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CHAPTER C5 – EXPANSIVE SOILS

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Abstract
Expansive soils present significant geotechnical and structural engineering challenges the world over, with costs associated with expansive behaviour estimated to run into several billion annually. Expansive soils are soils that experience significant volume change associated with changes in water contents. These volume changes can either in the form of swell or in the form shrinkage and this is why they are sometime known as swell/shrink soils. Key aspects that need identification when dealing with expansive soils include: soil properties, suction/water conditions, water content variations temporal and spatial, e.g. generated by trees, and the geometry/stiffness of foundations and associated structures. Expansive soils can be found in humid environments where expansive problems occur with soils of high Plasticity Index (I_p) or in arid/semi arid soils where soils of even moderate expansiveness can cause significant damage. In the UK damage often occurs as a direct result of interaction with vegetation and associated water content changes. Soils that experience swell/shrink problems in the UK are typically found in the south and east of the country, notably around London, corresponding to the drier parts of the UK. However, moderate swell/shrink potential can be exhibited across many parts of the country. 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 in the laboratory and the associated empirical and analytical tools to evaluate expansive behaviour. Following this design options for pre and post construction are highlighted for both foundations and pavements, together with method to ameliorate potentially damaging expansive behaviour.

Keywords
Expansive, swelling, shrinkage, volume change, trees

1. What is an expansive soil?
Essentially expansive soil is one that changes in volume in relation to changes in water content. Here the focus is on soils that exhibit significant swell potential and in addition shrinkage potential also exists. There are a number of cases where expansion can occur through chemically induced changes (e.g. swelling of lime treated sulphate 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 get 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 elsewhere in the Manual.

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 3m 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. When supporting structures, the effects of significant changes in water content on soils with a high shrink–swell potential can be severe.

Swelling and shrinkage are not fully reversible processes (Holtz & 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 time scales 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, thus resulting in enhanced swelling pressures.

Importantly, 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.

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 evapo-transpiration of vegetation), or 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, 1986). 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 C10 in this manual). 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 shrinking and swelling 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 has 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 a year (Driscoll & 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 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 1 – Structural damage to house caused by ‘end lift’ (© Peter Kelsey & Partners).

Damage to foundations in expansive soils commonly results from tree growth. This occurs in two principal ways – physical disturbance of the ground and 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 2. Vegetation induced changes to water profiles can also have a significant impact on other underground feature, including utilities. Clayton et al. (2010) reporting monitoring data over a two year period of a pipes in London clay, finding significant ground movements (both vertical and horizontal) of the order of 3-6 mm/m length of a pipe, generating significantly 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 5.4.5.

Figure 2 - Example of differential settlement due to influence of trees

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 south-east of the country (Figure 3). Here 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 have been hardened by processes resulting from deep burial and are less able to absorb water. Some areas (e.g. around The Wash north-west of Peterborough – see Figure 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 3)).

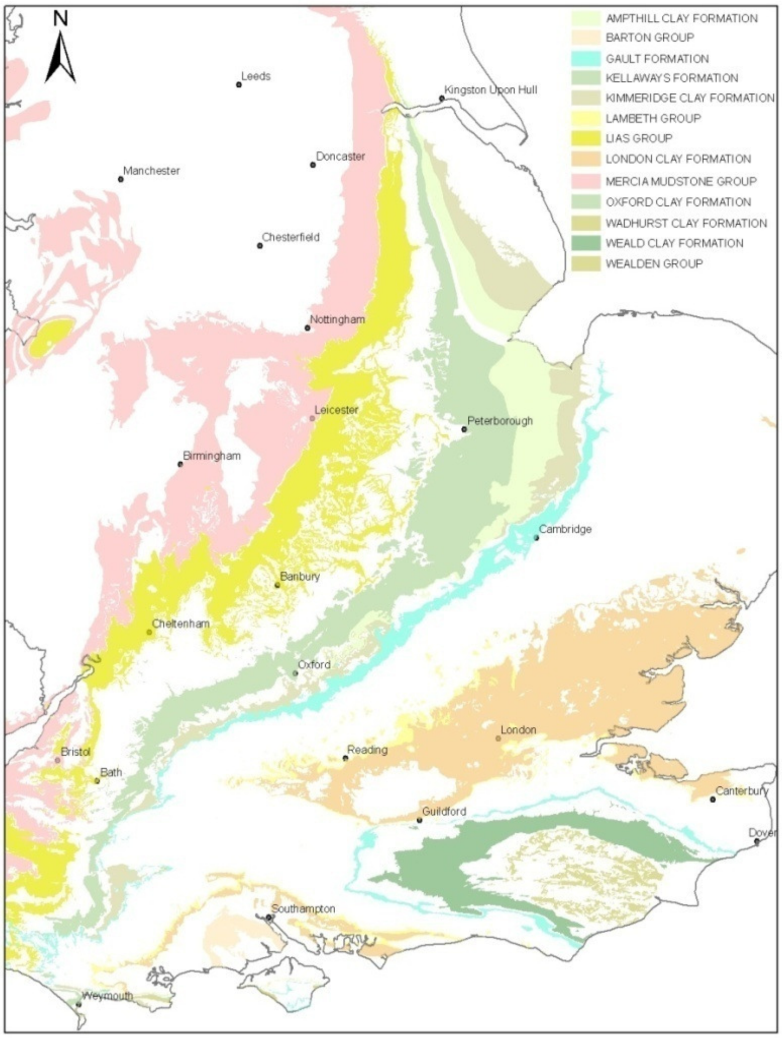


Figure 3 – Distribution of UK clay-rich soil formations (after XXX

Expansive soils are found throughout many regions of the world, particularly in arid and semi-arid regions, as well as 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 & Rahardjo, 1993; Stavridakis, 2006; Hyndman & Hyndman, 2009). Expansive soils occur and incur major construction costs around the world, with notable example found in 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 soil differ in many ways and depend not only on technical developments but legal framework and regulations of a country, insurance policies and the attitude of insurers, experience of the engineers and other specialist dealing with the problem and importantly the sensitivity of the owner of the property affected. In UK in particular there is high sensitivity to relative small crack (see section 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 detail informative study from Arizona US has more recent been presented by Houston et al (2011). This latter study demonstrated how the source of problems from expansive soils often stem from poor drainage, construction problems, home owner activity and its adverse effects and landscaping through use of vegetation and isoften associated with a combination of these. These aspects may be more important a predictor of expansive soil problems than landscape type itself.

Overall, in humid climates, problems with expansive soils trend 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 that exists and the larger changes water content regimes that results when water level change.

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 3m 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 & Culshaw, 2001). These minerals determine the natural expansiveness of the soil, and include smectite, montmorillonite, nontronite, vermiculite, illite and chlorite. Generally, the larger the amount of these minerals 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 aspects of expansive soils behaviour, however, are a 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 control 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, on 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), increase the chances of damage occurring significantly. 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 dried/wet periods of weather (Biddle 2001). In a fully saturated soil the shrink–swell behaviour is controlled by the clay mineralogy.

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, with many other elements which can become incorporated into the clay mineral structure (hydrogen, sodium, calcium, magnesium, sulphur). 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 has 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 2μm (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.

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:

- an immediate, but time dependent elastic rebound
- 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 Poissons 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 water pressures which are set up 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.

The accurate laboratory measurement of the controlling elastic properties at small strains in both rebound and swelling, i.e. before yield takes place, is 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 (Prowrie 2004; Atkinson (2007) – see also other section of the 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 air entry which affected both the modulus and strength of the soil. This process thus leads to a void ratio which is higher than 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 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 & Venkatappa Rao (1971).

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 for illustration Figure 4).

