher than for a clay consolidated to the same water content by simply increasing the confining load. Such a soil thus becomes inherently unstable and, if re-wetted, may collapse. Subsequent laboratory tests on partially saturated soils have shown that depending on their *in situ* stress conditions and fabric, some samples may also first swell and then collapse (Alonso *et al*., 1990). The processes of shrinkage due to evaporation have also been reviewed in detail using effective stress concepts by Sridharan and Venkatappa (1971).

33.4.3 Seasonal variations in water content

The seasonal volumetric behaviour of a desiccated soil is complex and this increases with severity of the shrinkage phenomena. This is reflected by the vertical *in situ* suction profile, water content profile and the degree of saturation (see **Figure 33.4**).

The relative values of suction depend on the composition of the soil, particularly its particle size and clay mineral content. The hydraulic conductivity of a soil may also vary both seasonally and over longer timescales. Secondary permeabilities can be induced through fabric changes, tension cracking and shallow shear failure during the swelling and shrinkage process which may influence subsequent moisture movements. For example, Scott *et al*. (1986) have shown in a microfabric study of clay soils that compression (swelling) cracks tended to run parallel to ground contours and dip into the slope at around 60º, and could usually be distinguished from shrinkage cracks which were randomly distributed. In the London Clay soils studied, for example, they found that the ratio between shrinkage and swelling discontinuities was about 2:1. Although not discussed, it seems likely that the nature and distribution of

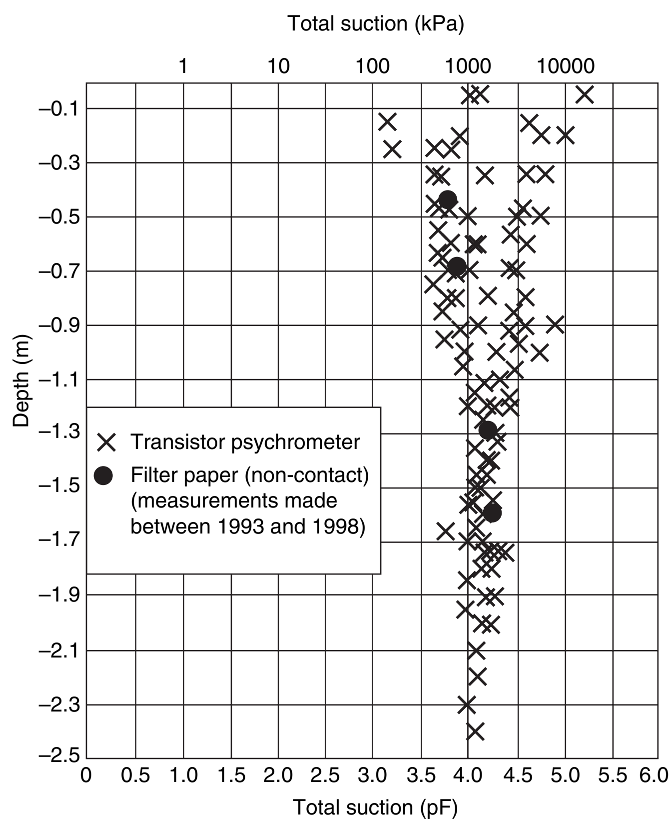

Figure 33.4 Examples of total suction profile
Reproduced from Fityus *et al*. (2004), with kind permission from ASCE

these discontinuities will also influence bulk volumetric seasonal strains.

Expansive soil problems typically occur due to water content changes in the upper few metres, with deep-seated heave being rare (Nelson and Miller, 1992). The water content in these upper layers is significantly influenced by climatic and environmental factors and is generally termed the zone of seasonal fluctuations, or active zone, as shown in **Figure 33.5**.

In the active zone, negative pore water pressures exist. However, if excess water is added to the surface or if evapotranspiration is eliminated, then water contents increase and heave will occur. Migration of water through the zone is also influenced by temperature, as shown in **Figure 33.5**, with further details provided by Nelson *et al*. (2001). Thus it is important to determine the depth of the active zone during a site investigation. This can vary significantly with different climatic conditions – it may be 5–6 m in some countries, but typically in the UK it is 1.5–2 m (Biddle, 2001). If the drying is greater than the rehydration, then the depth of this zone will increase, with 3–4 m having been observed in some cases in London Clay (Biddle, 2001). These effects are likely to become more significant with climate change.

The term 'active zone' can have different meanings. Nelson *et al*. (2001) provide four definitions for clarity:

1. *Active zone* The zone of soil that contributes to soil expansion at any particular time.
2. *Zone of seasonal moisture fluctuation* The zone in which water content changes due to climatic changes at the ground surface.
3. *Depth of wetting* The depth that water contents have reached owing to the introduction of water from external sources.
4. *Depth of potential heave* The depth at which the overburden vertical stress equals or exceeds the swelling pressure of the soil. This is the maximum depth of the active zone.

The depth of wetting is particularly important as it is used to estimate heave by integrating the strain produced over the zone in which water contents change (Walsh *et al*., 2009). Details of how this can be achieved and the relative merits of regional and site-specific approaches are considered in detail for a post-development profile by Walsh *et al*. (2009), with further discussion presented by Nelson *et al*. (2011); Aguirre (2011); and Walsh *et al*. (2011).

33.5 Engineering issues

As has been previously stated, many towns, cities, transport routes, services and buildings are founded on expansive soils. These may be solid (bedrock) geological strata in a weathered or unweathered condition, or superficial (drift) geological strata such as glacial or alluvial material, also in a weathered or unweathered condition. These materials constitute a significant

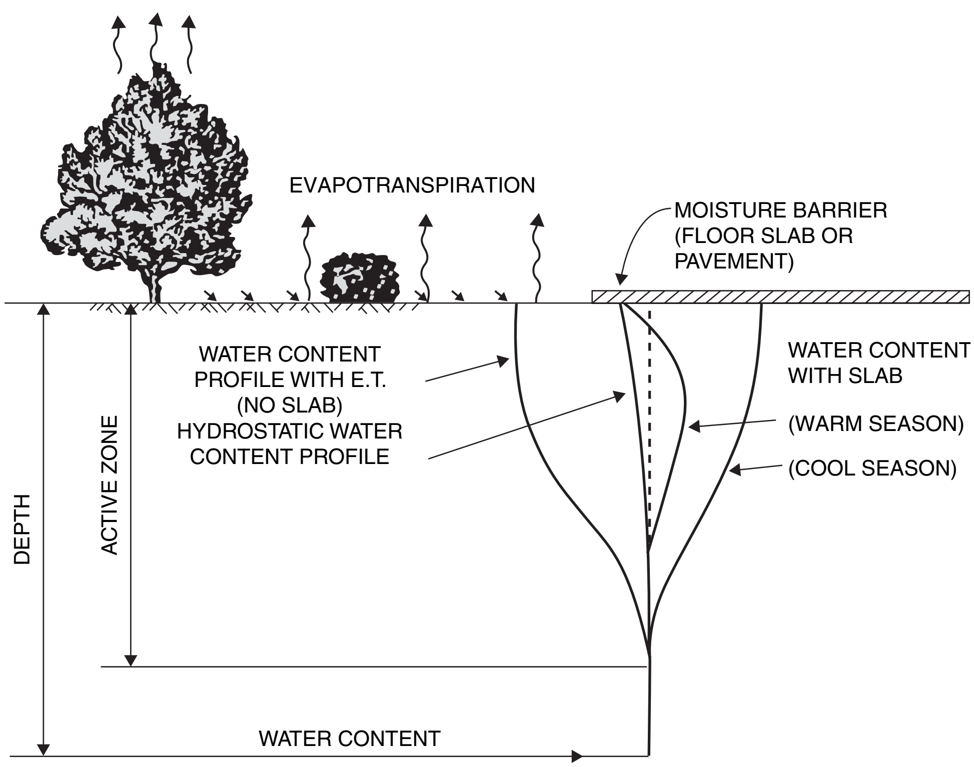

Figure 33.5 Water content profiles in the active zone
Reproduced from Nelson and Miller (1992); John Wiley & Sons, Inc

hazard to engineering construction in terms of their ability to swell or shrink, usually caused by seasonal changes in moisture content. Superimposed on these widespread climatic influences are local ones, such as tree roots and leakage from water supply pipes and drains. The swelling of shrinkable clay soils after trees have been removed can produce either very large uplifts or very large pressures (if confined), and the ground's recovery can continue over a period of many years (Cheney, 1988). It is the differential, rather than the total, movement of the foundation, or superstructure, that causes major structural damage. The structures most affected by expansive soils include the foundations and walls of residential and other low-rise buildings, pipelines, pylons, pavements and shallow services. Frequently, these structures only receive a cursory site investigation, if any. It is usually sometime after construction that problems come to light. Damage can occur within a few months of construction, develop slowly over a period of 3–5 years, or remain hidden until something happens that changes the water content of the soil.

Houston *et al*. (2011) examined the type of wetting that occurs in response to irrigation patterns. They observed that deeper wetting was common with irrigation of heavily turfed areas, and that if ponding of water occurred at the surface, there was more likely to be greater distress to buildings through differential movements. Walsh *et al*. (2009) also note that when heave is deep seated, differential movements are less significant than when the source of heave is at shallower depths.

The structures most susceptible to damage caused by expansive soils are usually lightweight in construction. Houses, pavements and shallow services are especially vulnerable because they are less able to suppress differential movements than heavier multi-story structures. For more information about design parameters and construction techniques for housing and pavements, reference should be made to:

- *NHBC Standards: Building near trees* (NHBC, 2011a)
- *Preventing foundation failures in new dwellings* (NHBC, 1988)
- *Planning Policy Guidance Note 14: Development on unstable land: Annex 2: subsidence and planning* (DTLR, 2002)
- *BRE Digests 240– 242: Low-rise buildings on shrinkable clay soils* (BRE, 1993a)
- *BRE Digest 298: The influence of trees on house foundations in clay soils* (BRE, 1999)
- *BRE Digest 412: The significance of desiccation* (BRE, 1996)
- *Criteria for selection and design of residential slabs-on-ground* (BRAB, 1968)
- *Evaluation and control of expansive soils* (TRB, 1985).

In many respects, engineering in expansive soils is still based on experience and soil characterisation, and so is often perceived as difficult and expensive (especially for lightweight structures). Engineers use local knowledge and empirically derived procedures, although considerable research has been done on expansive soils – for instance, the database on performance (Houston *et al*., 2011). However, through careful consideration of key aspects associated with expansive soils, problems and difficulties can be dealt with in a cost effective way.

Two major factors must be identified in the characterisation of a site where a potentially expansive soil exists:

- the properties of the soil (e.g. mineralogy, soil water chemistry, suction, soils fabric);
- environmental conditions that can contribute to changes in water content of the soil, e.g. water conditions and their variations (climate, drainage, vegetation, permeability, temperature), and stress conditions (history and *in situ* conditions, loading and soil profile).

Normal non-expansive site investigations are often not adequate and a more extensive examination is required to provide sufficient information. This may involve specialist test programs, even for relatively lightweight structures (Nelson and Miller, 1992). Although there are a number of methods available to identify expansive soils, each with their relative merits, there are no universally reliable ones. Moreover, expansiveness has no direct measure and so it is necessary to make comparisons, measured under known conditions, as a means to express expansive behaviour (Gourley *et al*., 1993). However, the stages of investigation needed for expansive soils follow those used for any site (see Section 4 *Site investigation* for further details).

33.5.1 Investigation and assessment

It is important to recognise the existence, and understand the potential problems, of expansive soils early on during site investigation and laboratory testing, to ensure that the correct design strategy is adopted before costly remedial measures are required. It is equally important that investigations determine the extent of the active zone.

Despite the proliferation of test methods for determining shrinkage or swelling properties, they are rarely employed in the course of routine site investigations in the UK. Further details of tests commonly employed around the world are given by Chen (1988) and Nelson and Miller (1992). This means that few datasets are available for databasing the directly measured shrink–swell properties of the major clay formations, and reliance has to be placed on estimates based on index parameters, such as liquid limit, plasticity index and density (Reeve *et al*., 1980; Holtz and Kovacs, 1981; Oloo *et al*., 1987). Such empirical correlations may be based on a small dataset, using a specific test method, and at only a small number of sites. Variation of the test method would probably lead to errors in the correlation. The reason for the lack of direct shrink–swell test data is that few engineering applications have a perceived requirement for these data for design or construction.

