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The Shrink Swell Test

ABSTRACT: Although the assessment of the expansive potential of clay soils has been the subject of active research for the past 40 years, its treatment in routine geotechnical practice around the world remains inconsistent. This paper describes the shrink swell test, which is used routinely in Australian geotechnical practice as the principal method for the experimental assessment of the expansive potential of clay soils. The test procedure and its underlying assumptions are described and discussed in the context of the historical development of the test and its routine application. It is shown that the shrink swell test is a simple and economical means of assessing soil expansiveness, which is achieved largely through the adoption of several simplifying assumptions that effectively circumvent the measurement of soil suction. The significance of these assumptions is discussed, and it is concluded that the shrink swell test can be conveniently and reliably employed to guide the routine design of foundations in expansive soils.

KEYWORDS: expansive soil, ground movement, shrink, swell, shrinkage index

Introduction

Expansiveness (also referred to as reactivity) is a phenomenon that affects many clay soils, particularly those that contain significant quantities of smectite clay minerals. In many cases, the shrinking and swelling of expansive clays in response to moisture content change can be a serious cause of damage to residential buildings (Jones and Holtz 1973; Krohn and Slossen 1980; Freeman et al. 1991). Yet despite this potential to cause costly damage, in vast areas of many countries throughout the world, its treatment in geotechnical engineering practice is inconsistent (being undertaken in what could be described as an ad hoc basis).

The shrink swell test is a simple test that is routinely employed in Australian geotechnical engineering practice to assess quantitatively the expansive potential of undisturbed or remolded clay soils, and to guide design of footings on these soils. However, despite its relatively successful adoption in Australia over the past 20 years or so, its existence and use within Australia are not widely known by the international geotechnical community. The method employed in Australia has never been described in international literature. The purpose of this paper is to raise international awareness of a method of assessing expansive soils that is broad enough in its scope to be able to be applied successfully and in a general way, to soil conditions as widely varying as those found in Australia.

Background

Expansive soils have long been recognized as important problem soils in geotechnical engineering. The significance of these soils throughout the world became prominent in 1965 in a conference that was to become the first in a series of seven conferences between 1965 and 1992. In 1995, the focus of these conferences shifted slightly toward the broader topic of unsaturated soils, and although the attention given to truly expansive soils may seem to have diminished, active research is still recorded in many parts of the world.

Despite 40 years of active research, expansive soil engineering remains one of the most inconsistently treated areas in international geotechnics. There are many different countries around the world where expansive soils are an issue, and almost as many different approaches used to treat them. Indeed, even within large countries like the USA, expansive soil engineering practice varies on a State by State basis.

In most cases, the testing methods used to assess the expansiveness of clay soil have been developed to obtain parameters for expansive soil behavioral models. The approaches that have been developed to estimate soil volume changes can generally be described as being based on either one-dimensional oedometer tests, or on soil suction/moisture content tests (Nelson and Miller 1992). A third group, which infers expansive potential on the basis of physio-chemical properties, such as molecular adsorption or cation exchange capacity (Cocka and Birand 1993; Fityus, Smith, and Jennar 2000) have received much less attention.

Many of the approaches employing results from oedometer type tests have been developed in particular geographic regions with specific expansive soil problems, although they appear to be applicable in general situations. Most have focused on the prediction of heave using a constitutive theory based on changes in effective stress, soil suction, void ratio, and oedometer testing methods. Early models include Bishop (1959), Croney et al. (1958), Coleman (1962), Richards (1966), Bishop and Blight (1963), Burland (1965), Matyas and Radhakrishna (1968), Barden, Madedor and Sides (1969), Aitchison and Woodburn (1969), and Aitchison and Martin (1973). These models were variously developed in terms of one or two different stress components, including mean total normal stress, matric suction, nett normal stress, and the deviatoric stress. Application of the more sophisticated models required the estimation of up to six compressibility parameters.

Fredlund and Morgenstern (1977) considered the use of nett normal stress and matric suction as state parameters for an unsaturated

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soil, but from the perspective of multi-phase continuum mechanics. They invoked a constitutive surface describing the void ratio of a soil in terms of the nett normal stress and the matric suction. The parameters required to define this constitutive surface are derived from a variety of tests including simple consolidation tests on saturated samples, suction measurements, and suction control using a pressure plate apparatus, shrinkage-type tests, free-swell, and constant volume oedometer tests. Because of hysteresis, rigorous application of the approach required two sets of coefficients to be determined for situations involving wetting and drying, or loading and unloading of soil.

Fredlund (1980) employed corrections to account for the effects of sample disturbance and compliance of the apparatus. Fredlund and Rahardjo (1993) suggested simplifications to facilitate predictions that only require soil parameter values that can be measured by a constant volume oedometer test, using relatively well-established techniques. However, in the authors' experience of Australian geotechnical practice, this still requires specialized apparatus that would not commonly be available in a typical geotechnical laboratory.

Alonso, Gens, and Hight (1987) proposed an alternative framework to describe volume changes in nonswelling unsaturated soils. Gens and Alonso (1992) extended this general framework to include expansive soils. However, application of this model requires relatively complicated laboratory testing that involves simultaneous control of both matric suction and normal stress.