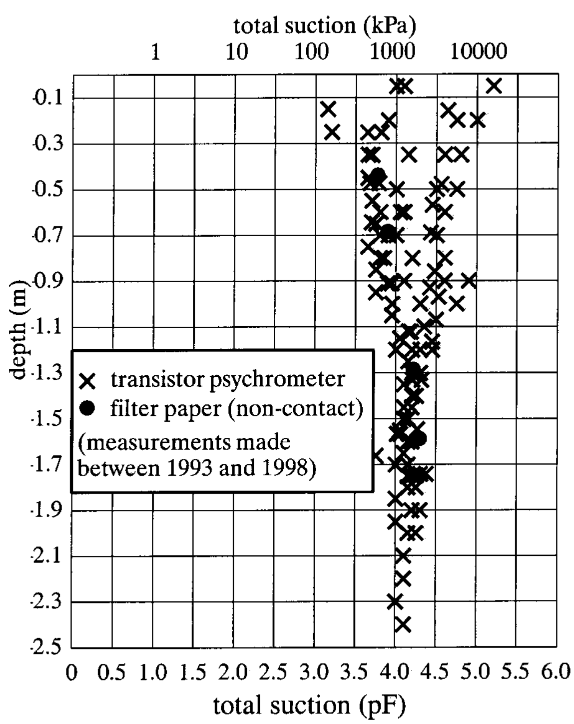


Figure 4 – Examples of total suction profile (Fityus et al., 2004)

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 micro fabric study of clay soils that compression (swelling) cracks tended to parallel ground contours and dip into the slope at c. 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 these structures 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 5.

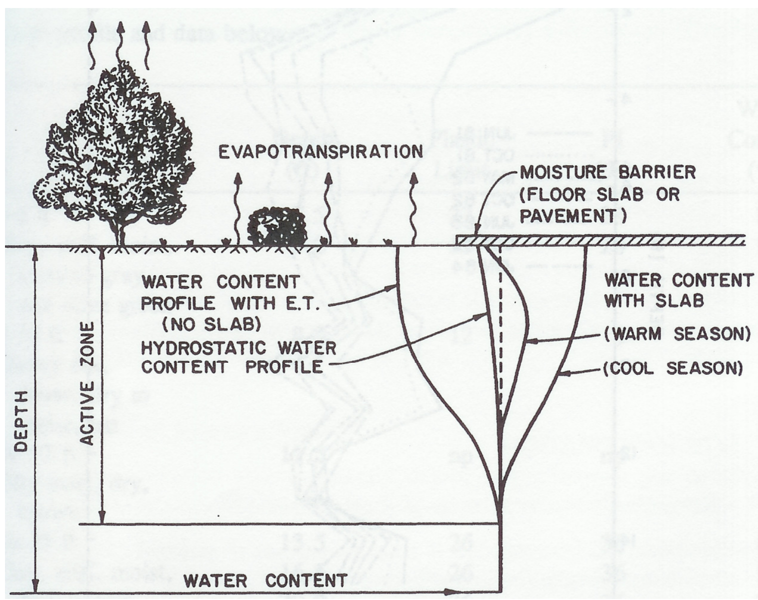


Figure 5 – Water content profiles in the active zone (Nelson and Miller, 1992)

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 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 significant with climate conditions with depths 5 to 6m in some countries where as in the UK 1.5m to 2m is typically what is seen (Biddle, 2001). If, however, the drying is greater than rehydration then the depth of this zone will increase, with 3 to 4m having been observed in some cases in London Clay (Biddle, 2001). As potential changes occur as a result in climate change, these effects are likely to become more significant.

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 change

due to climatic changes at the ground surface.

3. **Depth of wetting**: The depth to which water contents have increased due 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).

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 un-weathered condition, or superficial (drift) geological strata such as glacial or alluvial material, also in a weathered or un-weathered condition. These materials constitute a significant 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 grounds recovery can continue over a period of many years (Cheney, 1986). 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 not until sometime after construction, that problems may 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 occur in response to irrigation patterns. They obsereved that deeper wetting was common with irrigation of heavily turfed areas. If ponding of water occurs at the surface then there is 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 compared to 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 to damage 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, 241, 242: Low-rise buildings on shrinkable clay soils* (BRE, 1993)
- *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 respect engineering in expansive soils is still based on art and soil characterization and so is often perceived as difficult and expensive (especially for light weight structures). Engineers use local knowledge and empirically derived procedure, although considerable research has been done on expansive soils 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 contents 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 programmes even for relatively light weight 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 universially reliable methods available. Moreover, expansiveness has no direct measure and so it is necessary to make comparison, 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 D of the Manual for further details).

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. However, it is 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 data-basing 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 & Kovacs, 1981; Oloo et al., 1987). Such empirical correlations may be based on a small data set, 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.

5.1.1 Site Investigation

A key difficultly 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 investigation of any expansive soils is a good knowledge of local geology and the use of maps provides a framework for this. These maps are particularly useful when constructingtransportation 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 probelms. Some key features are 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 evidence by surface drainage and infiltration features

(3) Environmental conditions:

- Vegetation type
- Climate

Sampling in expansive soils are generally the same as those used in conventional soils with care to ensure disturbance, e.g. through water content changes or poor control during transportation. Further details are provided in Section D of this Manual, and an overview of practices specifically used for expansive soils in other countries is provided by Chen 1988; 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.

5.1.2 In-Situ Testing

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

- Soil suction measurement using thermocouple psychrometers, tensiometers or filter paper methods
- *In-situ* density and moisture tests
- Settlement and heave monitoring
- Piezometers or observations wells
- Penetration resistance
- Pressuremeter and dilatometers
- Geophyisical methods

Expansive soils can be tested in the field using methods that rely on empirical correlation such as the standard penetration tests (SPT) or the cone penetration test (CPT) to infer soil strength parameters (Clayton et al., 1995). Initial effective stresses can be estimated using a pyschrometer (Fredlund & Rahardjo, 1993) or a suction probe (Gourley et al., 1994), to 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 use the transmission of elastic waves through the ground in order to determine its density and elastic properties (see Section XX in the manual) 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 recent Jones et al. (2009) successfully monitoring tree induced subsidence in London Clay using electrical resistivity imaging.

Monitoring should also be considered and a number of approaches can be used, common with non-expansive soils. Key methods are settlement and heave monitoring for volume change and peizometers 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 el al., 2001). In cases where the soil is not uniform or several strata exist a correction can be applied using liquidity index. Nelson and Miller (1992) provide an example of this calculation.

Examples of monitoring associated with expansive soils are provided throughout the 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). Important for any monitoring in expansive soils is stable benchmarks, and details design and installation are given in many papers, e.g. Chao et al., (2006).

Further details can be found in Section D and Section I of this manual or specific discussion in context of expansive soils see Chen (1988) and Nelson and Miller (1992).

5.1.3 Laboratory Testing

Considerable research work has been carried out on behalf of the oil and mining industries, especially in the USA, 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 greater 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 phenomenon 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 test in the laboratory it is important to distinguish swelling in compacted, undisturbed and reconstituted samples, due to significant differences in their respective fabrics. This is lead to significant differences in an expansive behaviour measured as in particular measured swell is highly sensitive to changes in fabric

5.1.3.1 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 (1-D) or volumetric, i.e. three dimensional (3-D). Swelling pressure tests are almost always one dimensional and traditionally used oedometer type of testing arrangements (Fityus et al., 2005). However, shrinkage tests deal solely with the measurement of shrinkage strain in either 1-D or 3-D.

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, 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 problems 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 odemeter being the most commonly used and 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 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 vs. applied stress (e-logp) curve is referred to as the swelling index (Cs); that is the rebound or decompressional equivalent of the compression index (Cc). It is common however, for measured swell potential to be low to medium when soil units across a region have high potential as this is the result of natural soil variability (Houston et al., 2011)

5.1.3.2. 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 & Coleman, 1967), dielectric dispersion (Basu & Arulanandan, 1974), disjoining pressure (Derjaguin et al., 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. Physico-chemical and diagenetic bonding forces probably dominate in these materials whereas capillary forces are negligible. It is likely that the distance between clay platelets and 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.

5.1.3.3.Use index tests

The Volume Change Potential (VCP) (or Potential Volume Change, PVC) of a soil is the relative change in volume to be expected with changes in soil moisture content and is reflected by shrinking and swelling of the ground. That is, the extent to which the soil shrinks as it dries out, or swells when it gets wet. However, despite the various available test methods for determining these two phenomena, e.g. BS 1377, 1990: Part 2, tests 6.3 & 6.4, Shrinkage Limit and 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 data-basing 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 & Kovacs, 1981; 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.