33.5.1.1 Site investigation

A key difficulty with expansive soils is that they often exhibit significant variability from one location to another (i.e. spatial variability). These proper, adequate, site investigations in areas of potentially expansive soil are often worth the cost. Essential to the investigation of any expansive soils is a good knowledge of local geology: the use of maps provides a framework for this. These maps are particularly useful when constructing transportation networks. In some countries such as the US, mapping includes identification of expansive soil potential (Nelson and Miller, 1992). As with any site investigation, field observations and reconnaissance can provide valuable data of the extent and nature of expansive soils and their associated problems. Some key features may be observed locally and important observations include:

1. *Soil characteristics*
   - spacing and width of wide or deep shrinkage cracks;
   - high dry strength and low wet strength – high plasticity soil;
   - stickiness and low trafficability when wet;
   - shear surfaces have glazed or shiny appearance.
2. *Geology and topography*
   - undulating topography;
   - evidence of low permeability by surface drainage and infiltration features.
3. *Environmental conditions*
   - vegetation type;
   - climate.

Sampling in expansive soils is generally done in the same way as for conventional soils, with care taken to minimise disturbances through, for example, water content changes or poor control during transportation. Further details are provided in Section 4 *Site investigation* of this manual, and an overview of practices specifically used for expansive soils in other countries is provided by Chen (1988) and Nelson and Miller (1992). However, the depth and frequency of sampling may need to be increased in expansive areas due to their high spatial variability.

33.5.1.2 *In situ* testing

A suite of different field tests can be used to evaluate expansive soils and these include:

- soil suction measurements using thermocouple psychrometers, tensiometers or filter paper methods;
- *in situ* density and moisture tests;
- settlement and heave monitoring;
- piezometers or observations wells;
- penetration resistance;
- pressuremeters and dilatometers;
- geophysical methods.

Expansive soils can be tested in the field using methods that rely on empirical correlation such as the standard penetration test (SPT) or the cone penetration test (CPT) to infer soil strength parameters (Clayton *et al*., 1995). Initial effective stresses can be estimated using a psychrometer (Fredlund and Rahardjo, 1993) or a suction probe (Gourley *et al*., 1994) which will measure the soil suction. The undrained shear strength of the soil can be determined using a shear vane (Bjerrum, 1967). The stiffness parameters of the soil can be determined using a plate loading test (BSI, 1999), along with its strength and compressibility. Other tests include the pressuremeter and the dilatometer (ASTM, 2010) which measure strength, stiffness and compressibility parameters.

Seismic test apparatus uses the transmission of elastic waves through the ground in order to determine its density and elastic properties (see Chapter 45 *Geophysical exploration and remote sensing*). Electrical resistivity methods have also shown promise as a method to determine swell pressure and shrinkage of expansive soils. Resistivity was found to increase as both swell pressure and shrinkage increased (Zha *et al*., 2006). More recently, Jones *et al*. (2009) successfully monitored tree-induced subsidence in London Clay using electrical resistivity imaging.

Monitoring should also be considered and a number of approaches can be used which are common with non-expansive soils. Key methods are: settlement and heave monitoring for volume change, and piezometers for pore water changes. Monitoring of water content profiles over several wet and dry seasons are used to establish the extent of the active zone (Nelson *et al*., 2001). In cases where the soil is not uniform or several strata exist, a correction can be applied using the liquidity index. Nelson and Miller (1992) provide an example of this calculation.

Examples of monitoring associated with expansive soils are provided throughout literature. Examples include Fityus *et al*. (2004), where a site near Newcastle, Australia, was instrumented, and soil water and suction profiles together with ground movements were determined over a period (1993–2000). In addition, the work of the BRE at their London Clay site near Chattenden, Kent, provides details of similar monitoring regimes over a number of years (Crilly and Driscoll, 2000; Driscoll and Chown, 2001). Stable benchmarks are important for any monitoring in expansive soils, and design details and installation instructions are given in many papers, e.g. Chao *et al*. (2006).

Further details can be found in Sections 4 *Site investigation* and 9 *Construction verification* of this manual. For specific discussions in the context of expansive soils, see Chen (1988), and Nelson and Miller (1992).

33.5.1.3 Laboratory testing

Considerable research work has been carried out on behalf of the oil and mining industries, especially in the US, on the swelling behaviour of ‘compact’ clays and mudrocks, in particular clay shales. Swelling pressure has caused damage in tunnels (Madsen, 1979), as is the case – usually at great depths – in the mining industry. In the oil industry, the swelling of shales and ‘compact’ clays in borehole and well linings has been a topic of interest. Laboratory test methods developed differ considerably from those applied by the civil engineering industry, and tend to duplicate the particular phenomena causing problems. For example, the moisture activity index test (Huang *et al*., 1986) duplicates changes in relative humidity in the air passing through mine tunnels, and consequent swelling of the tunnel lining. However, the confined swelling pressure test is relatively universal. As shrinkage is a near-surface phenomenon in the UK, much work has been done by the soil survey and agricultural organisations. Reeve *et al*. (1980) describe the determination of shrinkage potential for a variety of soils classified on a pedological basis.

For geotechnical purposes, a suite of different tests can be used to identify expansive soils and include Atterberg limits, shrinkage limits, mineralogical tests such as X-ray diffraction, swell tests and suction measurements (see Nelson and Miller, 1992 for further details). Undisturbed samples are normally used for one-dimensional response to wetting tests. However, it should be noted that when conducting swell tests in the laboratory, it is important to distinguish between swelling in compacted, undisturbed and reconstituted samples, which occurs due to significant differences in their respective fabrics.

Swell–shrink tests

Swelling tests may be broadly divided into those tests attempting to measure the deformation or strain resulting from swelling, and those which attempt to measure the stress, or pressure, required to prevent deformation due to swelling. These two types are referred to here as swelling strain and swelling pressure tests, respectively. Swelling strain tests may be linear, i.e. one-dimensional (1D) or volumetric, i.e. three-dimensional (3D). Swelling pressure tests are almost always one-dimensional and traditionally used oedometer-type testing arrangements (Fityus *et al*., 2005). However, shrinkage tests deal solely with the measurement of shrinkage strain in either 1D or 3D.

Standards do exist for shrink–swell tests but these do not cover all the methods in use internationally. Like many 'index'-type soils tests, some shrink–swell tests are based on practical needs and tend to be rather crude and unreliable. Whilst measurement of water content is easily achieved with some accuracy, the measurement of the volume change of a clay soil specimen is not, particularly in the case of shrinkage. Solutions to this problem have been found by the measurement of volume change in only one dimension, or by immersion of the specimen in a non-penetrating liquid such as mercury. However, the use of mercury in this way is far from ideal. Measurement of volume change in the case of swelling, where the specimen is assumed to be saturated, is only slightly less problematic. In this case, dimensional changes are required to be made whilst the specimen is immersed in water. This introduces the problem of either immersed displacement transducers or sealed joints for non-immersed transducers.

Nelson and Miller (1992) provide a detailed account of various swell and heave tests (with the oedometer being the most commonly used) which are often developed based on geographic regions with specific expansive soil problems. However, they can be considered applicable in general situations (Fityus *et al*., 2005). These tests determine the applied stress required to prevent swelling strain when a specimen is subjected to flooding. The ability to do this is enhanced by computer control, or by at least some form of feedback control. The determination of swelling pressure should not be confused with the determination of rebound strain under consolidation stresses in the oedometer test. In the latter case, the slope of the rebound part of the familiar voids ratio versus applied stress (e–log p') curve is referred to as the swelling index (C_s); that is the rebound or decompressional equivalent of the compression index (C_c). It is common, however, for measured swell potential to be low to medium when soil units across a region have high potential; this is the result of natural soil variability (Houston *et al*., 2011).

Mineralogical testing

In addition to the traditional approaches used, several parameters have been investigated which are either wholly or largely dependent on clay mineralogy. These are surface area (Farrar and Coleman, 1967), dielectric dispersion (Basu and Arulanandan, 1974), and disjoining pressure (Derjaguin and Churaev, 1987). The factors affecting swelling of very compact or heavily overconsolidated clays and clay shales may differ from those affecting normally consolidated or weathered clays. Physicochemical and diagenetic bonding forces probably dominate in these materials, whereas capillary forces are negligible. It is likely that the distance between clay platelets, the ionic concentration of pore fluids, and fluids used in laboratory tests relative to the clay mineral activity of such materials, are the key factors in swelling. Traditional concepts of Darcian permeability and pore water pressure are thrown into doubt in these compact clays and clay shales. Diffusion may be the principal mode of fluid movement in these very low permeability clays.

Use index tests

The volume change potential (VCP) (also known as the potential volume change, PVC) of a soil is the relative change in volume to be expected with changes in soil water content, and is reflected by shrinking and swelling of the ground; in other words, the extent to which the soil shrinks as it dries out, or swells when it gets wet. However, despite the various test methods available for determining these two phenomena, e.g. BS 1377, 1990: Part 2, Tests 6.3 and 6.4 *Shrinkage Limit* and Test 6.5 *Linear Shrinkage* and Part 5, Test 4 *Swelling Pressure* (BSI, 1990), they are rarely employed in the course of routine site investigations in the UK. Hence few data are available for databasing the directly measured shrink–swell properties of the major clay formations. Consequently, reliance is placed on estimates based on index parameters, namely, liquid limit, plastic limit, plasticity index, and density (Reeve *et al.*, 1980; Holtz and Kovacs, 1981; and Oloo *et al.*, 1987). No consideration has been given to the saturation state of the soil and therefore to the effective stress or pore water pressures within it.

The most widely used parameter for determining the shrinkage and swelling potential of a soil is the plasticity index (I_p). Such plasticity parameters, being based on remoulded specimens, cannot precisely predict the shrink–swell behaviour of an *in situ* soil. However, they do follow properly laid down procedures, being performed under reproducible conditions to internationally recognised standards (Jones, 1999). A 'modified plasticity index' (I'_p) is proposed in the Building Research Establishment Digest 240 (BRE, 1993a) for use where the particle size data, specifically the fraction passing through a 425 μm sieve, is known or can be assumed as 100% passing (**Table 33.1**).

The modified I'_p takes into account the whole sample and not just the fines fraction; it therefore gives a better indication of the 'real' plasticity value of an engineering soil and eliminates discrepancies due to particle size, for example in glacial till. This compares with a classification produced by the National House-Building Council which forms the basis of the NHBC 'foundation depth' tables (**Table 33.2**), which uses the same modified I'_p approach as presented in **Table 33.1**.

The concept of 'effective plasticity index' has been described (BRAB, 1968) to deal with multi-layered soils of different plasticity index.

Ultimately, swelling and shrinkage *potential* may be considered to be the ultimate capability of a soil to swell and shrink, but this potential is not necessarily realised in a given moisture change situation. These do not therefore represent the fundamental properties of a soil. However, potential may be described differently. For example, swelling potential is described by Basu and Arulanandan (1974) as 'the ability and degree to which swelling is realised under given conditions'. So there is already some confusion in terminology. Oloo *et al*. (1987) differentiate between intrinsic expansiveness (swell) and heave. They define intrinsic expansiveness as that property which 'relates change in water content, and thus change in volume, to the suction change' of a clay soil. Thus a soil of high intrinsic expansiveness will exhibit a large water content or volume change compared with one of low intrinsic expansiveness for a given suction change – all other things being equal. Oloo *et al*. (1987) state that no procedure has been developed to measure this property. Swell is defined as 'a measure of the volume strain, or axial strain, in a soil under a particular set of stress and suction conditions'. Heave is defined as 'the displacement of a point in the soil due to suction and stress changes interacting with the intrinsic expansiveness'. Heave is not a soil property.

Overall, there are many methods of testing for the shrinkage and swelling properties of clay soils. These methods are covered in detail in Jones (1999), where the pros and cons of each method are discussed and the reasons for the selection and rejection of methods is determined. Further evaluation of these tests is also provided by Fityus *et al*. (2005).

I'_p (%)	Volume change potential
> 60	Very high
40–60	High
20–40	Medium
< 20	Low

Note: $I'_p = I_p \times (\%{<}425\,\mu m) / 100\%$

Table 33.1 Classification for shrink–swell clay soils
Data taken from BRE (1993a)

I'_p (%)	Volume change potential
> 40	High
20–40	Medium
10–20	Low

Table 33.2 Classification for shrink–swell clay soils
Data taken from NHBC (2011a)

33.5.2 Shrink/swell predictions

Common to all geotechnical predictions of volume change is the need to define initial and final *in situ* stress state conditions. In addition this requires characterisation of the stress–strain behaviour of each soil profile. Initial stress states and constitutive properties can be evaluated using a suite of approaches (highlighted by many texts, e.g. Fredlund and Rahardjo,1993; Powrie, 2004) but it is the final stress condition that must usually be assumed. Guidelines are presented by Nelson and Miller (1992), with calculations based on knowledge of effective overburden stress (i.e. the increment of stress due to applied load and soil suction). However, each situation requires engineering judgement and consideration of environmental conditions.