A variety of oedometer type tests were developed to determine parameters for the various volume change models listed above. Among the earliest of these was the double oedometer method of Jennings and Knight (1957). It employed the results of "free swell" and "natural water content" oedometer tests, carried out on sample pairs, with one sample consolidated from field moisture content, and the other, consolidated after first being swelled to saturation under a token load. This test gives an estimate of a parameter that can be used in the above constitutive frameworks that is believed to be free of the effects of sample disturbance. Predictions of heave using the double oedometer test are generally thought to be satisfactory (Fredlund and Rahardjo 1993). The double oedometer test was simplified by Jennings et al. (1973) to testing of a single sample.

Sullivan and McClelland (1969) proposed an approach based on a "constant volume" test. A laterally constrained sample is inundated after being preloaded to overburden stress, with subsequent volume changes being restrained by applying further load, and then the sample is rebounded after the maximum pressure is reached. The maximum applied pressure is defined as the swelling pressure for the soil, for the initial moisture content and density of the soil.

A large variety of other oedometer-based heave prediction approaches are also reported in the literature. These are summarized by Nelson and Miller (1992) and will not be discussed further here. They include the Noble method (Noble 1966), the Navy method (Navy 1971), and the USBR method (Gibbs 1973). The main criticism of most of these methods is also a failure to account for the effects of sample disturbance.

Volume change predictions based on suction (or moisture content) tests are an alternative to oedometer test based approaches. In a comparison with oedometer test methods, Johnson (1977) found that suction test methods were simpler, more economical, and more efficient. Suction test based approaches typically use swelling or shrinking tests on undisturbed samples to relate changes in suction to changes in volume. As such, they are typically less rigorous and are most suited to situations where the nett stress does not change, such as in the prediction of ground movements due to moisture changes, under constant, mean total normal stress conditions.

One of the simplest approaches arises from a consideration of the basic relationship between volumetric strain, ε_v, water content, ω, initial void ratio, e_o, particle density, ρ_s, and water density, ρ_w, in a saturated soil, as defined by the equation:

$$\varepsilon_v = \frac{\rho_s}{\rho_w}\frac{\Delta\omega}{1+e_o} \tag{1}$$

where $\Delta\omega$ is the change in water content from the initial void ratio condition.

An equation similar to (1) was employed in heave predictions by Richards (1967), who included a factor to account for the effects of lateral restraint. The term $\frac{\rho_s}{\rho_w(1+e_o)}$ effectively represents a parameter relating changes in moisture content to changes in soil volume. Hanafy (1991) also derived equations similar to (1) to relate changes in water content to changes in void ratio. Hanafy noted from his experiments that the proportionality of volume strains with moisture changes was only applicable (approximately) to the moisture range between the shrinkage limit and the equilibrium moisture content, where the equilibrium moisture content was defined as the moisture content beyond which no significant void ratio changes take place. Two further limitations of the approach embodied in Eq 1 are that the moisture content—volume change relationship breaks down in dry soil conditions, and secondly, there is no consideration of the influence of loading on volume change.

The WES method of the U.S. Army Corps of Engineers (Johnson and McAnear 1973) relates volume changes to changes in suction. This method employs an analysis in the plane of constant nett stress, using soil suctions measured by transistor psychrometer, as well as suction indices, calculated from estimates of the slope of the specific volume-water content curve of a soil. A similar approach was also adopted by Dhowian (1990).

Another commonly used suction based test method is the CLOD test (McKeen and Nielsen 1978). This method employs an unrestrained shrinkage test on an undisturbed soil sample, using a resin coating technique. Irregular lumps of soil can be used. The volume of the sample is determined as a function of its moisture content as it dries, providing a volume change index. The method has been found capable of providing good ground movement predictions (Nelson and Miller 1992), but is limited to situations where ground movements occur under relatively constant nett stress conditions.

Although the above suction/moisture based methods could reasonably be adapted to estimate volume changes in both shrinking and swelling soils, the emphasis in their development and use appears to have been mainly placed on the prediction of heave due to wetting soil profiles.

The behavior of expansive soils subjected to successive or cyclic shrink and swell events has been looked at by a number of researchers (Day 1994; Tripathy et al. 2002; Fityus and Smith 2003); However, such studies have primarily been undertaken for the purposes of examining fundamental soil behavior under such conditions, and not in the context of establishing a method for the routine evaluation of expansive soil potential.

A number of approaches have specifically been developed, more generally, to enable prediction of both swelling (or heave) and shrinkage in clay soil profiles. Hanafy (1991), realizing that measurements of expansive potential from shrinkage or swelling tests may be significantly biased by the natural water content of the sample, proposed a test involving both free swell and free shrink components. He suggested that the almost linear relationship between water content changes and void ratio changes over a reasonable range of moisture content could be used in the prediction of ground movements, although the effectiveness of the approach was not demonstrated by application to real soils. Alternatively, methods employing suction are based on the observation that changes in log suction are approximately linearly proportional to changes in soil volume, over the range of suction change commonly experienced under field conditions. The heave prediction method of McKeen (1992), based on the CLOD test, recognized this linearity and exploited it by defining its slope as an index (the "total suction-water content index") that could be used to predict soil volume changes.

In other approaches, the constant of proportionality was referred to as an instability index, I_{pt} (Aitchison 1973; Cameron and Walsh 1984*a*). In current Australian practice, the term instability index has been given a slightly different definition, although it depends upon the measured soil vertical strain for a unit change in total soil suction for a laterally unconfined sample, a property termed the shrinkage index, I_{ps}. I_{pt} is derived from I_{ps} by taking account of specific factors of the physical field situation, such as lateral stress and confinement effects. Both indices are termed "reactivity indices," as the vertical movement may be either shrinkage with increase of suction, or swelling with loss of suction.