However, 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, 1993) for use where the particle size data, specifically the fraction passing a 425µm sieve, is known or can be assumed as 100% passing (Table 2).

Table 2 – Classification for shrink–swell clay soils (BRE, 1993)

I_p' (%)	Volume Change Potential
> 60	Very high
40 - 60	High
20 - 40	Medium
< 20	Low

Where: $I_p' = I_p \times (\%<425\mu m) / 100\%$

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 3), which uses the same modified I_P' approached as presented in Table 2.

Table 3 – Classification for shrink–swell clay soils (NHBC, 2011a)

I_p' (%)	Volume Change Potential
> 40	High
20 - 40	Medium
10 - 20	Low

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, and that this potential is not necessarily realised in a given moisture change situation. These do not therefore represent fundamental properties of a soil. However, potential may be described differently. For example, swelling potential is described by Basu & 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. Of these, some are more relevant than others. These methods are covered in detail in Jones (1999), where the positive and negative points 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).

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 stress strain behaviour of each soil profile. Initial stress states and constitutive properties can be evaluated uses a suite of approaches (highlighted by many text, e.g., Fredlund & Rahardjo,1993, Prowrie 2004) but it is the final stress condition that must usually be assumed. Guidelines are presented by Nelson and Miller (1992), with calculation based on knowledge of effective overburden stress, the increment of stress due to applied load and soil suction. However, each situation requires engineering judgement and consideration of environmental conditions at each site.

Details of constitutive relationships for expansive soils have been reviewed and a useful description of these is also 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 test methods used to determine key soil parameters is provided in Fredlund & Rahardjo,(1993) and Fredlund 2006.

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

A number of heave prediction are available that are based on oedemeter test 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 (10) provide an illustration using free-field heave prediction and their use in foundation design, as well as methods for prediction heave rate..

5.2.1. Oedometer based methods

Oedoemeter based test include one-dimensional oedoemter tests and double odeometer tests (developed by Jennings and Knight, (1957)). Double oedometers 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 re-compress the swollen sample to its pre-swelling 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 6, with σ_0' representing the stress when inundation occurred and σ_S' representing the stress equated to swelling pressure.

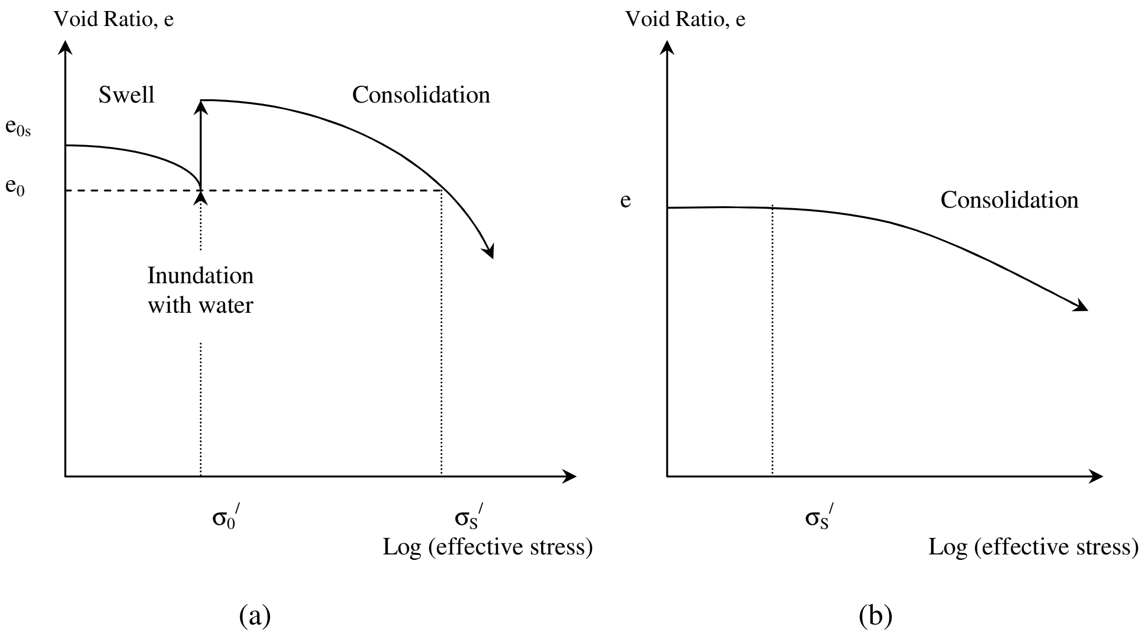


Figure 6 – Typical odeometer swell test curves: (a) An illustration of a free swell test result, (b) An illustration of constant volume test results.

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 & Rahardjo (1993) suggest simplifications to facilitate predicts using parameters measured by constant volume oedometer tests (pressures increase during swelling to maintain constant volume) using established techniques. This is illustrated in Figure 7.

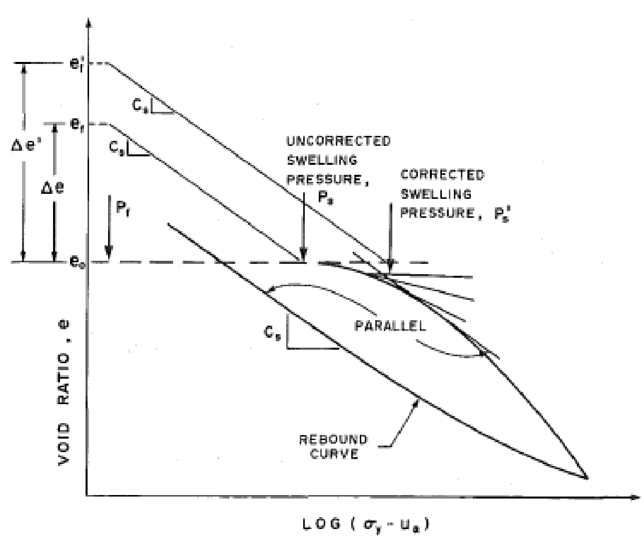


Figure 7 – One-dimensional oedometer test results showing effect of sampling

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, $P_s^{/}$ (Rao et al., 1988)

Fityus et al. (2005) question this 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 specimen is fully wetted are conservative as full saturation is not often reached in full in the field (Houston et al., 2011). Thus swell test based on submerged samples at the level of stress of interest will over predict heave. The effect of partial wetting may be as important as the depth to which wetting has occurred Fredlund et al. (2006).

5.2.2. Suction based tests

Suction tests are used to predict soil response in much the same manner as with response to saturated effective stress changes. Various methods have been developed, e.g. US Army Corps of Engineers (WES) method or the CLOD methods, details of which, including advantages and limitations can be found in Nelson and Miller (1992). Fredlund and Hung (2001) have subsequently developed suction based prediction to evaluate volume changes from both environmental and vegetation change 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. However, these approaches have a number of advantages as a means to estimate soil suction and hence suction profiles (see Figure 4).

For this reason increasingly numerical and semi-empirical methods use the soil water characteristic curves (SWCC) (Pappala et al., 2006). The SWCC describes the relationship between water content (either gravimetric or volumetric)and soil suction. Alternatively the SWCC can be used to describe the relationship between degree of saturation and soil suction. A more detailed discussion and examples of typical SWCC curves is also provided in Chapter C9 of this manual).

Only a limited of 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 and Puppala et al. (2006) details SWCC for both treated and untreated expansive soils. Further details of this are provide by Fredlund & Rahardjo,(1993) with Nelson and Miller (1992) provide details of this in 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). Often heave is estimated by integration of strain over zone in which water contents change. However, uncertainty occurs and results 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 changed coupled to prediction of the actual depth and degree of wetting that will occur in the field, both of which is 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 an arid/semi arid area to those undertaken using numerical approaches (in this case simple 1-D and 2-D unsaturated flow model) with details of site drainage and landscape practices also considered. Comparison was made after 1 year and concluded that drainage conditions was the more important factor in the prediction of foundation problems. This study revealed that the effect of poor drainage and roof run off ponding near a structure is the worst case scenario. Thus uncontrolled drainage and water ponding near foundations lead to significant suction reduction to greater depths (0.8m was found after 1 year in their study) with differential soil swell and foundation movement the result, see Figure 8.