Details of constitutive relationships for expansive soils have been reviewed and a useful description of these is given by Nelson and Miller (1992). These include unsaturated soil models dealing with matric and osmotic suctions. A detailed account of this, the theoretical basic, associated models used to predict partially saturated soils behaviour, together with test methods used to determine key soil parameters, is provided in Fredlund and Rahardjo (1993) and Fredlund (2006).

Overall prediction methods can be grouped into three broad categories: theoretical, semiempirical and empirical. They all rely on testing methods; particular care must be taken with empirical methods which are only valid within the bounds of the soil type, environment and engineering application for which they were developed.

A number of heave predictions are available that are based on oedometer or suction tests, and Nelson and Miller (1992) provide a detailed account of these, together with examples of associated predictions. For example Nelson *et al*. (2010) provide an illustration using free-field heave predictions and their use in foundation design, as well as methods for prediction heave rates.

33.5.2.1 Oedometer-based methods

Oedometer-based tests include one-dimensional and double oedometer tests (developed by Jennings and Knight, 1957).

Double oedometer tests consist of two near-identical undisturbed samples, one loaded at its natural water content and the other inundated under a small load and then loaded under saturated conditions. The use of the oedometer has distinct advantages due to familiarity amongst geotechnical engineers.

Tests can be conducted as free swell tests where swelling is allowed to occur at a pre-determined pressure after water is added. The swell pressure is then defined as the pressure required to recompress the swollen sample to its pre-swollen volume. These tests, however, suffer the limitation that volume change can occur and that hysteresis is incorporated into the estimation of the *in situ* state. An alternative approach that overcomes these problems involves inundating a sample placed in the oedometer and preventing it from swelling. The swell pressure is then the maximum applied stress required to achieve a constant volume. Typical results from these tests are shown in **Figure 33.6**, with σ_0' representing the stress when inundation occurred and σ_s' representing the stress equated to swelling pressure.

The constant volume test may overcome the difficulties of the free swell test, but as a result is more vulnerable to sample disturbance. To account for sample disturbance, Rao *et al*. (1988) and Fredlund and Rahardjo (1993) suggest simplifications to facilitate predictions using parameters measured by constant volume oedometer tests (pressures increase during swelling to maintain constant volume) using established techniques. This is illustrated in **Figure 33.7**.

Fityus *et al*. (2005) questioned this approach and considered that specialist apparatus not normally used in standard geotechnical engineering testing laboratories is needed to achieve meaningful results. However, not all authors agree, with Nelson and Miller (1992) believing good quality data and predictions can be obtained with such an approach. Moreover, a number of disadvantages exist, as tests where the specimen is fully wetted are conservative, as full saturation is not often reached in the field (Houston *et al*., 2011). Thus, swell tests based on submerged samples at the level of stress of interest will overpredict heave. The effect of partial wetting may be as important as the depth to which wetting has occurred (Fredlund *et al*., 2006).

33.5.2.2 Suction-based tests

Suction tests are used to predict soil response in much the same manner as with saturated effective stress changes. Various methods have been developed, e.g. the US Army Corps of Engineers Waterways Experiment Station (WES) method or the clod method, details of which (including advantages and limitations) can be found in Nelson and Miller (1992). Fredlund and Hung (2001) have subsequently developed suction-based predictions to evaluate volume changes from both environmental and vegetation changes – and they provide useful outline example calculations.

Nelson and Miller (1992) suggest that with careful sampling and testing it is possible to predict heave within a few centimetres. However, it is essential that the testing is conducted within the expected stress range in the field. Furthermore, experimental studies involving direct measurement of partially saturated properties is expensive and often time-consuming. For example, Chandler *et al*. (1992) provide details of suction measurements using the filter paper method, highlighting the need for careful calibration as results can be affected by temperature fluctuations, particle entrainment in the filter paper during testing, and hysteresis effects. Such approaches have a number of advantages as a means to estimate soil suction and hence suction profiles (see **Figure 33.4**).

For this reason, increasingly numerical and semiempirical methods use the soil–water characteristic curves (SWCCs) (Puppala *et al*., 2006). The SWCCs describe the relationship

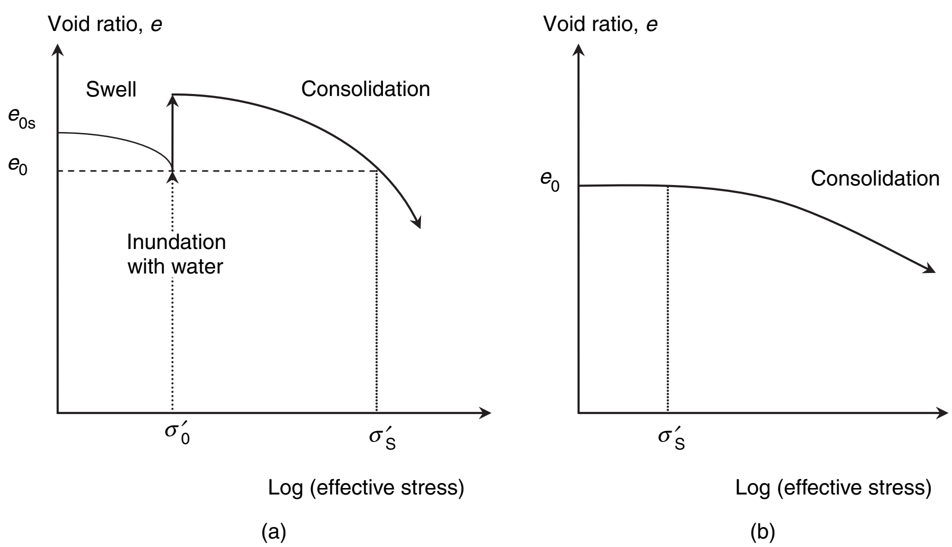

Figure 33.6 Typical oedometer swell test curves: (a) an illustration of a free swell test result; (b) an illustration of constant volume test results

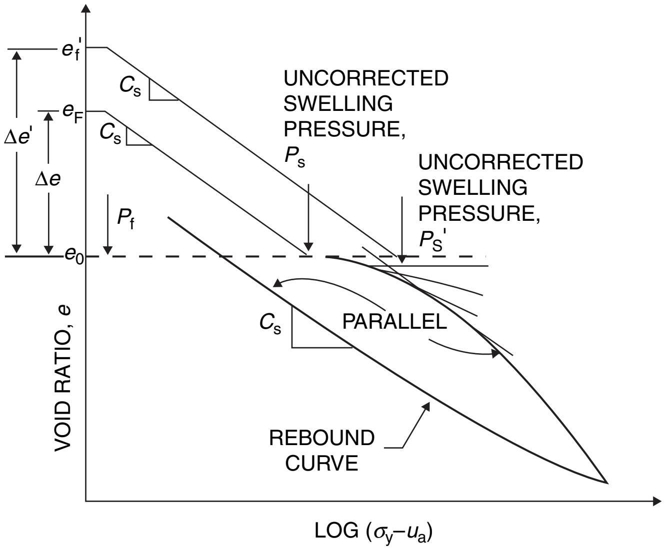

Figure 33.7 One-dimensional oedometer test results showing effect of sampling disturbance. Note: C_s is swell index; ($\sigma_y - u_a$) is overburden pressure; P_f is final stress state; e_f is final void ratio, and e_f' is final void ratio corresponding to corrected swell pressure, P_s'
Reproduced from Rao *et al.* (1988), with kind permission from ASCE

between water content (either gravimetric or volumetric) and soil suction. Alternatively, they can be used to describe the relationship between the degree of saturation and soil suction. A more detailed discussion and examples of typical SWCCs are also provided in Chapter 30 *Tropical soils*.

Only a limited number of investigations have been undertaken on expansive soils with Ng *et al*. (2000), Likos *et al*. (2003) and Miao *et al*. (2006) providing some example of these. Puppala *et al*. (2006) details SWCCs for both treated and untreated expansive soils. Further details of this are provided by Fredlund and Rahardjo (1993) with Nelson and Miller (1992) providing details in the context of expansive soils. However, it should be noted that suction measurements are subject to errors that can be substantial (Walsh *et al*. 2009).

Empirically-based methods are still common in geotechnical engineering (Houston *et al*., 2011). Heave is often estimated by the integration of strain over the zone in which the water contents change. However, uncertainty occurs and arises from three sources (Walsh *et al*., 2009):

1. the depth over which the wetting will occur;
2. the swell properties of the soil;
3. the initial and final suction over the depth of wetting.

Furthermore, care is needed with all models used, as small changes in input parameters can lead to significant changes in an estimated soil response. The real challenge is, therefore, to understand the relationship between soil–water stress level and volume changes, coupled with a prediction of the actual depth and degree of wetting that will occur in the field. Both are related to soil properties and control of site water (Houston *et al*., 2011).

Houston *et al*. (2011) compared predictions from a number of forensic studies from field and laboratory investigations in arid/semi-arid areas to those undertaken using numerical approaches (in this case, the simple 1D and 2D unsaturated flow model), with details of site drainage and landscape practices also considered. Comparisons were made after one year; they concluded that drainage conditions were the more important factor in the prediction of foundation problems. This study revealed that the effects of poor drainage and roof run-off ponding near a structure is the worst case scenario. Uncontrolled drainage and water ponding near foundations led to significant suction reduction to greater depths (0.8 m was found after one year), resulting in differential soil swell and foundation movement (see **Figure 33.8**).

33.5.2.3 Numerical approaches

1D simulations also dominate numerical studies, as unsaturated flow solutions are sensitive to accurate and detailed simulation of surface flux conditions, thus requiring an extremely tight mesh and time steps (Houston *et al*., 2011). This may result in very lengthy run times of several months, even for 1D assessments (Dye *et al*., 2011). However, Xiao *et al*. (2011) demonstrated how numerical simulations could be used to assess pile–soil interactions, providing an effective way to undertake sensitivity analysis, but noted that many parameters are needed when undertaking numerical assessments.

33.5.3 Characterisation

Many attempts have been made to find a universally applicable system for the classification of shrinking and swelling in order to characterise an expansive soil. Some have even attempted to produce a unified swelling potential index using commonly used indices (e.g. Sridharan and Prakash, 2000; Kariuki and van der Meer, 2004; Yilmaz, 2006) or from specific surface areas (Yukselen-Aksoy and Kaya, 2010), but these are yet to be adopted. Examples of various schemes commonly used around the world are illustrated in **Figure 33.9**. The various schemes that have been developed lack standard definitions of swell potential, since both sample conditions and testing factors vary over a wide range of values (Nelson and Miller, 1992).

33.5.3.1 Classification schemes

Most classification schemes give a qualitative expansion rating, e.g. high or critical. The different classification schemes can be categorised into four groups, depending on which method they employ to determine their results. These are:

1. free swell (see Holtz and Gibbs, 1956);
2. heave potential (see Vijayvergiya and Sullivan, 1974; Snethen *et al*., 1977);
3. degree of expansiveness (see US Federal Housing Administration (FHA), 1965; Chen, 1988);
4. shrinkage potential (see Altmeyer, 1956; Holtz and Kovacs, 1981).