There are a variety of simple approaches based in the instability index approach. Aitchison and Martin (1973) followed the approach of Richards (1967), and showed that in assuming that the change in water content is approximately linearly related to the change in the log of the soil suction, the approach of Eq 1 is consistent with the instability index approach. If "c" is an experimentally determined constant of proportionality between gravimetric moisture content, ω, and the logarithm of suction and ψ is a suction variable, then "c" can be expressed as

$$c = \frac{\Delta\,\omega}{\Delta\log(\psi)} \tag{2}$$

If the relationship between the swelling index, I_{ps}, the vertical strain component, ε_z, and the corresponding change in log suction is defined by

$$\varepsilon_z = \frac{\varepsilon_v}{3} = I_{ps}\Delta\log(\psi) \tag{3}$$

then eliminating the water content from Eq 1 gives

$$I_{ps} = \frac{1}{3}\frac{\rho_s}{\rho_w}\frac{c}{1+e_o} \tag{4}$$

The factor of one third in Eq 3 is a nominal correction factor, included to give the index in terms of the vertical strain component, assuming that the vertical strain in an unconstrained sample is one third of the volumetric strain.

Cameron (1989) reviewed a number of different tests for the measurement of swelling indices, I_{ps}, for the direct estimation of shrinkage indices for implementation in equations of the type of Eq 3. One of these is the core shrinkage test as described by Mitchell and Avalle (1984). In this test, a small, undisturbed core of clay soil is trimmed to a diameter of 38–50 mm, and a length of twice its diameter. It is then air dried for two days before being oven dried. Regular measurement of the sample mass and length are made throughout. Also, the suction of the sample at its initial moisture state is measured and a soil water characteristic curve is determined. Using the data corresponding to the initial linear part of the drying curve, a swelling index referred to as the core shrinkage index, I_{cs}, is determined as follows,

$$I_{cs} = \frac{\varepsilon}{\Delta\log(\psi)} = \frac{\varepsilon}{\Delta\omega}\cdot\frac{\Delta\omega}{\Delta\log(\psi)} = \frac{\varepsilon\cdot c}{\Delta\omega} \tag{5}$$

Another simple test is the loaded shrinkage test, (Cameron and Walsh 1984a, b). In this test, a small, undisturbed sample is placed in a perforated shrinkage cell under a nominal load of 25 kPa (to simulate the pressure of a typical lightly loaded foundation). The initial moisture content and suction are determined. The sample is then subjected to controlled shrinkage by being allowed to reach mass equilibrium in the airspace above a supersaturated copper sulphate solution in a vacuum desiccator, which produces an equivalent soil suction of 4.5 pF units or 3.1 MPa (pF is defined as log negative [hydraulic head in centimeters], or by $1.01 + \log$[suction in kPa]). After corrections are applied to the measured data for the initial load settlement of the sample, a reactivity index referred to as the loaded shrinkage index, I_{ls}, is determined using an equation similar to Eq 5. The controlled shrinkage test can take more than 8 weeks to achieve equilibrium of the small samples used.

The subject of this paper is the shrink swell test, which yields a reactivity or shrinkage index called the shrink swell index, denoted I_{ss}. The test is described, and its application to the engineering of expansive soils is discussed in the following sections.

History of the Shrink Swell Test

A paper about the shrink swell test would not be complete without a brief history of how the shrink swell test has evolved.

The first quantitative assessments of soil expansiveness in Australia are thought to have been carried out in the late 1960s. Despite the growing awareness of expansive soils that arose in this era, the use of such assessments to guide engineering practice were extraordinary in the 1970s, and where performed, they were based on simple core shrinkage tests, and guided by work such as that of Aitchison and Woodburn (1969) and Aitchison and Martin (1973). In the early 1980s, the need for improved engineering of expansive soil foundations was becoming apparent, and the assessment of soil expansiveness through specific testing was being undertaken on an infrequent basis by the consulting industry, mostly using core shrinkage methods such as that described in Mitchell and Avalle (1984).

The use of a core shrinkage test was considered inadequate by many, due to its sensitivity to the initial moisture content of the sample. Samples that were unusually wet at the time of collection allowed measurement of shrinkage over most of their "working range" of moisture content, and so the values obtained were considered adequately representative. Samples that were unusually dry at the time of collection shrank little, if at all, and so the results were often unreliable. Cameron (1989) also found in a later study that the reliability of the core shrinkage test was strongly dependent on the moisture content-suction relationship for the soil, which could not be either simply determined or reliably derived by empirical relationships over a wide range of soil types.

Colin Thorne, a senior consulting engineer with Coffey Partners, is understood to have been the first to propose a shrink swell test. He proposed that a swell test be carried out in parallel with the shrink test, so that volumetric strains could be measured regardless of the initial moisture content. His idea was simply to add the axial strains measured from an unrestrained shrinkage specimen (ε_{sh}) and axial swell strains measured from a simple odometer specimen (ε_{sw}), both tested from the initial water content. Two reports by Colin Thorne, issued through Coffey Partners to the New South Wales Builders Licensing Board (Coffey Partners 1984, 1985), are also considered to have helped shift the focus of foundation design in eastern Australia for lightly loaded structures away from settlement, and onto soil expansiveness. By this time, the need for better engineering in expansive soil foundations had become so obvious nationally that a committee was convened to prepare a draft Australian Standard to provide necessary guidance. Colin Thorne, and the second and third authors of this paper, were all members of that committee, while Paul Walsh was the founding chairperson of the committee.