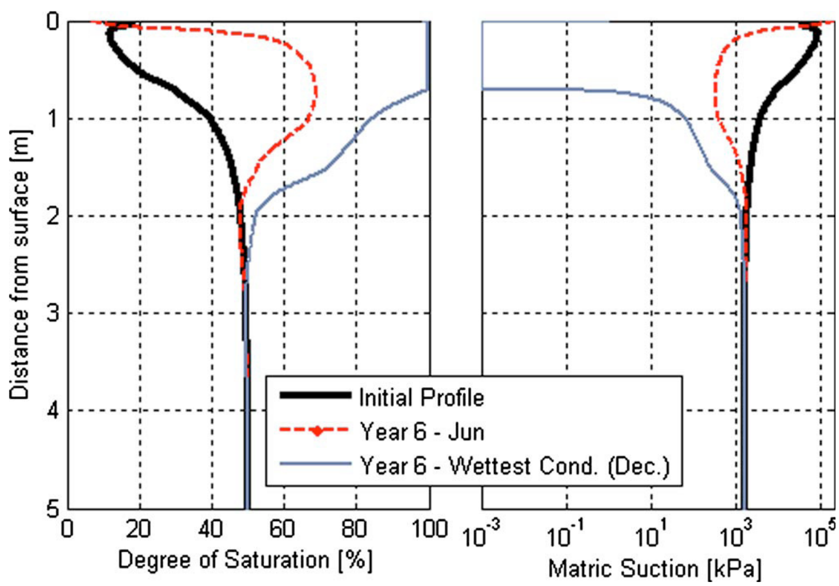


Figure 8 – Profile for 1 year of roof run-off water ponding next to foundation after 6 years of desert landscape. Wettest and driest conditions in 1-D (Houston et al. 2011).

5.2.3. Numerical approaches

1-D 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 1-D 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.

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 et al., 2004: Yilmaz 2006) or from specific surface areas (Yukselen-Aksoy and Kaya, 2010), but these are as yet to be adopted. Examples of various schemes commonly used around the world are illustrated in Figure 9. Core to the various schemes that have been developed is the lack of standard definitions of swell potential, since both sample conditions and testing factors vary over a wide range of values (Nelson and Miller 1992).

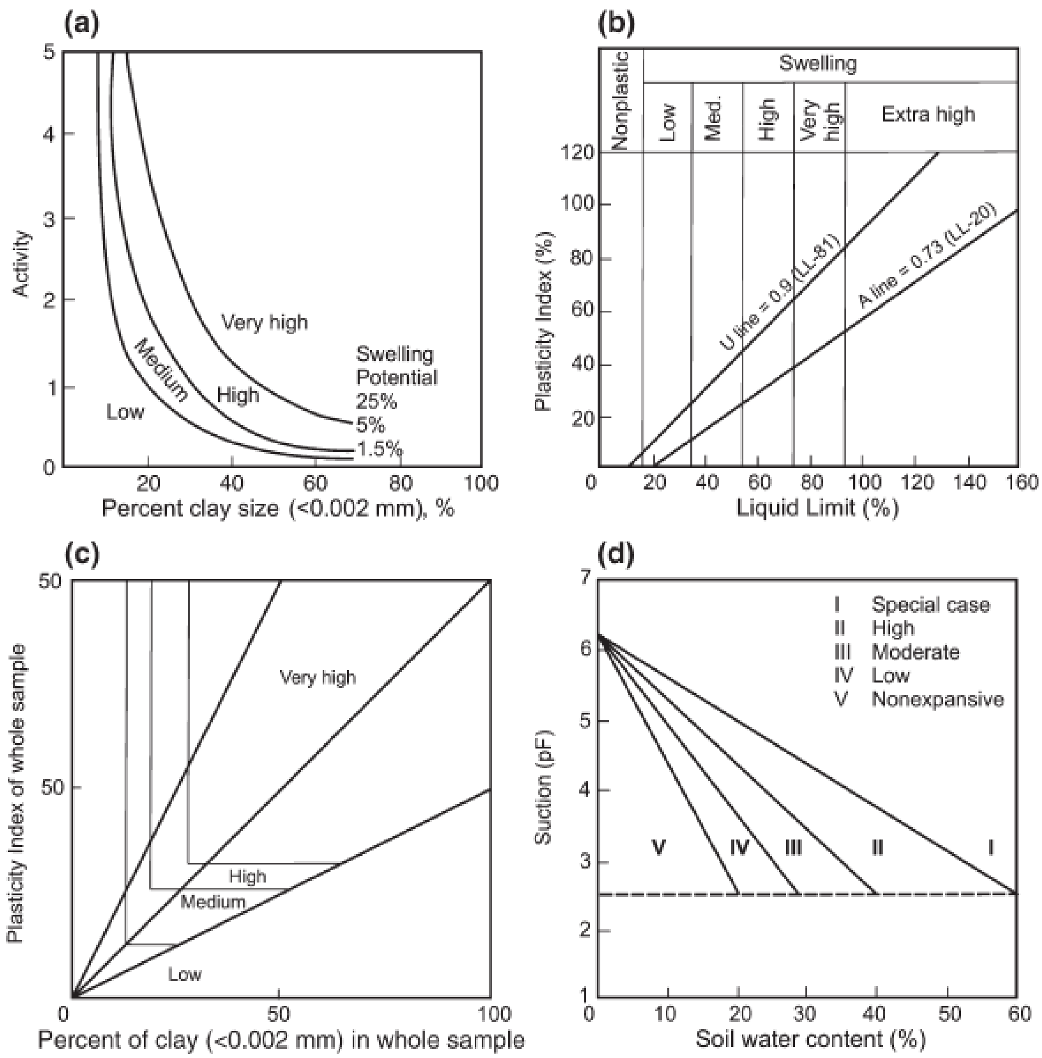


Figure 9 – Commonly used criteria for determining swell potential from across the world (Yilmaz, 2006)

5.3.1. Classification schemes

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

- Free swell (see Holtz and Gibbs, 1956 for further details)
- Heave potential (see Vijayvergiya & Sullivan (1974) and Snethen et al. (1977) for

further details)

- degree of expansiveness (see US Federal Housing Administration (FHA, 1965) and Chen (1988)).
- shrinkage potential. Altmeyer (1954) Holtz & Kovacs (1981)

However, 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 the relation between swelling potential of clay and 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 schemes relate to expansion potential, based on the Skempton 'Activity' plot (Skempton, 1953) and its development by Williams & Donaldson (1980) from Van der Merwe (1964). Details are described in Taylor & 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 USA (see Chen 1988; Nelson and Miller 1992), most of which use swelling and suction as their basis (Snethen, 1984). Sarman et al. (1994) concluded 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 these samples' ability to absorb and adsorb water. It was found that correlations between swelling and more than parameter were unsuccessful.

With all classification schemes only indications of expansion are obtained with in reality field conditions varying considerably. Such rating can be of little use unless the user is familiar with the soil type and test conditions used to develop the rating. Ratings themselves can be misleading and if used with design options outside the region where rating 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 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).

5.3.2 UK approach

Whilst much study has been carried out world-wide to infer swelling and shrinkage behaviour from soil index properties such as plasticity (section 5.1.1.3 see above), few direct data are available in UK geotechnical databases (Hobbs et al., 1998). Two schemes that are commonly used within the UK, and are based on BRE and NHBC schemes

Volume change potential has been more recently defined for over-consolidated clays, in terms of a modified plasticity index term (I'_p), by Building Research Establishment Digest 240 (BRE, 1993) – see Table 2. This classification aims to eliminate discrepancies due to particle size where, for example, glacial till and other well graded soils are concerned.