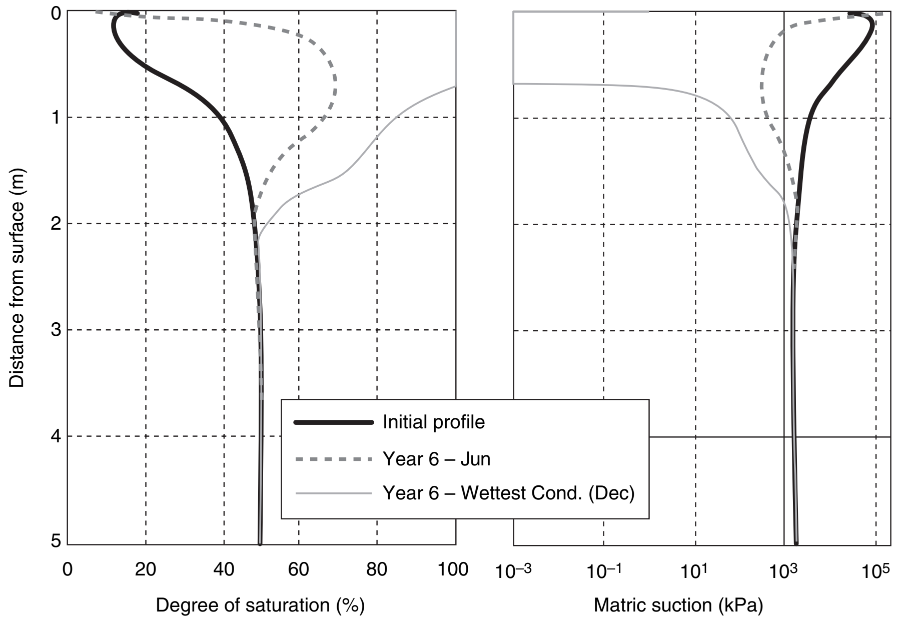

Figure 33.8 Profile for 1 year of roof run-off water ponding next to foundation after 6 years of desert landscape. Wettest and driest conditions in 1-D
Reproduced from Houston *et al.* (2011), with kind permission from ASCE

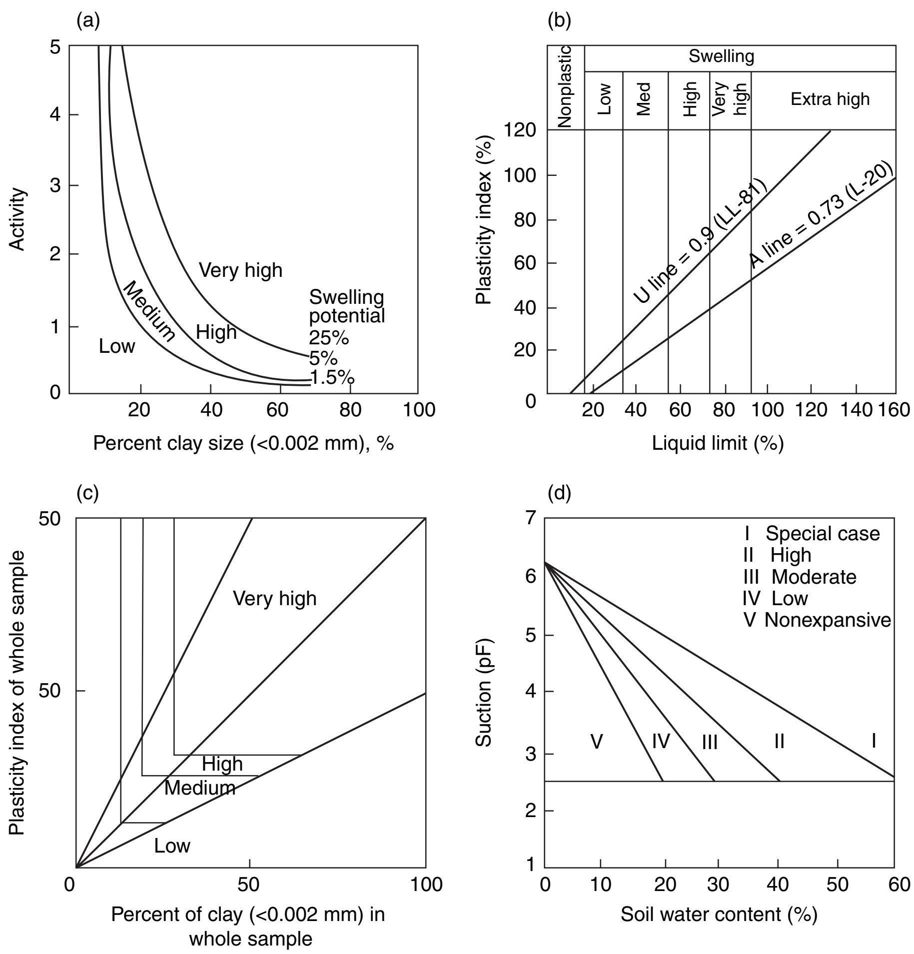

Figure 33.9 Commonly used criteria for determining swell potential from across the world
Reproduced from Yilmaz (2006), with permission from Elsevier

Since liquid limit and swelling of clays both depend on the amount of water a clay tries to imbibe, it is not surprising that they are related. Chen (1988) suggested that a relationship between the swelling potential of clay and its plasticity index can be established. While it may be true that high swelling soil will manifest high index properties, the converse is not always true.

Other classification schemes relate to expansion potential, based on the Skempton 'activity' plot (Skempton, 1953) and its development by Williams and Donaldson (1980) from Van der Merwe (1964). Details are described in Taylor and Smith (1986) with respect to various UK clay mudstone formations.

A host of schemes have been put forward for estimating shrink–swell, particularly in the US (see Chen, 1988; Nelson and Miller, 1992), most of which use swelling and suction as their basis (Snethen, 1984). Sarman *et al*. (1994) concluded that swelling was not related solely to clay mineral type, but also to pore-morphology. It was found that samples showing high swelling had a large pore volume combined with a high percentage of small-sized pores. The high swelling was attributed to the samples' ability to absorb and adsorb water. It was found that correlations between swelling and other parameters were unsuccessful.

With all classification schemes only indications of expansion are obtained with, in reality, field conditions varying considerably. Such ratings can be of little use unless the user is familiar with the soil type and the test conditions used to develop the ratings. Ratings themselves may be misleading and can, if used with design options outside the region where the rating was established, cause significant difficulties (Nelson and Miller, 1992). Classifications, therefore, should only be considered to provide an indication of potential expansive problems, and further testing is needed. If such schemes are used as a basis of design, the result is either over-conservative solutions or inadequate construction (Nelson and Miller, 1992).

33.5.3.2 UK approach

Whilst much study has been carried out worldwide to infer swelling and shrinkage behaviour from soil index properties such as plasticity (see section 33.5.1.1.3), few direct data are available in UK geotechnical databases (Hobbs *et al*., 1998). Two schemes that are commonly used within the UK are based on the BRE and NHBC schemes.

Volume change potential has more recently been defined for overconsolidated clays in terms of a modified plasticity index term (I'_p) by Building Research Establishment Digest 240 (BRE, 1993a) – see **Table 33.1**. This classification aims to eliminate discrepancies due to particle size.

High shrinkage potential soils may not behave very differently from low shrinkage ones, because environmental conditions in the UK do not allow full potential to be realised (Reeve *et al*., 1980). The National House-Building Council (NHBC, 2011a) classified volume change potential as shown in **Table 33.2**. This classification forms the basis of the NHBC's 'foundation depth' tables.

Since a set of soil properties will often not fit neatly into one category, the determination of shrinkage potential requires some judgement. The BRE (1993a) suggests that plasticity index and clay fraction can be used to indicate the potential of a soil to shrink, or swell, as follows:

Plasticity index (%)	Clay fraction (<0.002 mm)	Shrinkage potential
>35	>95	Very high
22–48	60–95	High
12–32	30–60	Medium
<18	<30	Low

The overlap of categories reflects the fact that figures were obtained from multiple sources.

33.5.3.3 National versus regional characteristics

A meaningful assessment of the shrink–swell potential of soil in the UK requires a considerable amount of high quality and well-distributed spatial data of a consistent standard. The British Geological Survey's *National Geotechnical Properties Database* (Self *et al*., 2008) contains a large body of index test data. At the time of writing, the database contained data from more than 80 000 boreholes, comprising nearly 320 000 geotechnical samples, with 100 000 containing relevant plasticity data.

The British Geological Survey (BGS) GeoSure *National Ground Stability Data* provides geological information about potential ground movement or subsidence, including the GeoSure shrink–swell dataset (Booth *et al*., 2011). It should be noted that this assessment does not quantify the shrink–swell behaviour of a soil at a particular site; it indicates the potential for such a hazard to be present with regard to the behaviour of the underlying geological unit throughout its outcrop.

The VCP of a soil provides the relative change in volume to be expected with changes in soil water content. This was calculated from the I'_p values and a classification made based on the *upper quartile* value (**Table 33.3**). This is based on the BRE (1993a) scheme shown in **Table 33.1**. In this way, a VCP was assigned to each of the geological units and a map of shrink–swell potential built (**Figure 33.10**).

Looking at clays on a national scale can give a good indication of the potential problems associated with them and provide

Classification	I'_p (%)	VCP
A	< 1	Non-plastic
B	1–20	Low
C	20–40	Medium
D	40–60	High
E	> 60	Very high

Table 33.3 Classification of VCP

initial information regarding planning decisions. However, no two clay soils are the same in terms of their behaviour or their shrink–swell potential. Therefore, it is useful to look at a particular clay formation on a more regional basis. For illustration, the London Clay formation will be used.

The London Clay formation is of major importance in the fields of geotechnical engineering and engineering geology. This is because it has hosted a large proportion of sub-surface engineering works in London over the last 150 years. It has also been the subject of internationally recognised research in soil mechanics

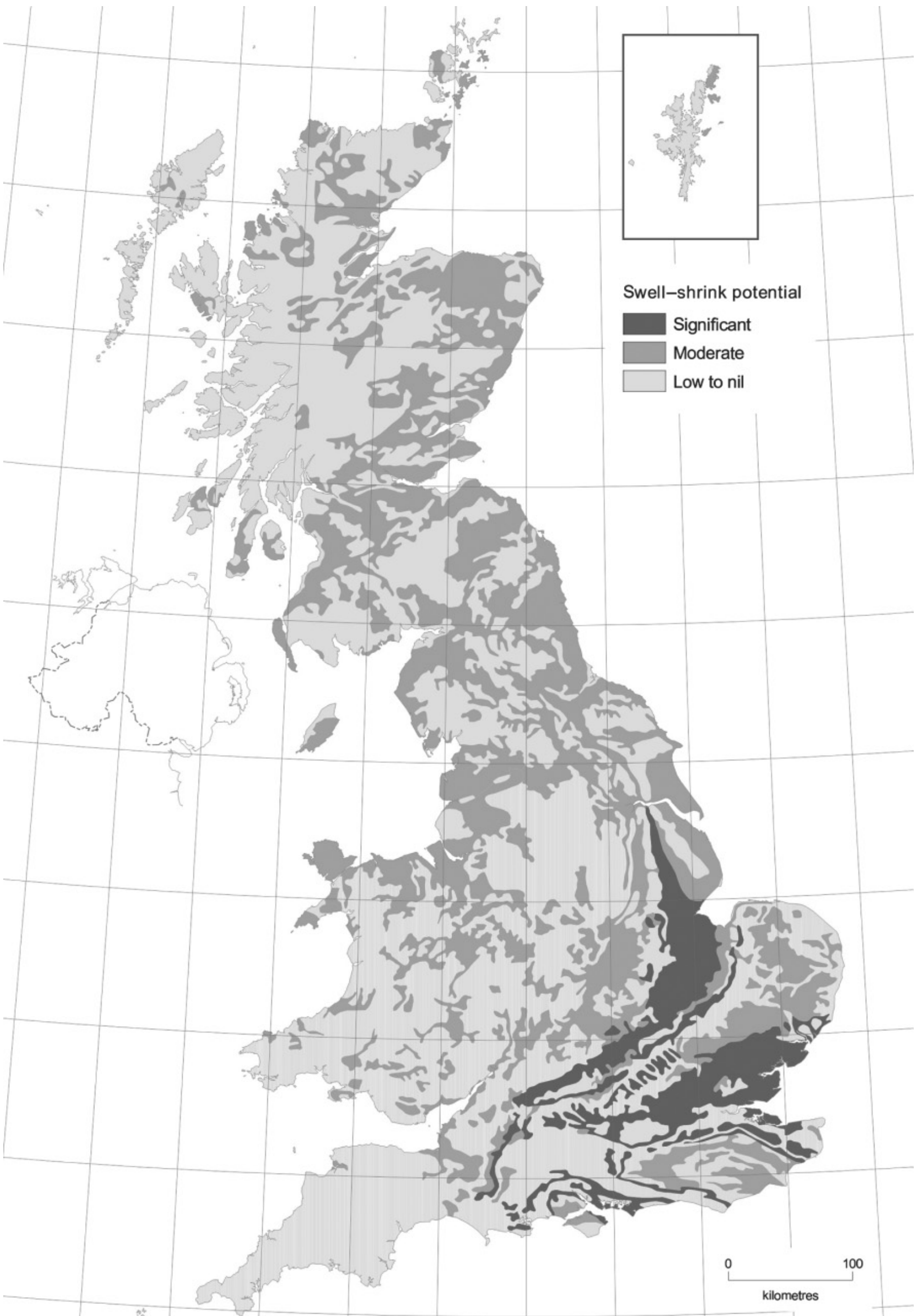

Figure 33.10 Shrink–swell potential map, based on VCP
Reproduced from Jackson (2004) © NERC, with permission from the British Geological Survey

over the last 50 years (Skempton and DeLory, 1957; Chandler and Apted, 1988 and Takahashi *et al*., 2005). The London Clay is subject to shrinkage and swelling behaviour, which has resulted in a long history of foundation damage within the outcrop.