During the deliberations of this committee, the shrink swell test, as it currently exists, was formulated. It was considered that the axial swell strain from a 1D consolidation test, and the axial shrinkage strain component from an unrestrained core shrinkage test, were fundamentally different quantities, and that they could not simply be added together to produce a consistent result. It was decided that the swell component should be corrected to account for lateral restraint, by assuming that the suppressed lateral expansion would be redirected (to a greater or lesser extent) into the vertical direction. It was decided that a realistic swelling strain could be estimated (corresponding to a sample allowed to swell freely in all directions) by dividing the swelling strain measured in the restrained test by a factor of 2.0. The merits of this were explored in subsequent research and are discussed in a following section.

In order to derive a swelling index from this axial strain measurement, the suction change, corresponding to the variation in the measured axial strain, needs to be estimated. The use of parallel shrinkage and swelling tests meant that in all cases, the tests were conducted between the ranges of oven dry and effective saturation. It was considered, in the experience of the committee, that the significant, linearly varying portion of the strain-suction change relationship had a consistent suction range of 1.8 pF. It was thus decided that the test could be made attractive to routine geotechnical practice by adopting this value as being appropriate for all soils, in all tests, and hence, circumvent the need to determine suctions through direct measurement. The shrink swell index is thus defined as

$$I_{ss} = \frac{\varepsilon_{sh} + \frac{\varepsilon_{sw}}{2}}{1.8} \tag{6}$$

The units of the shrink swell or reactivity index are %strain/pF of suction change.

In order to derive the instability index from the reactivity index, the index must be adjusted to account for the effects of surcharge and lateral confinement that act on the soil in its *in situ* condition. This is done by application of a factor, α, usually applied at the time at which ground movements are being estimated.

$$I_{pt} = \alpha \cdot I_{ss} \tag{7}$$

The committee considered that in a cracked clay soil there is no lateral confinement, and so, the I_{ss} parameter should give a good estimate of the vertical soil strain component in an effectively unrestrained soil. Therefore, a value of α of unity was adopted in this zone. It was assumed that the effects of surcharge could be ignored at the shallow depths where shrinkage cracking occurs. In the soil below the cracked zone, lateral confinement exists, and it was considered that expansion in the vertical direction should be increased, but that below this depth, an increase in vertical swelling would be tempered by the surcharge restraint due to overlying soils.

In order to be consistent with the definition of I_{ss}, the effects of lateral restraint could be accounted for by using a value of $\alpha = 2$. This effectively reverses the correction applied to the swell component in determining the instability index, and applies a reversed correction to the shrink component. The effects of surcharge were more difficult to take account of. It was decided that the swell test could be carried out under a nominal applied load of 25 kPa, and that the effects of surcharge pressure could be included when the measured I_{ss} values were used in a ground movement prediction. The methodology for this was developed by making recourse to anecdotal experience. Observations from the black soil plains of Moree in NSW, Australia, suggested that no movement occurred below 10 m, whilst at Vicksburg, Mississippi, it was similarly observed that "at 30 feet, clays don't swell." It was decided that the applied correction should result in no movement at a depth of 10 m, and that the effects of surcharge at any depth, z, should be linearly interpolated from total swell suppression at 10 m ($\alpha = 2.0$) depth, to no suppression at the surface ($\alpha = 0.0$). The resultant adjustment factor was thus defined as

$$\alpha = \begin{cases} 1.0 & \textit{cracked zone} \\ 2.0 - \frac{z}{5} & \textit{uncracked zone} \end{cases} \tag{8}$$

Other authors such as McKeen (1992) have proposed theoretical α values. McKeen effectively incorporated an α factor as the product of two factors, f and s. Factor f was the lateral restraint factor, which is primarily related to the coefficient of earth pressure at rest, K_o, as follows:

$$f = \frac{1 + 2K_o}{3} \tag{9}$$

Factor f was reported to lie within the range 0.50–0.83.

McKeen's second factor, s, was a load effect factor, which was assumed to depend upon the percentage of applied pressure to the swelling pressure, $\%SP$. The relationship for $\%SP$ less than or equal to 50 % was:

$$s = 1.0 - 0.01(\%SP) \tag{10}$$

McKeen did not consider the influence of shrinkage cracking upon the ability of the soil to change volume. It should be noted also that factors f and s were applied to a reactivity index from CLOD tests, which was based on volume strain, and not vertical strain.

The first edition of "Australian Standard: Residential slabs and footings," was released in 1986 (AS2870-1986), and the testing procedure of the shrink swell test was formalized in 1992 in AS1289-7.1.1-1992, "Methods for Testing Soils for Engineering Purposes: Method 7.1.1: Determination of the Shrinkage Index of a Soil; Shrink Swell Index." The method is outlined in the following section.

The shrink swell test is now a routine part of Australian geotechnical practice. Its use has been growing steadily since the first tests in the early 1980s. In a study of shrink swell testing practice by the geotechnical consulting industry in the city of Newcastle, Australia, (with a regional population of around 500,000), Fityus and Welbourne (1996) gathered data on the number of tests performed over a 15-year period. This data are reproduced in Fig. 1.

Figure 1 indicates a steady increase in the number of shrink swell tests carried out since 1980, and supports the observation that it has become a routine part of local geotechnical practice.