High shrinkage potential soils may not behave very differently from low potential soils 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 3. 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 (1993) suggests that plasticity index and clay fraction can be used to indicate the potential of a soil to shrink, or swell, as follows:

PI (%)	**Clay Fraction (<0.002 mm)**	**Shrinkage Potential**
>35	>95	Very High
22 – 48	60 – 95	High
12 – 32	30 – 60	Medium
<18	<30	Low

Overlap of categories reflects fact that figures were obtained from multiple sources.

5.3.3. National v Regional characteristics

A meaningful assessment of the shrink–swell potential of the UK requires a considerable amount of high-quality and well-distributed spatial data, of a consistent standard throughout. 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 Volume Change Potential (VCP) of a soil provides arelative change in volume to be expected with changes in soil water content. This was calculated from the Ip' values and a classification made based on the *upper quartile* value (Table 4). This draws off the BRE (1993) scheme shown in Table 2.

Table 4 – Classification of VCP

Classification	**Ip' (%)**	**VCP**
A	< 1	Non-Plastic
B	1 – 20	Low
C	20 – 40	Medium
D	40 – 60	High
E	> 60	Very High

In this way a VCP was assigned to each of the geological units and a map of shrink–swell potential built (Figure 10).

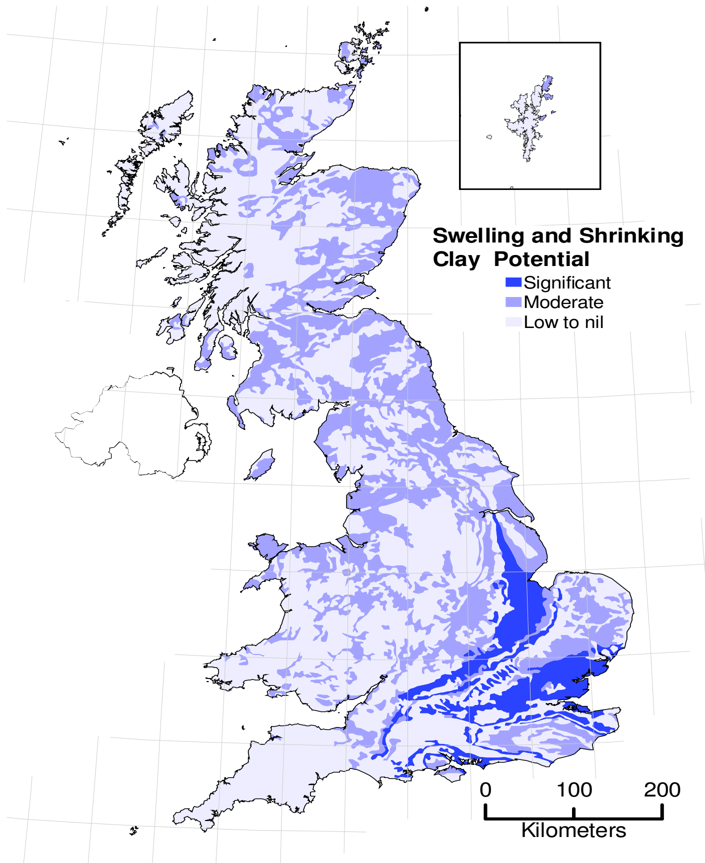


Figure 10 – Shrink–swell potential map, based on VCP (Jackson, 2004).

Looking at clays on a national scale can give a good indication of the potential problems associated with them and provide initial information regarding planning decisions. However, no two clays 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 over the last 50 years (Skempton & Delory, 1957; Chandler & 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 & Terrington (2011) follow the methodology described in Diaz Doce et al. (2011) using 11,366 samples across the London Clay outcrop, splitting it into 4 distinct areas, based on geographical location, plasticity values and depth of overlying sediment. In this way a more detailed assessment of the outcrop was able to be carried out, and a 3-D 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 11). This type of analyses indicate that 3-D modelling methods have considerable potential for predicting the spatial variation of VCP within expansive clay soils, so long as they large enough data sets.

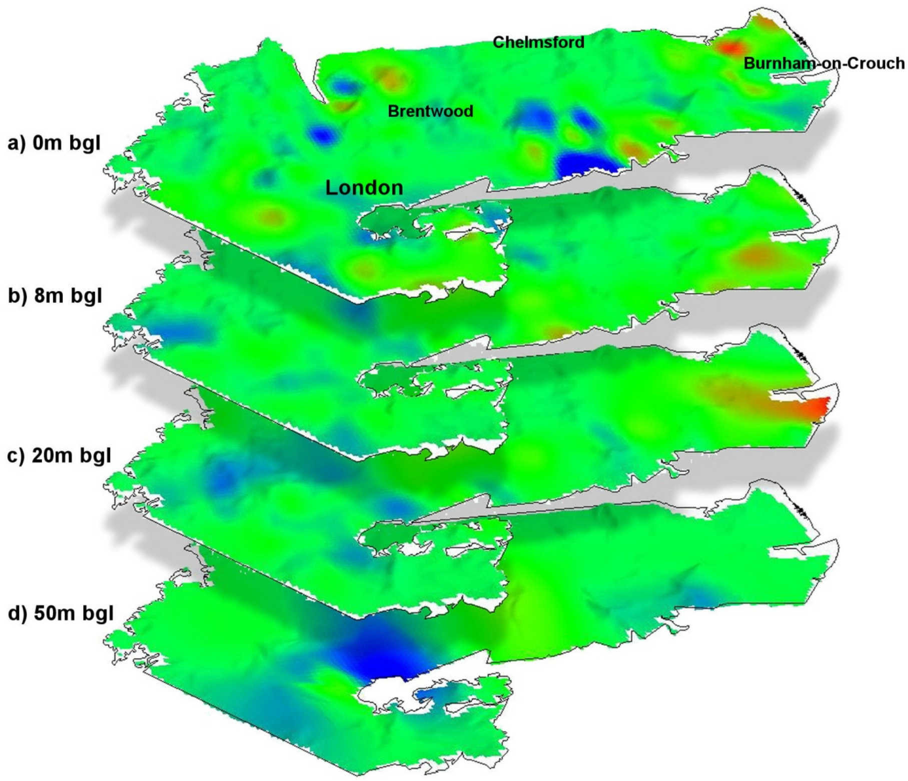


Figure 11 – S-Grid interpolations for Area 3, showing surfaces at 0m, 8m, 20m and 50m bgl (Jones & Terrington, 2011). [blue-medium, green-high, yellow/red-very high VCP].

5.4. Specific problems with expansive soils

The principle adverse effects of the swell/shrink process arise when either swelling pressures resulting in heaving (or lifting) of structures or shrinkage leading 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 provide by NHBC (NHBC 2011a). Below a summary is provided highlighting the key features associated with these (see Section 5.4.1 to 5.4.4). In addition discussion of some of the key issues faced in the UK is provided (see section 5.4.5.) where impact of vegetation is often the major cause of soil-structure problems faced with expansive soils.

5.4.1. Foundation options in expansive soils

A large number of factors influence foundation types and design methods (see Section E of the manual for further details), and these included aspects such as climate, 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 stakeholder is essential. Often higher initial costs are offset many times over by a reduction in post construction maintenance costs when dealing with expansive soils (Nelson and Miller, 1992).

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

(i) Use of structural alternatives, e.g. stiffened raft
(ii) Use of ground improvement techniques
(iii) A combination of (i) and (ii)

As with any foundation option the main aim is to minimise effects of movement, principally differential, and 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 raftsand modified continuous perimeter spread footings and these are summarised in Table 5.

Table 5 – Foundation types used in expansive soils (after Nelson and Miller, 1992; NHBC 2011a)

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 needed 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 raftfoundation – includes stiffened perimeter beams	Simple construction with no specialist equipment needed	Ineffective in highly expansive soils or within the zone of influence of tress

In addition a brief description is given below. However, further details are provided by Chen (1988) Nelson and Miller (1992) and NHBC (2011a, 2011b, 2011c). It should be noted that terminology used to describes the foundation types listed in Table 5 very across the world, with for example, slab-on-grade used in the USA for raft foundations..