Jones and Terrington (2011) follow the methodology described in Diaz Doce *et al*. (2011) using 11 366 samples across the London Clay outcrop, splitting it into four distinct areas based on geographical location, plasticity values and depth of overlying sediment. In this way, a more detailed assessment of the outcrop could be carried out, and a 3D model providing a seamless interpolation of the VCP of the London Clay was created. This model gives a visualisation of the I'_p values, allowing them to be examined at a variety of depths relative to ground level (**Figure 33.11**). This type of analysis indicates that 3D modelling methods have considerable potential for predicting the spatial variation of VCP within expansive clay soils, so long as they have large enough data sets.

33.5.4 Specific problems with expansive soils

The principal adverse effects of the swell–shrink process arise when either swelling pressures result in heaving (or lifting) of structures, or shrinkage leads to differential settlement. As a result, a number of mitigation and design options exist either in the form of specific foundation types, or through the use of a range of different ground improvement techniques. Excellent reviews of the full range of these are provided by both Chen (1988) and Nelson and Miller (1992), together with details provided by NHBC (2011a). A summary is provided in the following sections (33.5.4.1–33.5.4.4) highlighting the key features associated with these options. In addition, discussion of some of the key issues faced in the UK is provided (see section 33.5.4.5) where impact of vegetation is often the major cause of soil–structure problems faced by expansive soils.

33.5.4.1 Foundation options in expansive soils

A large number of factors influence foundation types and design methods (see Section 5 *Design of foundations*); these include climatic, financial and legal aspects, as well as technical issues. Importantly, swell–shrink behaviour often does not manifest itself for several months and so design alternatives must take account of this. Other issues, such as financial considerations, can place strain on this and so early communication with all relevant stakeholders is essential. Higher initial costs are often offset many times over by a reduction in post-construction maintenance costs when dealing with expansive soils (Nelson and Miller, 1992).

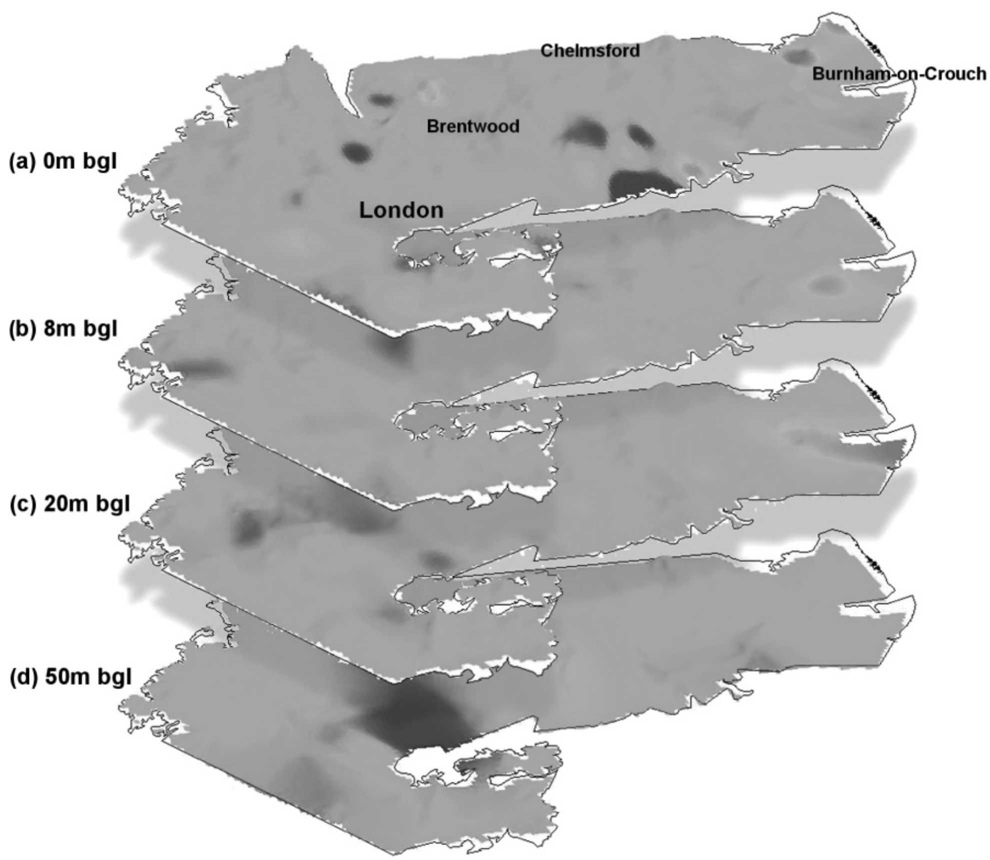

Figure 33.11 S-grid interpolations for area 3, showing surfaces at 0 m, 8 m, 20 m and 50 m bgl. [blue: medium, green: high, yellow/red: very high VCP]
Reproduced from Jones and Terrington (2011) © The Geological Society of London. A colour version of this figure is available online

Foundation alternatives when dealing with potentially expansive soils follow three options:

1. use of structural alternatives, e.g. stiffened raft;
2. use of ground improvement techniques;
3. a combination of (1) and (2).

As with any foundation option, the main aim is to minimise the effects of movement, principally differential. Two strategies are used when dealing with expansive soils:

- isolate structure from soil movements;
- design a foundation stiff enough to resist movements.

The major types of foundations used in expansive soils from around the world are pier and beam or pile and beam systems, reinforced rafts and modified continuous perimeter spread footings. These are summarised in **Table 33.4**; further details are provided by Chen (1988), Nelson and Miller (1992) and NHBC (2011a, 2011b, 2011c), and are discussed further below. It should be noted that terminology used to describe the foundation types listed in this table vary across the world with, for example, slab-on-grade used in the US for raft foundations.

Pier and beam; pile and beam foundations

These foundations consist of a ground beam to support structural loads, transferring the load to the piers or piles. A void is provided between the pier/pile and the ground beam to isolate the structure and prevent uplift from swelling. NHBC (2011a) provides guidance on minimum void dimensions. Floors are then constructed as floating slabs. The piers/piles are reinforced (with reinforcement taken over the whole length to avoid tensile failures) using concrete shafts with or without bell bottoms, steel piles (driven or pushed), or helical piles whose aim is to transfer loads to stable strata. Under-reamed bottoms and helical piers/piles can be effective in soils with a high swell potential, overcoming the impractical length that would otherwise be required with straight shaft piers/piles, or where there is a possibility of a loss of skin friction due to rising groundwater levels. If a stable non-expansive stratum occurs near the surface, the piers/piles can be designed as rigid anchoring members. If, however, the depth of potential swell is high, the piers/piles should be designed as elastic members in an elastic medium. **Figure 33.12** illustrates a typical pier and beam foundation from US practice. Very similar arrangements are used in the UK and are illustrated in NHBC (2011a, Figures 10 and 11, therein).

Design and construction procedures for each of these systems are provided in detail (including sample design calculations) by Chen (1988) and Nelson and Miller (1992). Additional discussion and example design calculations are provided by Nelson *et al*. (2007). It is important to ensure sufficient anchorage below the active zone. Pier/pile diameters are kept small (typically 300–450 mm). Any smaller, and problems will result in poor concrete placement and associated defects, e.g. void spaces. Another problem that can occur is 'mushrooming' near the top of the pier/pile, which provides an additional area for uplift forces to act upon. To avoid this, cylindrical cardboard forms are often employed and removed after the beam is cast to prevent a means to transmit swell pressures. The size of this void space depends on the magnitude of potential swell, with 150–300 mm often being used. In the upper active zone, shafts should be treated to reduce skin friction and hence minimise uplift forces. It is important that any chosen approach does not provide potential pathways to allow water to ingress to deeper layers, as this will cause deep-seated swelling.

Stiffened rafts

Stiffened slabs are either reinforced or post-tensioned systems, the latter being common in countries like the US. Design procedures consist of determining bending moments, shear, and deflections, associated with structural and swell pressure loads. The general layout used is illustrated in **Figure 33.13**, which shows examples used commonly in the US. Similar approaches are used in the UK and are presented in NHBC (2011a; 2011b).

Designs are modelled on the soil–structure interaction at the base of the slab, by considering the slab as a loaded plate or beam resting on an elastic medium. Essentially, two extremes exist – the first where a ground profile develops assuming a

Foundation type	Design philosophy	Advantages	Disadvantages
Pier and beam; pile and beam	Isolate structure from expansive movement by counteracting swell with anchoring to stable strata	Can be used in a wide variety of soils; reliable for soils of high swell potential	Relatively complex design and construction processes requiring specialist contractors
Raft; stiffened raft	Provides a rigid foundation to protect structure from differential settlements	Reliable for soils of moderate swell potential; no specialist equipment needed in construction	Only works for relatively simple building layout; requires full construction quality control
Modified continuous perimeter footing; deep trench fill foundations	Same as raft or stiffened raft foundation – includes stiffened perimeter beams	Simple construction with no specialist equipment needed	Ineffective in highly expansive soils or within the zone of influence of trees

Table 33.4 Foundation types used in expansive soils
Data taken from Nelson and Miller (1992); NHBC (2011a)

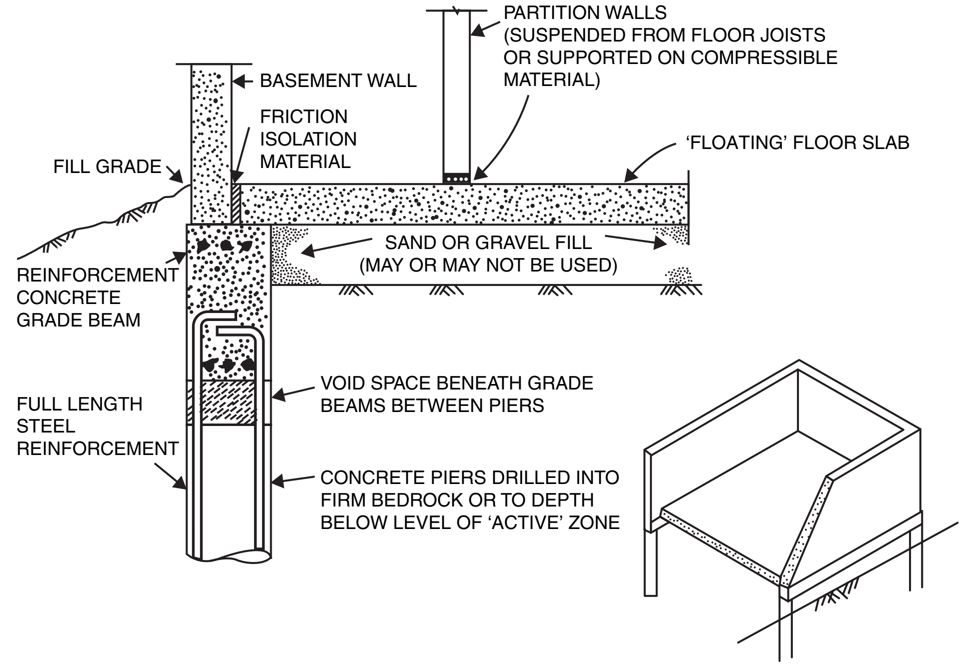

Figure 33.12 Illustration of a pier and beam foundations
Reproduced from Nelson and Miller (1992); John Wiley & Sons, Inc