The Shrink Swell Test

The procedure of the shrink swell test is detailed in AS1289 7.1.1-1992: Soil Reactivity Tests—Determination of the Shrinkage Index of a Soil—Shrink Swell Index, and the reader is directed to this document for precise details. Only the basic steps in the procedure will be summarized here.

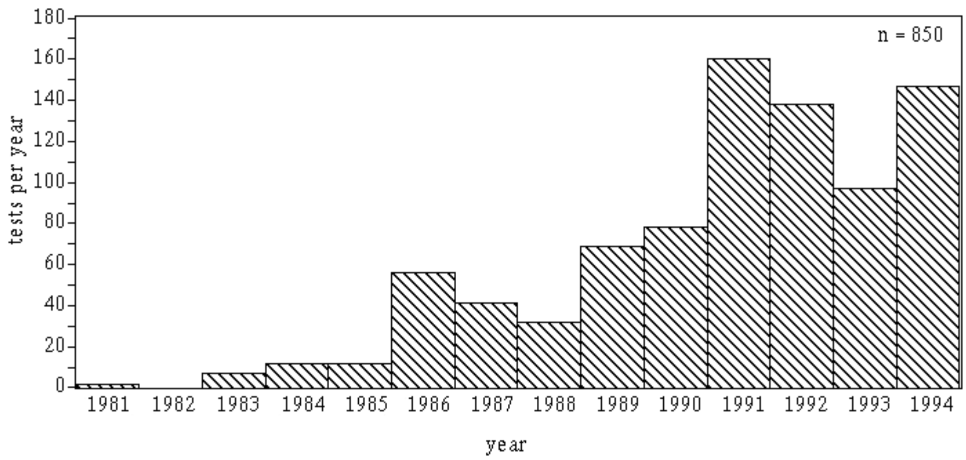

FIG. 1—*Frequency of shrink swell testing in Newcastle, Australia, 1981–1994*.

As described previously, the shrink swell test is composed of companion core shrinkage and swelling tests, carried out on undisturbed soil samples from their initial field moisture contents. The vertically oriented sample is usually obtained from the ground, using a 50-mm-diameter thin-walled tube. The sample is extruded from the tube as a soil core and a suitable portion of the sample is selected for the preparation of a shrinkage core (70–100 mm long) and a swell core (20–25 mm long). Test samples must be from adjacent portions of the core to ensure that water content and both compositional and structural differences are minimized. The shrinkage and swell tests are then conducted simultaneously, as follows.

Shrinkage Test—This component of the test is identical in procedure to the core shrinkage test, although fewer measurements are required as the shrink-swell index is based on the oven-dried state. A shrinkage core, 45–50 mm in diameter and a length of 1.5–2 diameters, is trimmed from the soil sample. Where possible, it is selected and trimmed to be free of major structural defects and loose material. Initial dimensions and mass are recorded. Small pins are added to each end as reference points to facilitate consistent measurements of sample length as drying proceeds. The shrinkage core is firstly air-dried. Regular measurements of length and mass are taken until shrinkage ceases. The core is then oven-dried to a constant mass at 105–110°C, and final mass and dimensions are recorded. The data recorded enables initial and final water contents, axial strain to be calculated, and a graph of axial strain against water content to be plotted. Throughout the drying process, the core is kept in a shallow tray, so that any crumbs that become loose during the test are not lost, as this would affect moisture content calculations.

Swelling Test—This involves a simplified oedometer test in which the sample (of measured mass) is installed in a rigid ring, (of measured volume; usually around 20 mm high and 40–45 mm in diameter) and placed between porous stones in a consolidation apparatus. A gage to monitor the sample height is then zeroed under a nominal seating pressure of 5 kPa. A load of 25 kPa or the estimated in situ overburden pressure (whichever is greater) is then applied for 30 min to record any initial settlement or seating adjustment. This displacement is used to correct the initial sample height for determination of swelling strain. After re-zeroing the displacement gage, the sample is inundated with distilled water and allowed to swell until the swelling increment, in a period of not less than 3 h duration, is not more than 5 % of the total recorded swell. The initial water content is determined from the sample trimmings, and the final water content is measured from the extracted sample at the end of the test. In the sample preparation process, particular care is taken to ensure that the sample neatly fills the sample ring, as voids and recompacted or remolded portions will accommodate internal adjustments in the volume of the sample and hence, affect the realized vertical swell.

Shrinkage strains (ε_{sh}) and swell strains (ε_{sw}), measured in the respective tests, are then combined to give a swelling (shrink-swell) index, I_{ss}, according to Eq 6.

Application of the Shrink Swell Test

The shrink swell index gives a quantitative measure of the vertical strain that will occur in a clay soil per unit change in suction. Thus, if the vertical variation in the I_{ss} and the soil suction changes (in pF units; ΔpF) are both known (or can be estimated) for a soil profile of n layers, then the vertical strains can be calculated in each layer, and integrated to give a nett ground surface movement, according to the equation (Aitchison 1973)

$$y_s = \sum_{i=1}^{n} I_{pt,i} \cdot \Delta pF_i \cdot \Delta z_i = \sum_{i=1}^{n} \alpha \cdot I_{ss,i} \cdot \Delta pF_i \cdot \Delta z_i \qquad (11)$$

In Eq 11, y_s is the nett vertical ground surface movement, provided the suction changes represent the design suction changes for the climatic region or local conditions of the site. In the context of Australian practice, y_s is defined as the characteristic ground surface movement at a site, and it is defined in statistical terms as being that ground surface movement that is likely to occur in an undeveloped area, over a nominated time period, largely as a result of climatic influences. The required statistical significance is achieved by selecting an appropriate distribution of likely suction changes with respect to depth (see Walsh and Cameron 1997 for further details). The value of y_s is then used, in accordance with the recommendations of AS2870, to estimate mound heights and edge distances that can be used in the structural design of shallow foundations.