5.4.1.1. Pier and Beam/ Pile and BeamFoundations

These foundations consist of a ground beam used to support structural loads, transferring the load to the piers or piles. Between the pier/pile and ground beam a void is provided 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 whole length to avoid tensile failures) concrete shafts with or without belled bottoms, steel piles driven or pushed, or helical pile 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 surface pier/pile can be designed as rigid anchoring members. If however, the depth of potential swell is high, piers/piles should be designed as an elastic member in an elastic medium. Figure 12 illustrates a typical pier and beam foundation from US practice. However, very similar arrangements are use in the UK and are illustrated in NHBC (2011a) see Figures 10 and 11 in this NHBC document).

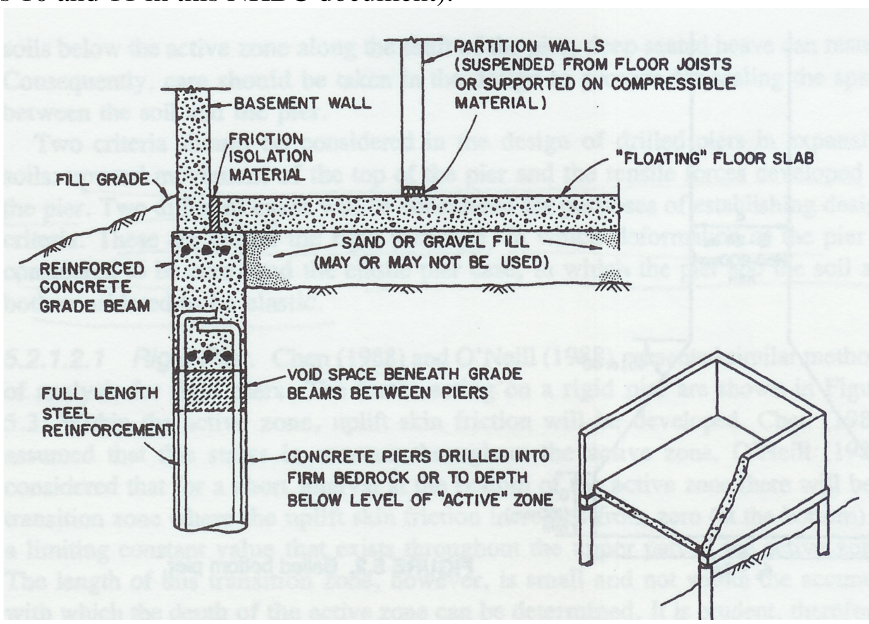


Figure 12 – Illustration of a Pier and beam foundations (Nelson and Miller, 1992)

Design and construction procedures for each of these systems in provided in detail (including sample design calculations) by Chen (1988) and, Nelson and Miller (1992) with additional discussion and example design calculations provided by Nelson et al. (2008/9??). It is important though to ensure sufficient anchorage below the active zone. Pier/pile diameters are kept small (typically 300 to 450mm diameters). Any smaller and problems will result poor concrete placement and associated defects, e.g. void spaces. Other problems that can occur include 'mushrooming' near the top of the pier/pile, which if occurs provides added area for uplift forces to act. To avoid this, in many countries 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 between 150mm to 300mm 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 approach used does not provide potential pathways to allow water to ingress to deeper layers as this will cause deep seated swelling.

5.4.1.2. Stiffened Rafts.

Stiffened slabs are either reinforced and commonly in countries like the US are post tensioned systems. 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 13, which shows examples used commonly in the US. Similar approaches are used in the UK and are presented in NHBC (2011a; 2011c).

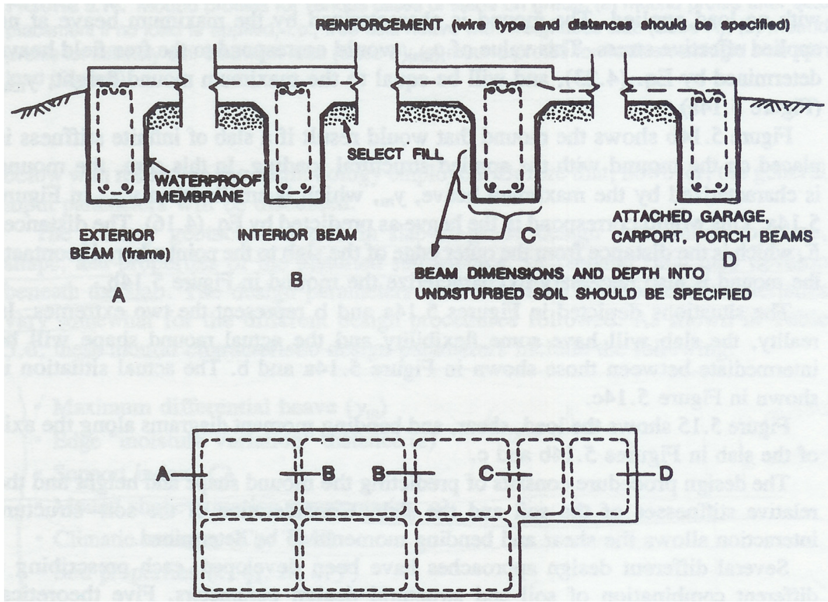
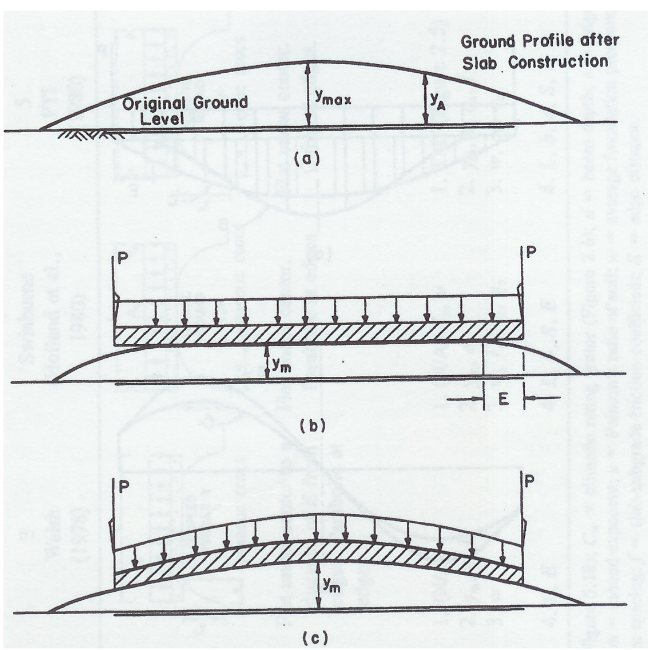


Figure 13 – Typical detail of a stiffened raft (Nelson and Miller, 1992)

Design is modelled on 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 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 actual heave produced by swelling soils lies somewhere between these two extremes. These three modes of movement are illustrated in Figure 14.



Note:
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 contact foundation;
P = loading
y_A= height of free field heave along ground profile

Figure 14 – Profiles after construction for various stiffness of raft: (a) profile with no load applied; (b) Profile with infinitely stiff slab; (c) Profile with flexible slab (Nelson and Miller, 1992)

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).

However, the primarily geotechnical information required includes size, shape and properties of the distorted soil surface that develops below the slab. This depends on a number of factors including: heave, soil stiffness, initial water content, water distributions, climate, time post construction, loading and slab rigidity. It should be noted that the slab through its elimination of evaportranspiration (see Figure 5) promote the greatest increase in water content near to the centre of the slab and hence where long term distortion are most severe. However, the maximum differential heave (y_m in Figure 14) has been found to vary between 33 to 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 areas.

5.4.1.3. 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
- Provide void spaces within support beam/wall to concentrate loads at isolated points
- Increase 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 place in a non-expansive layer that overlies expansive soils, provided:

- Soil is consistent across site
- Depth of non-expansive material is greater than ¾ of the equivalent foundation depth assuming all soil is expansive (guidance provide within NHBC 2011a)
- The thickness of the non-expansive soil below they foundation at least eaqual to the foundation width

5.4.1.4. Case studies

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

(1) Changes is 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
- Void space inadequate

(3) Lack of appreciation of soil profile

- Underlying geology contains inclined bedding of bedrock so 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.