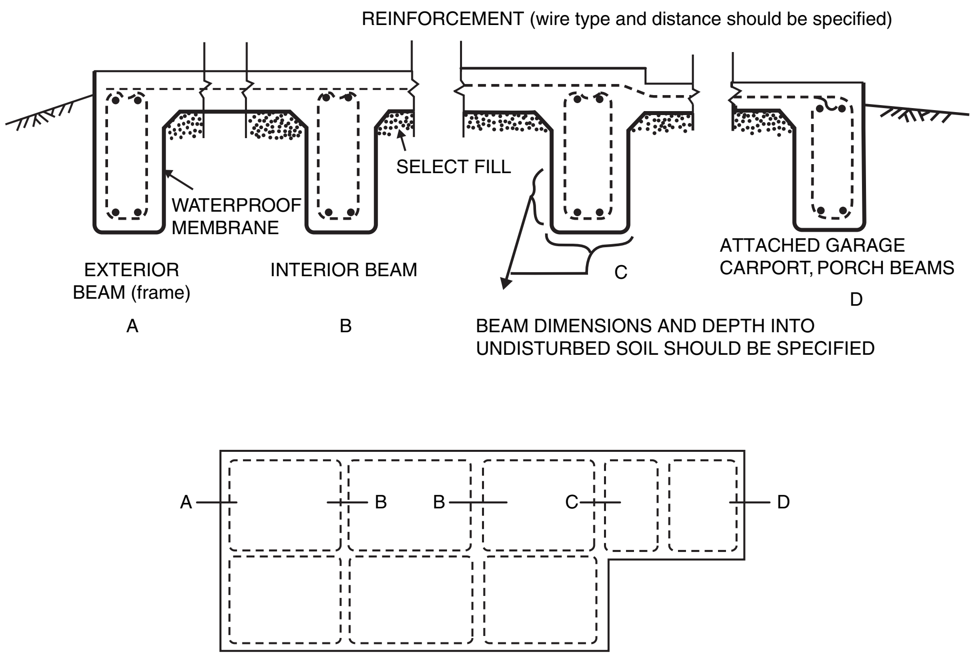

Figure 33.13 Typical detail of a stiffened raft
Reproduced from Nelson and Miller (1992); John Wiley & Sons, Inc

weightless slab, and the second where a slab of infinite stiffness is placed on the swelling soil. In reality, slabs exhibit some flexibility and so the actual heave produced by swelling soils lies somewhere between these two extremes. These modes of movement are illustrated in **Figure 33.14**.

Several design approaches have been developed, each using a range of different combinations of soil and structural design parameters. A detailed account of these is provided by Nelson and Miller (1992) with additional discussion provided by Houston *et al*. (2011).

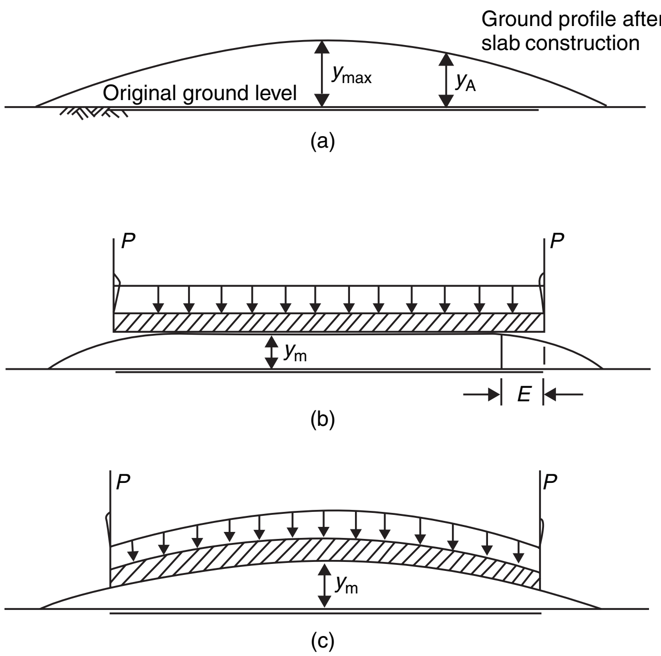

Figure 33.14 Profiles after construction for various stiffness of raft: (a) with no load applied; (b) with infinitely stiff slab; (c) with flexible slab. Notes: y_{max} = maximum heave, no foundation present – the free field heave; y_m = maximum differential heave; E = distance from outer edge to point where swelling soil contacts foundation; P = loading; y_A = height of free field heave along ground profile
Reproduced from Nelson and Miller (1992); John Wiley & Sons, Inc

The primarily geotechnical information required includes size, shape and properties of the distorted soil surface that develop below the slab. These depend on a number of factors including heave, soil stiffness, initial water content, water distribution, climate, post-construction time, loading, and slab rigidity. It should be noted that the slab, through its elimination of evapotranspiration (see **Figure 33.5**), promotes the greatest increase in water content near to the centre of the slab – and hence to where long-term distortion is most severe. However, the maximum differential heave (y_m in **Figure 33.14**) has been found to vary between 33 and 100% of total maximum heave (Nelson and Miller, 1992). On occasion, edge heave can occur when the exterior of a structure experiences increases in water content before the interior.

Modified continuous perimeter footing

Shallow footing should be avoided where expansive soils are found. However, where they are used, a number of approaches can be employed to minimise the effects of swelling/shrinkage. Modifications include:

- narrowing footing width;
- providing void spaces within support beam/wall to concentrate loads at isolated points;
- increasing perimeter reinforcement – taking this into the floor slab stiffening foundations.

The use of narrow spread footing in expansive soils should be restricted to soils exhibiting 1% swell potential and very low swell pressures (Nelson and Miller 1992).

NHBC (2011a) suggested that strip and trench fill foundations can be used when placed in a non-expansive layer that overlies expansive soils, provided that:

- soil is consistent across the site;
- the depth of non-expansive material is greater than ¾ of the equivalent foundation depth, assuming all soil is expansive (guidance provided within NHBC, 2011a);
- the thickness of the non-expansive soil below the foundation is at least equal to the foundation width.

Case studies

Chen (1988) provides a series of case study examples of foundations and problems that arise when dealing with expansive soils, including distress caused by the following: pier/pile uplift, improper pier/pile design and construction, heaving of a pad and floor slab, heaving of a continuous floor, and a rising water table. Further reviews of issues related to other foundation types, for example the use of post-tensioned stiffened raft foundations, are discussed by Houston *et al*. (2011). Other useful case studies are provided by Simmons (1991) and Kropp (2011). It is clear that a number of foundation failures occur and these can be summarised as follows:

1. *Changes in water content*
   - chiefly high water tables;
   - poor drainage under foundations;
   - leaks due to sewer failure or poorly managed runoff;
   - irrigation and garden watering.
2. *Poor construction practice*
   - insufficient edge beam stiffness;
   - inadequate slab thickness;
   - inadequate anchorage from piers;
   - pier length inadequate or 'mushrooming' of piers/piles resulting in uplift as swelling occurs;
   - lack of reinforcement making structure intolerant to movements;
   - inadequate void space.
3. *Lack of appreciation of soil profile*
   - underlying geology contains inclined bedding of bedrock, causing swell to be both vertical and horizontal;
   - uncontrolled fill placement;
   - areas of extensive depth of expansive soil, so drilled pier and beam foundation may not be practical and a more flexible system should be used.

When assessing failure from swell–shrink behaviour it is important to isolate structural defects from foundation movement, as both can cause cracking distress in buildings (Chen, 1988). Useful reviews of geotechnical practice in relation to expansive soils have been provided by Lawson (2006) for Texas, Kropp (2011) for the San Francisco Bay Area, and Houston *et al*. (2011) for Arizona. Although these are US-based, there are many lessons that geotechnical engineers can learn from these studies. Ewing (2011) provides an interesting case from Jackson, Mississippi, USA, of a series of repairs over a 30-year period to a house (on the US's register of historic places) built on 1.5 m of non-expansive soils overlying expansive clay some 8 m thick.

33.5.4.2 Pavement and expansive soils

Pavements are particularly vulnerable to expansive soil damage, with estimates suggesting that they are associated with approximately half of the overall costs from expansive soils (Chen 1988). Their inherent vulnerability stems from their reasonably lightweight nature, extended over a relatively large area. For example, Cameron (2006) describes problems with railways built on expansive soils where poor drainage exists, and Zheng *et al*. (2009) provide details (from China) of highway sub-grade construction on embankments and in slopes. Damage to pavements on expansive soils comes in four major forms:

- severe unevenness along significant lengths – cracks may or may not be visible (particularly important for airport runways);
- longitudinal cracking;
- lateral cracking, developed from significant localised deformations;
- localised pavement failure associated with disintegration of the surface.

Pavement design is essentially the same as that used for foundations. However a number of different approaches are required as pavements cannot be isolated from the soils and it is impractical to make pavements stiff enough to avoid differential movements. Therefore it is often more economic to treat sub-grade soils (see section 33.5.4.3 below for further details). Pavement designs are based on either flexible or rigid pavement systems; these procedures are discussed in Section 7 *Design of earthworks, slopes and pavements* and Chapter 76 *Issues for pavement design* of this manual. However, when dealing with expansive soils a number of approaches should be considered:

1. choose an alternative route and avoid expansive soil;
2. remove and replace expansive soil with a non-expansive alternative;
3. design for low strength and allow regular maintenance;
4. physically alter expansive soils through disturbance and re-compaction;
5. stabilise through chemical additives, such as lime treatment;
6. control water content changes – although very difficult over the life of a pavement. Techniques include pre-wetting, membranes, deep drains, slurry injection treatment.

Nelson and Miller (1992) provide further details on testing undertaken to mitigate expansive soil behaviour for pavement construction. Cameron (2006) has advocated the use of trees as they can be beneficial in semi-arid environments to manage poorly-drained areas under railways. However, this needs careful management and may require several years to be fully effective.

33.5.4.3 Treatment of expansive soils

Essentially, treatment of expansive soils can be grouped into two categories:

1. soil stabilisation – remove/replace; remould and compact; pre-wet, and chemical/cement stabilisation;
2. water content control methods – horizontal barriers (membranes, asphalt and rigid barriers); vertical barriers; electrochemical soil treatment, and heat treatment.

A detailed account of the various treatment approaches is provided by Chen (1988) and Nelson and Miller (1992), with a detailed review of stabilisation over the last 60 years provided by Petry and Little (2002). As with any treatment approach, it is essential to undertake appropriate site investigations and evaluations (see Section 6 *Design of retaining structures* and section 33.5.1 above). Special consideration should be given to the following: depth of the active zone, potential for volume change, soil chemistry, water variations within the soil, permeability, uniformity of the soils, and project requirements. An overview of each of the two categories of treatments applied to expansive soils is provided below, with **Table 33.5** providing brief details of soil stabilisation approaches.

In a recent survey, Houston *et al*. (2011) found that many geotechnical and structural engineers considered chemical stabilisation approaches, such as the use of lime, as ineffective for pre-treatment of expansive soils for foundations. Preference is typically given for use of either pier/pile and beam foundations, or stiffened raft foundations. This is not true for pavements, where lime and other chemical stabilisation approaches are commonly used worldwide. The various stabilisers can be grouped into three categories (Petry and Little, 2002):

- traditional stabilisers – lime and cement;
- by-product stabilisers – cement/lime kiln dust and fly ash;
- non-traditional stabilisers – e.g. sulfonated oils, potassium compounds, ammonium compounds and polymers.

Further details of these can be found in Petry and Little (2002). However, as with any soil treated with lime, care is needed to assess chemical as well as physical soil properties to prevent swelling from adverse chemical reactions (Petry and Little, 2002). For example, Madhyannapu *et al*. (2010) provide details of quality control when stabilising expansive sub-soils using deep soil mixing, demonstrating the use of non-destructive tests based on seismic methods.

Chemical stabilisation can be used to provide a cushion immediately below foundations placed on expansive soils, e.g. for pavements (Ramana and Praveen, 2008). Swell mitigation has also been achieved by mixing non-swelling material e.g. sand (Hudyma and Avar, 2006) or granulated tyre rubber (Patil *et al*., 2011) into expansive soils to dilute swell potential.

In some cases surcharging may be used, but this is only effective with soils of low to moderate swelling pressures. This requires enough surcharge load (see the first row in **Table 33.5**) to counteract expected swell pressures. This method is therefore only used for soil of low swell pressure and with structures that can tolerate heave. Examples include secondary highway systems, or where high foundation pressures occur. Pre-wetting – due to its uncertainties – can only be used with caution, with both Chen (1988) and Nelson and Miller (1992) indicating that it is unlikely to play an important role in the construction of foundations on expansive soils.

Fluctuations in water content are one of the primary causes of swell–shrink problems, with non-uniform heave occurring due to non-uniformity of water content, soil properties, or both. Thus, if water content fluctuations can be minimised over time, then swell–shrink problems can be mitigated. Moreover, if water content changes can be slowed down and water distributions in expansive soils made uniform, then differential movements can also be reduced. In essence, this is the aim of the introduction of moisture/water barriers. These act to:

1. move the edge effects away from the foundation/pavement and so minimise seasonal fluctuation effects;
2. lengthen the time for water content changes to occur – due to longer migration paths under foundations.