It is now routine in Australian practice for a "site classification" to be carried out to guide foundation design, for every new residential structure. This is usually carried out as part of a larger geotechnical investigation that also assesses issues such as slope stability, and that makes specific recommendations on good site development practice. For a single, isolated site, this usually involves logging, and possibly testing, the soils from at least one borehole. AS 2870 suggests a minimum of 3 boreholes per site for regions where the depth of the design suction change extends beyond 3 m and the soil profile in the general area is known to be variable. For a larger subdivision on fairly uniform soil profiles and relatively shallow depths of movements, the number of soil profiles logged, and shrink swell tests performed, may be as few as one for every three to five building sites.

Discussion

Whilst the shrink swell test is a rationally based test, it can by no means be considered rigorous in its attempts to model accurately expansive soil behavior. The shrink swell test only takes account of factors that are of significance to its specific purpose, and although it does so in an approximate way, it achieves sufficient accuracy to yield a useful result.

The simplicity of the shrink swell test is achieved, in part, by a number of important assumptions. One of these relates to the magnitude of the suction change that corresponds to the soil volume change, assumed to be equal to 1.8 pF units for all soils. This assumption effectively bypasses the need to measure suctions. The value of 1.8 pF was based on the collective experience of the AS2870 code committee, and is supported by the observation that most of the volume change occurs between the "wilting point" for trees and a water content close to saturation. Wray (1998) reported that the suction at the wilting point is usually around 1500 kPa (4.2 pF) in a clay soil. Cameron (2001) reported wilting points ranging between 930 and 2600 kPa (4.0–4.4 pF). Data in Fredlund and Rahardjo (1993) suggest that the total suction at water contents near saturation is in the order of 2.2–2.5 pF units. This suggests that the value of 1.8 pF units is not unreasonable, in the context of the usual range of suction changes in the field. However, as stated previously, the adopted range has more to do with the observation that expansive soils change volume relatively consistently and linearly with suction change over this range.

It should also be noted here, that the need to measure suctions can be reduced also in the approach of McKeen (1992), by adopting a "benchmark intercept" suction value at zero water content. Application of McKeen's approach still requires the measurement of a suction at the natural water content to establish the slope or index value used in volume change prediction. Likos et al. (2003) investigated the Colorado practice of adopting a value of 5.25 pF for the benchmark intercept. This value is significantly greater than the wilting point values given above, but strangely is less than the benchmark suction value of 5.5 pF attributed to McKeen (1992). Furthermore, McKeen (1992) and Mitchell and Avalle (1984) suggested "benchmark intercept" suction values at zero water content of 6.5–7 pF and 6.7 pF, respectively, while Cameron (1989) found that a single benchmark value for a wide range of soils was not apparent, but instead seemed to vary with soil plasticity. Accordingly, Likos et al. (2003) concluded that the assumption of 5.25 pF was inadequate for prediction of soil movements.

The other significant assumption of the shrink swell test relates to the use of the factor of 2.0 to estimate the vertical strain in an unconfined swell specimen, from the measured vertical strain in a rigidly confined swell specimen. Since its adoption, the validity of this assumption has been considered on two occasions. A simplified theoretical discussion of appropriate correction factors, based on elastic theory, is presented in Cameron (1989). Using the strain measurements from 66 unrestrained core shrinkage tests, it was suggested that values between 1.7 and 2.15 are likely to be appropriate, although these were sensitive to the adopted values of the Poisson's ratio.

Fityus (1996) reconsidered the theoretical estimation of the factor, also on the basis of elastic theory, and proposed that values between 2.2 and 2.9 are more appropriate. However, it was concluded that the lack of consistency between these estimates and those of Cameron (1989) was likely to reflect the gross simplifications implicit in assessing a time dependent, elastoplastic deformation problem as a purely elastic phenomenon. It was concluded that simplified isotropic-elastic models, as used by Cameron (1989) and Fityus (1996), are not sufficiently accurate to describe the behavior of the confined swell test.

An assessment of the appropriateness of the factor of 2.0 has also been made experimentally. Fityus (1996) conducted two series of shrink swell tests on remolded, homogenized clay samples; one reconstituted from an expansive natural clay, and one "manufactured" from a mixture of industrially purified bentonite, kaolinite, and fine sand. In each case, tests were carried out on samples of identical composition, starting from six different initial moisture contents. Moisture contents ranged from the shrinkage limit to a state of effective or quasi-saturation.

To avoid possible effects due to differences in structure between samples prepared at different moisture contents, all test samples were initially compacted at similar moisture contents, using a standardized compactive effort. The water contents of the compacted samples were then conditioned to a range of target moisture contents prior to testing. After calculation of the total final weight corresponding to the targeted initial moisture content, the samples (in tubes) were either soaked or oven dried. At the desired weight, the tubes were plugged and wax-sealed at the ends. The sealed tubes were stored, in a horizontal position, for a minimum of four weeks to allow the moisture throughout the sample to equilibrate. Shrink swell testing was then performed.