However, 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 gain from these studies. Ewing (2011) provides an interesting case fpm 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 1.5m of non-expansive soils overlying expansive clay some 8m think.

5.4.2. Pavement and expansive soils

Pavements are particular vulnerable to expansive soil damage with estimates suggestion that approximately half of the overall costs from expansive soils are associated with pavements (Chen 1988). Their inherent vulnerability stems from their relative light weight nature extended over a relatively large area. For example Cameron (2006) described problems with railways built on expansive soils where poor drainage exists, with Zheng et al. (2009) providing details of highway sub-grade construction on embankments and in slopes, in China. 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 subgrade soils (see Section 5.4.3. below for further details). Pavement designs are considered based on either flexible or rigid pavement systems or procedure for such have been discussed in Section G – chapter G8 of the manual. However, when dealing with expansive soils a number of approaches should be considered and include:

i. Choose an alternative route and avoid expansive soil;
ii. Remove and replace expansive soil with a non-expansive alternative

iii. Design for a low strength and allow regular maintenance
iv. Physically alter expansive soils through disturbance and re-compaction
v. Stabilisation through chemical additives, such a lime treatment
vi. 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 testing undertaken to mitigate expansive soil behaviour for pavement construction. Cameron (2006) has advocated the use of tree as they can be beneficial in semi arid environments to managed poorly drained areas under railways. However, this need careful management and may require several years to realize full effects.

5.4.3. Treatment of expansive soils

Essentially treatment of expansive soils can be grouped under two categories:

(1) Soil Stabilisation – removal/replacement; remould and compact; pre-wetting, 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 F of the Manual) and a brief discussion pertinent to expansive soils has been discussed above, see Section 5.1. Special consideration should be given to: depth of active zone; potential for volume change; soil chemistry; water variations within the soil; permeability; uniformity of the soils and project requirements. A brief overview of each of the two categories of treatments applied to expansive soils is provided below, with Table 6 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 across the world. 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 foundation placed on expansive soils, e.g. pavements (Murty and Praveen, 2008). Swell mitigation has also been achieved by mixing non-swelling material into an expansive soils to dilute swell potential, e.g. sand (Hudyma and Avar, 2006) or granulated tyre rubber (Patil et al., 2011).

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 Table 6 – remove and replace techniques) to counteract expected swell pressures. Thus this method is 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.

Table 6 – Soil Stabilisation approaches applied to expansive soils (Nelson and Miller, 1992)

Improvement approach	Outline of approach	Advantage	Disadvantage
Removal & 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 to 1.3m.	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 & compaction	Less expansion observed for soil compacted at low densities above OWC[a] than those at high densities and below OWC[a] (see Figure 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 reduce 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 may be used and vertical drains can be used to also 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 depth less than the active zone; Water redistribution can occur causing heave after construction.
Chemical Stabilisation	Lime (3 to 8% by weight) common with cements (2 to 6% by weight) sometimes used, and salts, fly ash and organic compounds less commonly used. Generally lime mixed into surface (~300mm), sealed, cured and than 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; Is effective is reducing plasticity and swell potential of an expansive soil.	Soil chemistry may be detrimental to chemical treatment; Health and safety needs careful consideration as chemical stabilisers carry potential risks; Environmental risk may also occur – e.g. quick lime is particularly reactive; Curing inhibited in colder temperatures.

[a] OWC – optimum water content as determined by standard proctor test, BS1377: 1990.

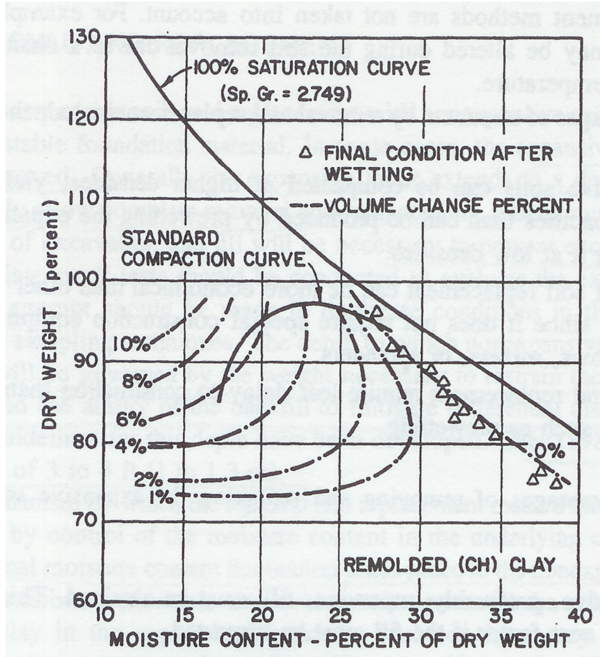


Figure 15 – Percentage expansion for various placement conditions (c.f. Table 6) (after Holtz and Gibbs, 1959)

Fluctuations in water contents are one of the primary causes of swell/shrink problems, with non-uniform heave occurring due to non-uniformity of water contents, 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 movement 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 minimising seasonal fluctuation effects.
(2) Lengthen the time for water content changes to occur due to longer migration paths under foundations.

Techniques used include:

- Horizontal Barriers – using membranes, bituminous membranes or concrete.
- Vertical Barriers – polyethylene, concrete, impervious semi-hardening slurries

A detailed account of these are provided in both Chen (1988) and, Nelson and Miller (1992). In addition to these, 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. (2008). Further to 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 occurs.

5.4.4. Remedial options
Expansive soils cause significant damage to building as discussed throughout this chapter and so remedial action is required to repair damage caused. 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 is it better to wait?
- Who pays?
- 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?

Clearly, to select an appropriate remedial measure an adequate forensic site investigation is required. Key information needed includes: cause and extent of damage; soil profile (as it is often difficult to determine whether settlement/heave is the cause of structural distress) and the soils expansive potential. Other information required have been discussed above, see Section 5.1. However, 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 Digest 251, 298, 361, 412, 471.

Examples of remedial measures employed for foundations include:

- Repair and replace structural elements or correct improper design features
- Underpinning
- Provide structural adjustments of addition structural support 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 see as unnecessary) and it is more common for underpinning work to applied to only 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 de 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. A detailed experimental study, Buzzi et al. (2010) however, concluded that resin injected expansive soils did not exhibit enhanced swelling as a number of crack remained unfilled providing swell relief. Problem with lateral swelling can be sometimes accommodated by cracking within the soil matrix. However, if no cracks are presence problems can occur, particularly with retaining structure. 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 as highlighted above in section 5.4.2. Most common remedial measures are either removal and replacement or construction of overlays. Whichever method is used care is needed to ensure causes of the original distress are dealt with.

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

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, depth to 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 16).

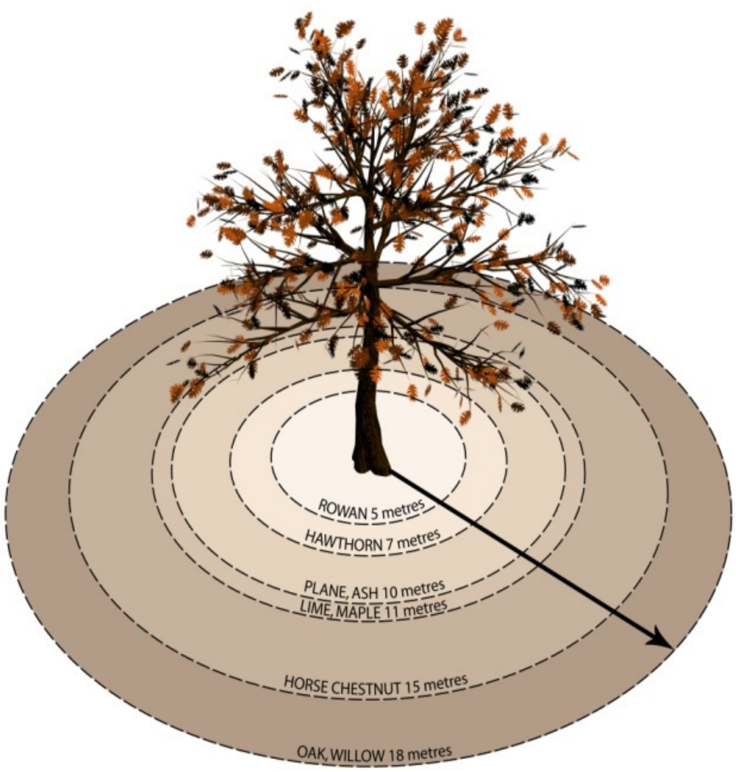


Figure 16 – The zone of influence of some common UK trees (Jones et al., 2006).