Barrier techniques comprise:

- horizontal barriers – using membranes, bituminous membranes or concrete;
- vertical barriers – polyethylene, concrete, impervious semi-hardening slurries.

Improvement approach	Outline of approach	Advantages	Disadvantages
Removal and replacement	Expansive soil removed and replaced by non-expansive fill to a depth necessary to prevent excessive heave. Depth governed by weight needed to prevent uplift and mitigate differential movement. Chen (1988) suggests a minimum of 1–1.3 m	Non-expansive fill can achieve increase bearing capacities; simple and easy to undertake; often quicker than alternatives	Preferable to use impervious fill to prevent water ingress which can be expensive; thickness required may be impractical; failure can occur during construction due to water ingress
Remoulding and compaction	Less expansion observed for soil compacted at low densities above OWC[(1)] than those at high densities and below OWC (see **Figure 31.15**). Standard compaction methods and control can be used to achieve target densities	Uses clay on site, eliminating cost of imported fill; can achieve a relatively impermeable fill, minimising water ingress; swell potential reduced without introducing excess water	Low density compaction may be detrimental to bearing capacity; may not be effective for soil of high swell potential; requires close and careful quality control
Pre-wetting or ponding	Water content increased to promote heave prior to construction. Dykes or berms used to impound water in flooded area. Alternatively, trenches and vertical drains can be used to speed infiltration of water into soil	Has been used successfully when soils have sufficiently high permeabilities to allow relatively quick water ingress, e.g. with fissure clays	May require several years to achieve adequate wetting; loss of strength and failure can occur; ingress limited to a depth less than the active zone; water redistribution can occur – causing heave after construction
Chemical stabilisation	Lime (3–8% by weight) common with cements (2–6% by weight) sometimes used, and salts, fly ash and organic compounds less commonly used. Generally lime mixed into surface (~300 mm), sealed, cured and then compacted. Lime may also be injected in slurry form. Lime generally best when dealing with highly plastic clays	All fine-grained soils can be treated by chemical stabilisers; effective in reducing plasticity and swell potential of an expansive soil	Soil chemistry may be detrimental to chemical treatment; health and safety need careful consideration as chemical stabilisers carry potential risks; environmental risks may also occur – e.g. quick lime is particularly reactive; curing inhibited in colder temperatures

[(1)] OWC – optimum water content, as determined by standard proctor test BS1377 (BSI, 1990).

Table 33.5 Soil stabilisation approaches applied to expansive soils
Data taken from Nelson and Miller (1992)

Detailed accounts of these are provided in both Chen (1988) and in Nelson and Miller (1992). In addition, electrochemical soil treatment approaches are being developed that utilise electrical current to inject stabilising agents into the soils. Further details are provided by Barker *et al*. (2004). As well as barrier methods, water management can be employed with restrictions applied to avoid irrigation within certain distances of the structure. However, monitoring is needed to ensure compliance with these restrictions.

33.5.4.4 Remedial options

Expansive soils cause significant damage to buildings, as discussed throughout this chapter, and so remedial action is required to repair any damage. However, it is important to establish a number of factors before embarking on a remedial plan. Key questions that should be considered are (after Nelson and Miller, 1992):

- Are remedial measures needed – is damage severe enough to warrant treatment?
- Is continued movement anticipated and so would it be better to wait?
- Who will pay?
- What criteria should be selected?
- How has the damage been caused and what is its extent?
- What remedial measures are applicable?
- Are there any residual risks post remediation?

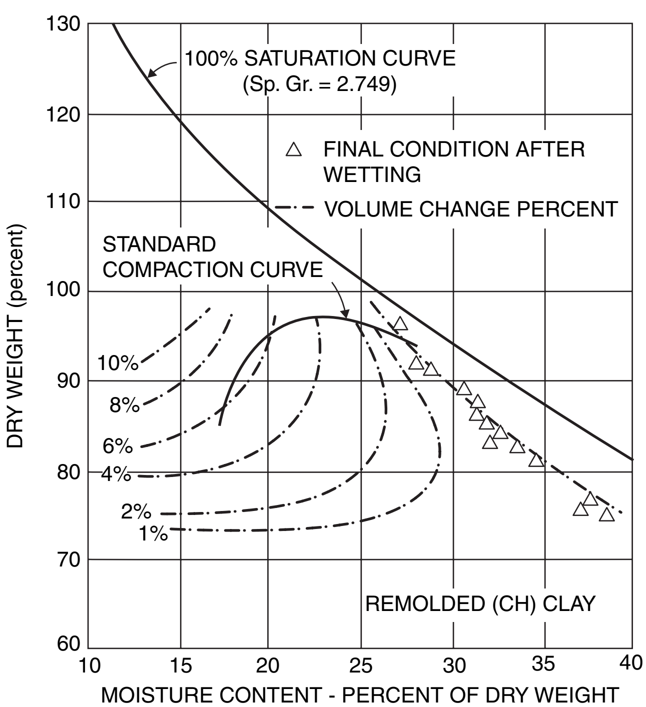

Figure 33.15 Percentage expansion for various placement conditions (c.f. **Table 33.5**)
Reproduced from Holtz (1995) all rights reserved

Clearly, to select an appropriate remedial measure, an adequate forensic site investigation is required. Key information required includes the cause and extent of the damage, the soil profile (as it is often difficult to determine whether settlement/heave is the cause of structural distress), and the soil's expansive potential. Other necessary information has already been discussed in section 33.5.1 above. Failure to carry out an adequate site investigation can lead to false diagnoses and inappropriate remedial measures employed. Further details are provided by Nelson and Miller (1992) as well as BRE Digests 251 (1995a), 298 (1999), 361 (1991), 412 (1996) and 471 (2002).

The following are examples of remedial measures employed for foundations:

- repair and replace structural elements or correct improper design features;
- underpin;
- provide structural adjustments of additional structural supports e.g. post tensioning;
- stiffen foundations;
- provide drainage control;
- stabilise water contents of foundation soils;
- install moisture barriers to control water content fluctuations.

Full underpinning of an operational structure is often impractical (and increasingly seen as unnecessary) and it is more common for underpinning work to be applied only to key parts of the foundations (Buzzi *et al*., 2010). Moreover, localised application of underpinning to deal with differential settlements may not improve the overall performance of the foundation (Walsh and Cameron, 1997). Thus any localised treatment must be designed to take account of all factors, otherwise there is a danger of exacerbating the problems due to the inherent natural spatial variability of expansive soils. Recently, underpinning using expanded polyurethane resin has met with some success, because resin can be injected using small diameter tubes directly where it is needed (Buzzi *et al*., 2010). However, due to concerns about its long-term stability and the possibility that swelling in injected soils could be exacerbated if all the cracks were filled, its adoption has been slow. However, a detailed experimental study (Buzzi *et al*., 2010) concluded that resin injected expansive soils did not exhibit enhanced swelling as a number of cracks remained unfilled, providing swell relief. Problems with lateral swelling can sometimes be accommodated by cracking within the soil matrix. However, if no cracks are present, problems can occur – particularly with retaining structures. Expanded polystyrene geofoam has demonstrated some success with dealing with lateral expansion, and has been shown to reduce the subsequent impact of vertical swelling (Ikizler *et al*., 2008).

With respect to pavements, distress can be considered as one of four possible types of damage, as highlighted in section 33.5.4.2 above. Most common remedial measures are either removal and replacement, or construction of overlays. Whichever method is used, care is needed to ensure that the causes of the original distress are dealt with.

Many of the pre-construction approaches can also be used for post-construction treatments; for pavements these include moisture barriers, removal, replacement and compaction, and drainage control.

33.5.4.5 Domestic dwelling and vegetation

Tree roots will grow in the direction of least resistance and where they have the best access to water, air and nutrients (Roberts, 1976). The actual pattern of root growth depends upon, amongst other factors, the type of tree, the depth to the water table, and local ground conditions. Trees will tend to maintain a compact root system. However, when trees become very large, or where trees are under stress, they can send root systems far from the trunk. There is some published guidance on 'safe planting distances' that can be used by the insurance industry to inform householders of the potential impacts of different tree species on their properties. Further details are also given in NHBC (2011a).

Paving of previously open areas of land, such as the building of patios and driveways, can cause major disruption to the soil–water system. If the paving cuts off infiltration, many trees will send their roots deeper into the ground or further from the trunk in order to source water. The movement of these tree roots will cause disturbance of the ground and will lead to the removal of water from a larger area around the tree. Problems occur when houses are situated within the zone of influence of a tree (**Figure 33.16**).

If an impermeable method of paving is used, it may prevent water from penetrating into the ground. This can affect

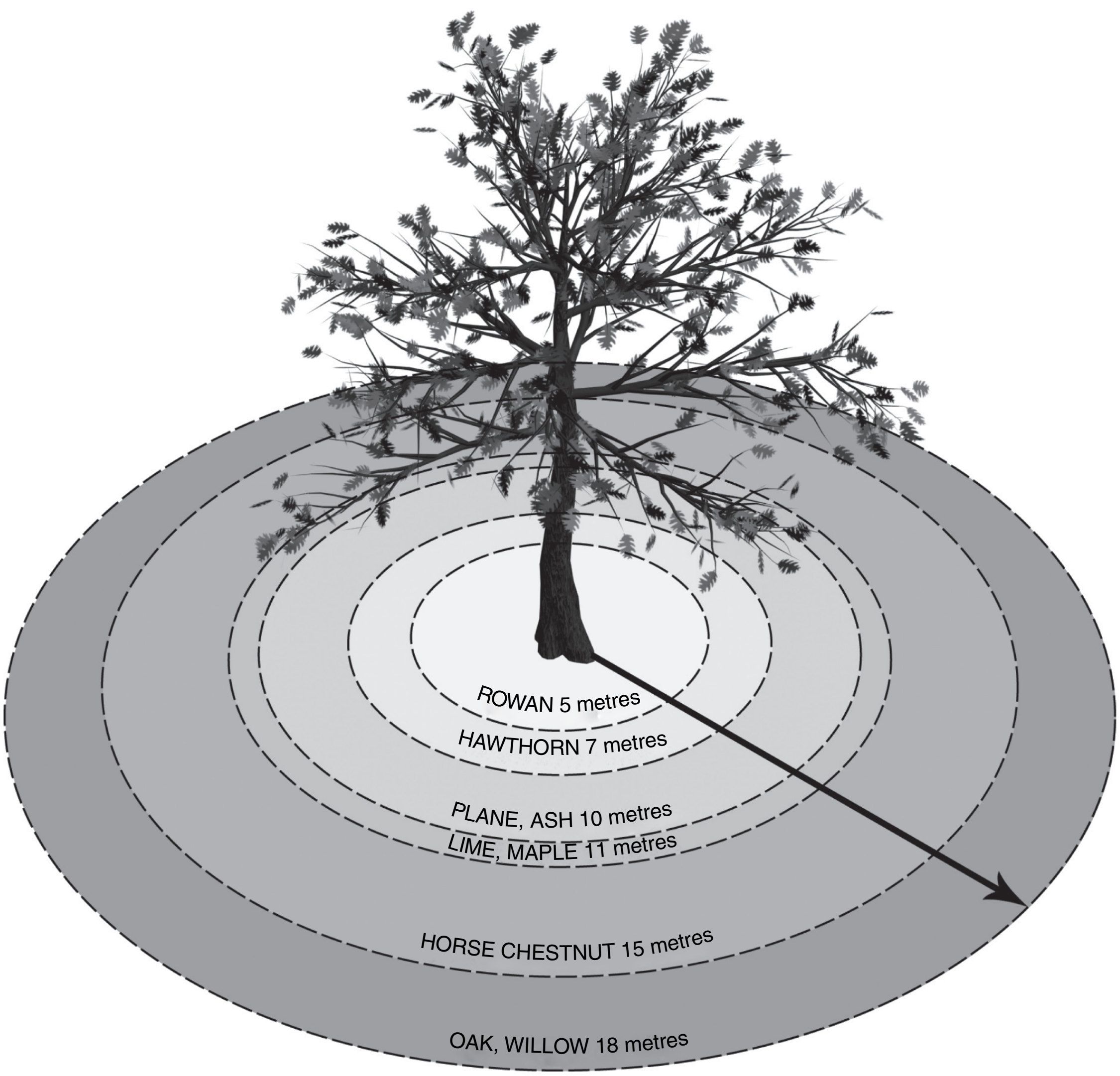

Figure 33.16 The zone of influence of some common UK trees
Reproduced from Jones *et al*. (2006) © NERC, with permission from The British Geological Survey

the shrink–swell behaviour of the ground and also the growing patterns of nearby trees. A well-designed impermeable paving system, in good condition, may actually reduce the amount of shrink–swell activity in the ground immediately below it. Paving moderates variations in water content of the soil and thus the range of shrink–swell behaviour. However, if the paving seal is broken, water can suddenly enter the system, causing swelling of the ground.