From the results in each case, trend lines for both shrinkage strains and swell strains were plotted against initial water content. It was found, in the case of the reconstituted natural soil, that if the measured swell strains were divided by a factor of 2.2, the sum of the shrink and swell trendlines was a horizontal line, indicating insensitivity to the initial water content. Similarly, in the case of the "manufactured" soil, if the swell strains were divided by a factor of 1.9, the sum of the shrink and swell trendlines was a horizontal line.

It is likely that the correct factor is somewhat unique for particular soils. However, the results indicate that the value of 2.0 is reasonably appropriate for the soils used, and it is considered likely that it is generally appropriate for most clay soils.

It should also be noted that the appropriateness of the value of 2 is conditional upon all of the soil swell being constrained laterally. Many oedometers allow the soil sample to swell beyond the sample ring where lateral confinement is not provided. Based on observations from numerous shrink swell tests on initially, relatively dry soils, it is apparent that when the swell displacement becomes large (exceeding 1 or 2 mm) the results of the shrink swell test are affected by this lack of confinement. Apart from the associated lateral strains, which reduce the magnitude of 1-D swell, some soils will also disperse. The effect of unrestrained swelling near the top of the sample was quantified in the experimental study of Fityus (1996), who reported that the recorded swell strain may be underestimated by as much as 25 %. It was suggested that extension rings should be fitted to the oedometer ring to constrain all swell to occur in an axial direction. The suggestion was trialed and proved successful.

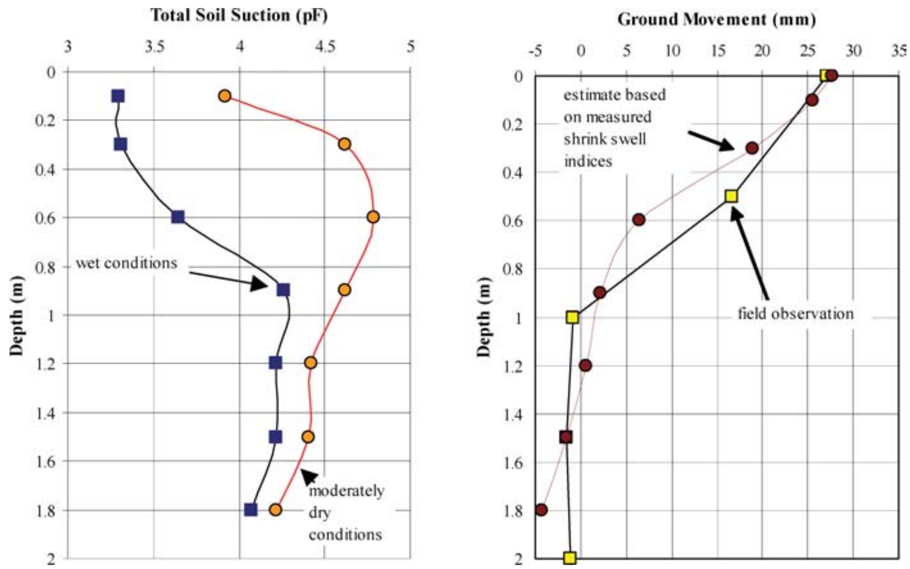

FIG. 2—*Comparison of measured ground movements with those predicted using measured shrink swell indices. The corresponding suction change is also shown (Cameron 1989).*

The ability of the shrink swell based approach to predict accurately ground movements was considered in a major experimental study by Cameron (1989). This study was based on data from twelve field sites, which included measurements of seasonal ground movement and the experimental determination of a variety of different soil expansiveness parameters. From 14 observations of ground movements (12 were reported in Cameron (1989)), the mean movement estimate was 81 % of the average measured soil surface movement, and the standard deviation of the estimates was 25 %. It should be noted that the 14 observations were based on seven sites, with a drying and a wetting cycle taken from each site. An example of the predicted and measured soil movement profiles with depth is provided in Fig. 2. The site is Gilles Plains in Adelaide, South Australia. These data were not available at the time of publication of the original paper. Soil suction profiles are given as well as the movement profiles. The shrink-swell index tended to increase with depth from approximately 3 % strain/pF near the surface to 6 % strain/pF at two metres depth.

The study concluded that soil movement predictions based on an index determined from the shrink swell test were generally more successful than predictions made on indices such as those determined from core shrinkage and loaded shrinkage tests. It was found that assumptions of suction change and lateral restraint effects, which were employed in the procedure, were able to give acceptable prediction results, considering the variable nature of soil profiles and moisture distribution patterns. The other methods of evaluating reactivity suffered particularly because of the dependence on initial moisture condition of the soil.

In another illustration, Fityus et al. (2004) presented the findings of a 10-year field study on an expansive soil site at Maryland in central eastern Australia. From a series of 30 open ground monitoring points, a range of ground movements of 47–75 mm was recorded, with an average ground movement of 58 mm. Using measured shrink-swell indices and values of seasonal suction change parameters derived from the study, the ground movement model described above predicted a value of 57 mm.

The shrink-swell indices in Cameron's study have been plotted against linear shrinkage in Fig. 3. A trend line is apparent, but with a low regression coefficient. The lines either side of the trend line indicate ± 1 %/pF departure from the trend line, and serve to reinforce the observation that reactivity cannot be predicted with any accuracy by this correlation with linear shrinkage for the range of soils tested. Furthermore, the regression coefficients were even lower when either the plasticity index or liquid limit was substituted for the dependent parameter. Linear shrinkage, plastic limit, and liquid limit tests are all performed on remolded and screened soils, whereas the shrink-swell tests were usually conducted on relatively undisturbed natural soil specimens, sampled with thin-walled sampling tubes.