If an impermeable method of paving is used, it may prevent water from penetrating into the ground. This can affect 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 that might be expected. 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 or remediating existing damaged buildings. New build guidelines in the terms of domestic dwellings recognises the need for thorough ground investigations to design systems to cope with the hazards presented by presence of existing trees or building following their recent removal. Reference should be made National House Building Council (NHBC) Standards Chapter 4.2 Building Near Trees (NHBC 2011a) and the Efficient Design of 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 Digest 298, 412) and a summary of good Technical Practice guide provided by Driscoll and Skinner (2007).

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

- Shrinkage/heave linked to changes in water contents
- 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 the terms of vertical movement may result in little by way of structural damage (IStructE 1994). However, in the UK this is rare and by far the most overwhelming cause of damage to property results from the desiccation of clay subsoil, which as a consequent 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 usually in September, followed by upward recovery in the winter, see Figure 16. If it is occurring, there is no need to try to demonstrate shrinkable clay or desiccation as no other cause produces a similar pattern – soil drying by vegetation must be involved (unless the foundations are less than 300mm). Furthermore, there is no need to demonstrate the full cycle as it is just 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 Chattenden, Kent, set in expansive London Clay (see Figure 16). In addition they provide details of instrumented piles, discussing design implications.

Level monitoring can demonstrate this pattern. BRE Digest 344 (1995) makes recommendations for the taking of measurements of the 'out-of-level' of a course of masonry or of the DPC can be made to estimate the amount of differential settlement or heave that has already taken place. BRE Digest 386 (1993) 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).

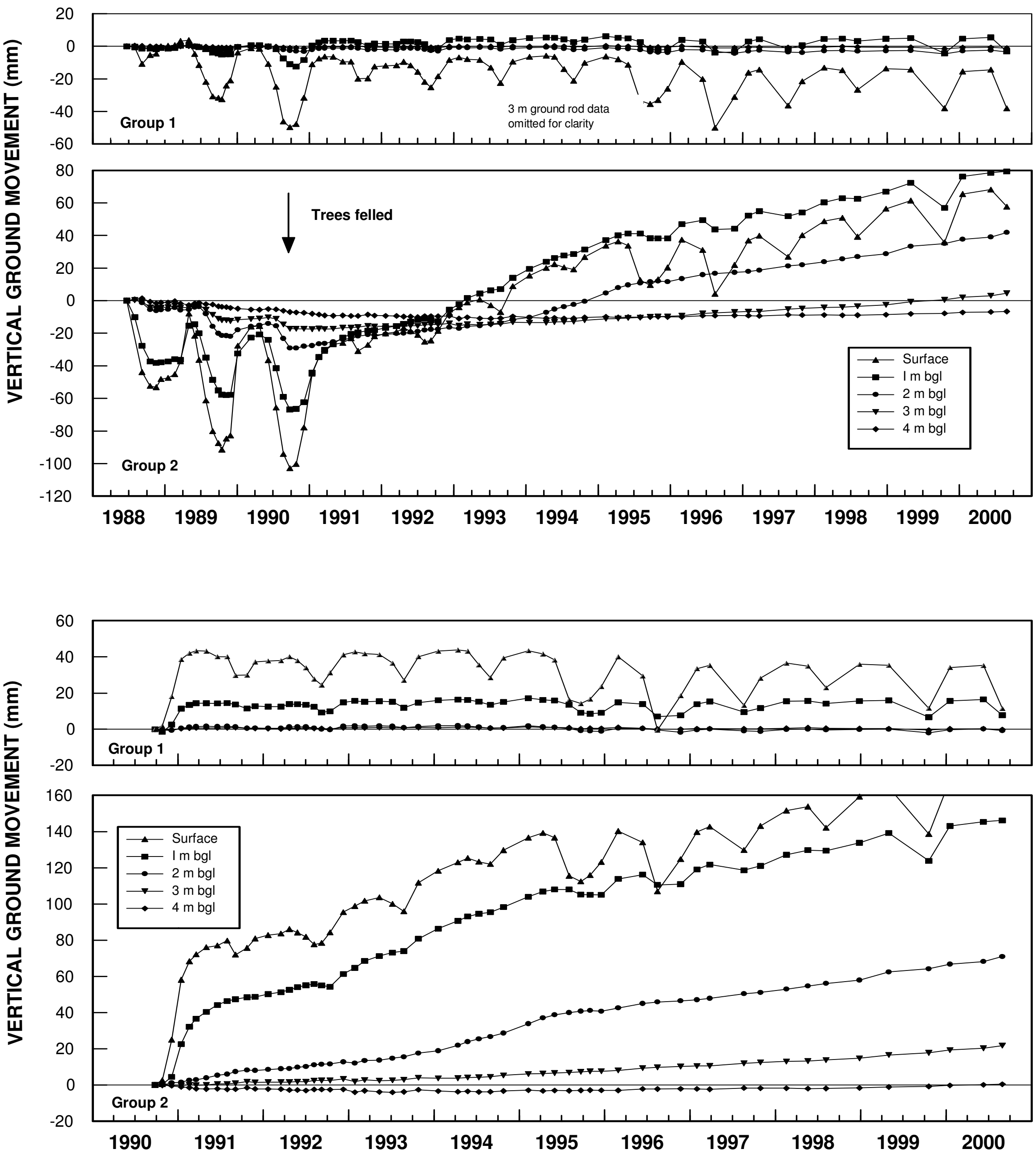


Figure 16 – Examples of round movements due seasonal fluctuations at Chattenden. The upper plot shows results obtained since the first movements in June 1988. The lower plot shows an enlarged scale with results obtained since the trees were felled – group 1 is remote from tree and group 2 near to trees (Crilly and Driscoll (2000); Driscoll and Chown, 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 tree to reduce drying and the amplitude of seasonal movement
(3) Control root spread to prevent drying under foundations
(4) Provide supplementary watering to prevent soil from drying

Biddle (2001) states it is now recognized that in most situations underpinning is unnecessary and that foundations can be stabilized by appropriate tree management, usually felling the offending tree or 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 need for any other site investigations.
- If the tree warrants retention, assessment of whether partial underpinning would be sufficient.
- Confirmation that vegetation management has it been effective in stabilising the foundations.
- Provision of information within an acceptable timescale.

Tree pruning to reduce its water use and therefore its influence on the surrounding soil is often employed. However, if the trees are thereafter not subjected to a frequent and on-going regime of management the problem will very quickly return and problems continue. Whilst 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 e.g. constructed from in-situ concrete. However, such barriers often prove ineffective over time. Barriers are currently being developed that incorporates a bioroot-barrier, which is a mechanically bonded geocomposite consisting of a copper-foil firmly embedded between 2 layers of geotextile. Specifically used in Arboriculture and Japanese knotweed control where a permeable barrier is required. The biobarrier acts as a signal barrier diverting root growth both biologically and physically rather than any attempt to physically restrain their progress.

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

If a mature tree is felled heave building on a clay soil that is dry can occur. Unfortunately the evidence is rarely evident. However, a number of clues can help and 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 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 pre-dates the planting of the tree.

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

6. Conclusions

Expansive soils are one of the most significant ground related hazards found globally, contributing billions of pounds annually. Expansive soils are found throughout the world and are commonly found in arid/semi arid regions where their high suctions and potential for large water content changes on exposure/deficient with water can cause significant volume changes. In humid regions such as the UK problematical expansive behaviour is generally occurs in clays of high I_p. Either way, expansive soils have the potential to demonstrate significant volume change 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 influence 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 expansive behaviour. 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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