Different problems are faced when considering the distinctly separate areas of designing new build structures and remediating existing damaged buildings. New build guidelines for domestic dwellings recognise the need for thorough ground investigations to design systems to cope with the hazards presented by existing trees or their recent removal. Reference should be made to National House Building Council (NHBC) Standards Chapter 4.2 *Building Near Trees* (NHBC, 2011a) and the *Efficient Design of Piled Foundations for Low-Rise Housing – Design Guide* (NHBC, 2010). In the case of existing dwellings, a range of reports and digests are available (e.g. BRE Digests 298, 1999; 412, 1996) and *A Good Technical Practice Guide* provided by Driscoll and Skinner (2007).

Essentially, foundations should make allowances for trees in expansive (swell–shrink) soils and should take account of (NHBC, 2011a):

- shrinkage/heave linked to changes in water content;
- soil classification;
- water demand of trees (this is species-dependent);
- tree height;
- climate.

In the case of existing structures, the main cause of distress results from the effects of differential settlement, where different parts of the building move by varying amounts due to variations in the properties of the underlying soil. Equal or proportionate movements across the plan area of a building, though significant in terms of vertical movement, may result in little structural damage (IStructE, 1994). However, in the UK this is rare; by far the most overwhelming cause of damage to property results from the desiccation of clay subsoil which consequently causes differential settlements/movements, often stemming from the abstraction of water by the roots of nearby vegetation.

If vegetation is involved, it produces a characteristic seasonal pattern of foundation movement: subsidence in the summer, reaching a maximum around September, followed by upward recovery in the winter (see **Figure 33.17**). If subsidence followed by recovery is occurring, there is no need to try to demonstrate shrinkable clay or desiccation. No other cause produces a similar pattern – soil drying by vegetation must be involved (unless the foundations are less than 300 mm). Furthermore, there is no need to demonstrate the full cycle as it is sufficient to confirm movement is consistent with this pattern. Monitoring upward recovery in the winter is particularly valuable in this case. Further details are given by Crilly and Driscoll (2000) and Driscoll and Chown (2001), drawn from a test site in Chattenden, Kent, set in expansive London Clay (see **Figure 33.17**). In addition, both articles provide details of instrumented piles, discussing design implications.

Level monitoring can demonstrate this pattern. BRE Digest 344 (1995b) makes recommendations for the taking of measurements of the ‘out-of-level’ of a course of masonry or of the damp-proof course, which can be used to estimate the amount of differential settlement or heave that has already taken place. BRE Digest 386 (1993b) discusses precise levelling techniques and equipment which can monitor vertical movements with an accuracy consistently better than ±0.5 mm. Precise levelling can be conducted easily, quickly and accurately and so provides one of the most effective ways to distinguish between potential causes of foundation movement (Biddle, 2001).

The choice of mitigation should be proportionate to the problem and specific to the true area of the affected structure. It is important not to become distracted by extraneous but nevertheless interesting features.

Biddle (2001) suggests one of four remedial options to deal with the adverse actions of trees:

1. fell the offending tree to eliminate all future drying;
2. prune the tree to reduce drying and the amplitude of seasonal movement;
3. control the root spread to prevent drying under foundations;
4. provide supplementary watering to prevent soil from drying.

Biddle (2001) states that it is now recognised that in most situations, underpinning is unnecessary and that foundations can be stabilised by appropriate tree management – usually by felling the offending tree or by carrying out heavy crown reduction. Site investigations should reflect this change and be aimed at providing the information to allow appropriate decisions on tree management. In particular:

- confirmation that vegetation-related subsidence is involved;
- identification of which tree(s) or shrub(s) are involved;
- assessment of the risk of heave if a tree is felled or managed;
- identification of the need for any other site investigations;
- if the tree warrants retention, assessment of whether partial underpinning would be sufficient;
- confirmation that vegetation management has been effective in stabilising the foundations;
- provision of information within an acceptable timescale.

Trees are often pruned to reduce their water use and therefore their influence on the surrounding soil. However, unless the trees are thereafter subjected to a frequent and ongoing regime of management, the problems will very quickly return. Whilst

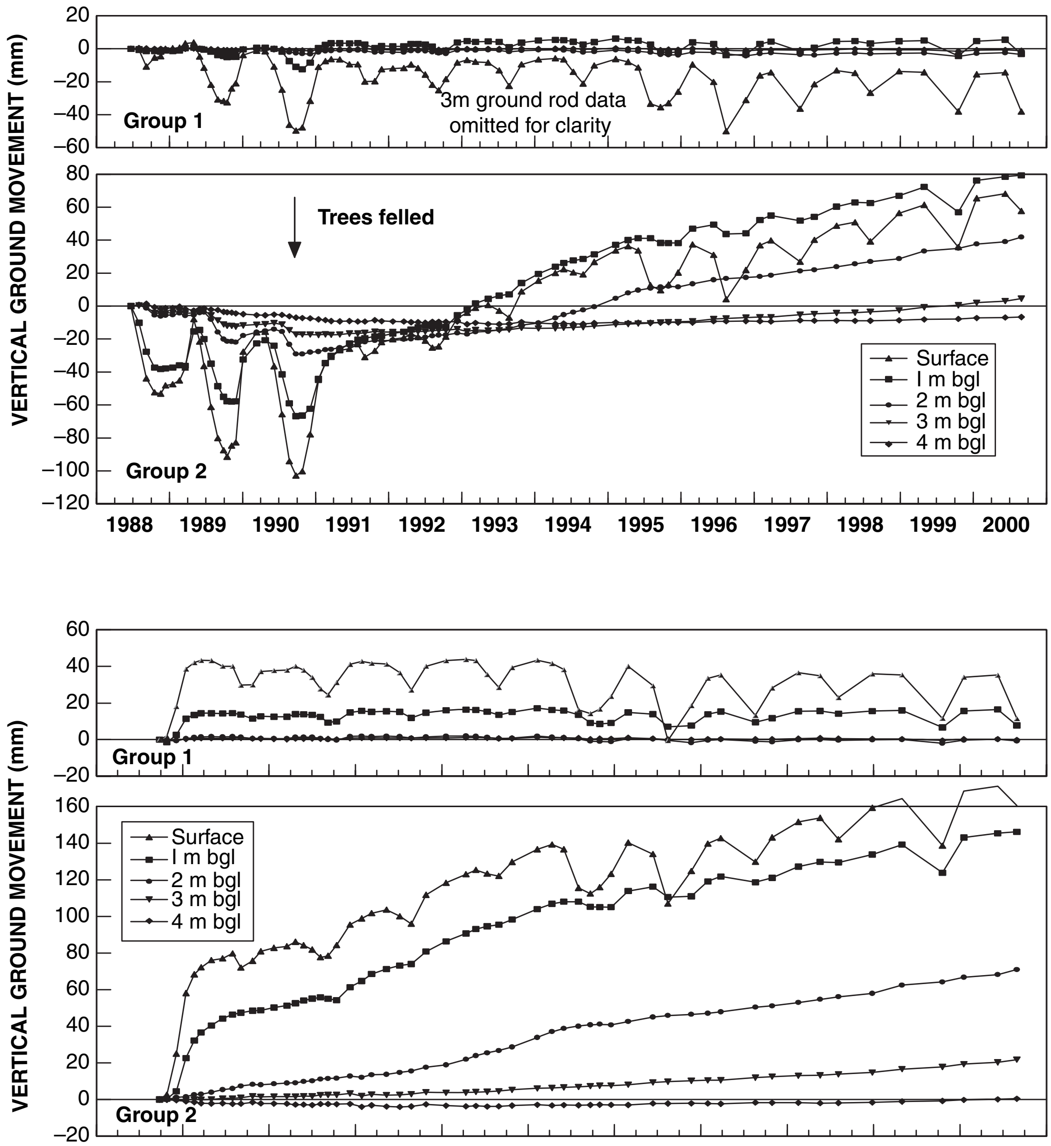

Figure 33.17 Examples of ground movements due to seasonal fluctuations at Chattenden. The upper plot shows results obtained since the first movements in June 1988. The lower plot shows an enlarged scale with results obtained since the trees were felled – group 1 is remote from tree and group 2 near to trees

Reproduced from Crilly and Driscoll (2000); Driscoll and Chown (2001); all rights reserved

tree removal will ultimately provide an absolute solution in the majority of cases, there are situations where this is not an option (e.g. protected trees, adverse risk of heave, incomplete evidence in contentious issues, and physical proximity of trees).

In the past, an obvious and often knee-jerk solution has been to provide significant and often disproportionate support to the structure through foundation strengthening schemes, incorporating various forms of underpinning. This approach is often ecologically, financially and technically incongruent with the problems faced. Alternatively, various forms of physical barriers can be used, constructed from, for example, *in situ* concrete. However, such barriers often prove ineffective over time. Barriers are currently being developed that incorporate a bioroot barrier, which is a mechanically-bonded geocomposite consisting of a copper-foil firmly embedded between two layers of geotextile. Such biobarriers are now being used specifically in arboriculture and for Japanese knotweed control where a permeable barrier is required. They act as signal barriers by diverting root growth (both biologically and physically) without making any attempt to physically restrain their progress.

Alternative remediation by supplementary watering is usually considered impractical due to the quantities required by the tree. This approach can suffer from the unavailability of water precisely when it is needed – due to prevailing drought conditions.

If a mature tree is felled, a building may incur heaving on a dry clay soil. Unfortunately, the evidence is rarely obvious; however, clues to look out for include:

- the house is new – less than 20 years old;
- there is expansive soil present;
- the crack pattern might appear a bit odd – wider at the bottom than at the top, with no obvious cause; and
- cracks continue to open, even in the wet months.

Heave problems can be costly and always require thorough investigation involving soil sampling, precise levels and aerial photographs. Heave is a threat but rarely a reality where established existing properties are involved, and the structure predates the planting of the tree.

Ultimately, if the offending tree can be accurately targeted and dealt with rapidly before the next growing season, the extent of any damage and need for remedial work will be kept to a minimum (Biddle, 2001).

33.6 Conclusions

Expansive soils are one of the most significant ground-related hazards found globally, costing billions of pounds annually. They are found throughout the world – commonly in arid/semi-arid regions – where their high suctions and potential for large water content changes can cause significant volume changes. In humid regions, such as the UK, problematic expansive behaviour generally occurs in clays of high plasticity index. Either way, expansive soils have the potential to demonstrate significant volume changes in direct response to changes in water content. This can be induced through water ingress, through modification to water conditions, or via the action of external influences such as trees.

To understand, and hence engineer expansive soils in an effective way, it is necessary to understand soil properties, suction/water conditions, water content variations (temporal and spatial), and the geometry/stiffness of foundations and associated structures. This chapter provides an overview of these features and includes methods to investigate expansive behaviour both in the field and in the laboratory, together with associated empirical and analytical tools to evaluate it. Following this design, options for pre- and post-construction are highlighted for both foundations and pavements, together with methods to ameliorate potentially damaging expansive behaviour, including dealing with the impact of trees.

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33.7.1 Further reading

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33.7.2 Useful websites

Association of British Insurers; www.abi.org.uk
British Geological Survey (BGS); www.bgs.ac.uk
International Society of Arboriculture, UK and Ireland Chapter; www.isa-arboriculture.org
Royal Institution of Chartered Surveyors; www.rics.org
Subsidence Claims Advisory Bureau; www.subsidencebureau.com
The Clay Research Group, UK; www.theclayresearchgroup.org
The Subsidence Forum; www.subsidenceforum.org
US Geological Survey (USGS); www.usgs.gov

It is recommended this chapter is read in conjunction with

- Chapter 7 *Geotechnical risks and their context for the whole project*
- Chapter 40 *The ground as a hazard*
- Chapter 76 *Issues for pavement design*

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 out in Section 4 *Site investigation.*