The shrink swell test offers a test method that is much simpler than those of most other tests that have been used to characterize expansive soils. The shrink swell test can be reliably performed by laboratory technicians with limited experience, with relatively rudimentary laboratory equipment and at relatively low cost. A typical shrink swell test takes around 4 days to complete, but involves only 1–2 h of actual "hands on" experimental work. When multiple tests are performed simultaneously, an experienced technician can carry out the tasks involved in 8–10 tests in a time that is roughly equivalent to a single day. Thus, if the costs of sampling are excluded, the shrink swell test can be carried out for the approximate cost of one hour of a technician's wages.

Its relative simplicity, low equipment cost, and relatively quick turn around time have facilitated the widespread adoption of the shrink swell test into routine Australian practice. It is used in all geographical areas of Australia, and in geological environments that include recent to ancient alluvial soils, and residual soils derived from both sedimentary and crystalline rocks. It is used less frequently as a means of confirming expansive potential in areas where geological trends are relatively well established and regionally persistent. However, it is most useful, and indeed relied upon, in areas where geological conditions exhibit frequent and rapid variation, and where soil mapping, and extrapolation of geological trends, cannot be reliably employed (Fityus and Delaney 1995).

To the authors' knowledge, a general assessment of the effectiveness of the shrink swell test, and its associated application in routine geotechnical practice, has not been undertaken formally in any quantitative way. The authors believe, however, on the basis of

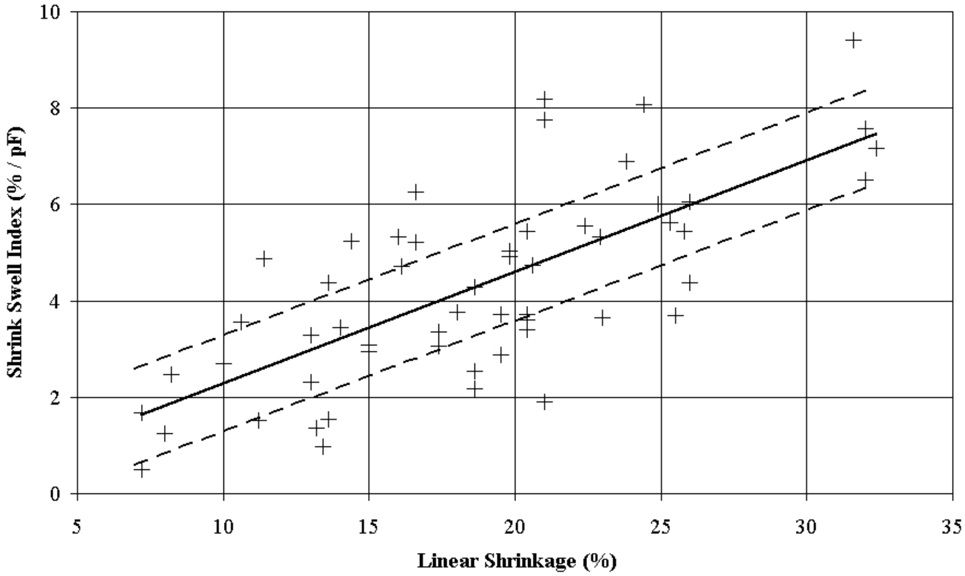

FIG. 3—*Reactivity data from a range of soil types in Australia (Cameron 1989).*

their collective experience in expansive soil engineering practice in many areas of Australia over an extended period, that the shrink swell test has served the geotechnical industry well, despite its inherent simplifying approximations. This experience is in contrast to that of the study of Likos et al. (2003), which found that by adopting benchmark values of suction as a simplification of the method of McKeen (1992), significant potential errors arose in the majority of cases.

In the opinion of the authors, the design of foundations, guided by the results of shrink swell testing, is successful in the vast majority of cases, considered from the context of both achieving adequacy in design, and avoiding significant over design. The authors would suggest that where the guidance of AS2870 (including its inherent use of the shrink swell test) is accepted and applied (in the manner intended by AS2870), the rate of design "failures" is certainly less than 5 %, and probably less than 2 %, provided the active soil profile is properly appreciated. Further, where failures occur in new structures, these usually result from anomalous conditions (which may or may not be reasonably expected to be identified by routine geotechnical investigation), from a failure to interpret the guidance of AS2870 correctly, or more usually, from a disregard/ignorance of expansive soil phenomena by structural designers or builders.

Conclusions

The shrink swell test is a simple and economical laboratory test that is performed on undisturbed clay soil samples to yield a reactivity index that enables free surface ground movements to be predicted. It has been employed in routine geotechnical practice in Australia for the past 20 years, and during that period, it is considered to have served the Australian geotechnical industry well. The successful, widespread adoption into Australian industry practice is due to several factors. Firstly, it has a rational and intuitive basis, making it attractive to practicing geotechnical and structural engineers. In particular, the test evaluates the soil over the full range of volume change, not just the swell or the shrinkage phase, thereby making the method independent of the initial moisture state of the soil. Secondly, it involves a simple and economical laboratory test that can be performed on a routine basis, without adding excessively to the cost of light residential construction.

The inherent simplicity of the shrink swell test derives from several important simplifying assumptions that effectively avoid the physical measurement of soil suction. On the basis of available research, and a qualitative assessment of the successful employment of the shrink swell test in routine practice, the error introduced by these assumptions is considered to be acceptably small.

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