



Prof Eben Rust
Dept of Civil Engineering
University of Pretoria
eben.rust@up.ac.za



Prof Gerhard Heymann
Dept of Civil Engineering
University of Pretoria
gerhard.heyman@up.ac.za



Prof Gary Jones
Dept of Civil Engineering
University of Pretoria
Tel: 011 441 1128

Collapse potential in unsaturated soil was first identified and quantified by researchers in South Africa. A landmark paper was published by Ken Schwartz in 1985 presenting the state of the art at that time. Since then, international researchers have expanded on the understanding of what collapsible soils might entail. These include saturated silts and sensitive clays. This article highlights some of the new developments and presents a theoretical yield model in an attempt to improve the understanding of the mechanism involved

INTRODUCTION

The subject of collapsible soils has not received much attention in southern Africa recently and the authors are aware of only two or three publications since the exposition by Ken Schwartz in 1985. This is surprising since development has been intense in the areas of collapsible soils in South Africa, namely the granitic soils of the Highveld and the Berea Red Sands along the east coast. The other area of collapsible soils, the Kalahari Sands, has been subject to less development, hence it can be expected that less would have been written about it. This article first summarises collapsible soils from an international perspective and then focuses on the testing and modelling of collapsible soils from southern Africa.

Collapsible soils an overview

COLLAPSIBLE SOILS

If soil collapse is defined as an abrupt decrease in volume for whatever reason, then the definition encompasses a vast range of soils.

For example, the sensitive clays of Scandinavia and eastern Canada are by this definition collapsible, despite being plastic and fully saturated. On the other hand, unsaturated soil such as the loess formations of China, Russia and eastern Europe cover enormous areas of those countries and constitute perhaps the classical image of collapsible soils, as do the Kalahari Sands. Residual soils such as the Highveld granites and the brick earths of Kent in the UK form another well-recognised group, as, to a lesser extent, do the Berea Red Sands of the southern African east coast.

The definition may be further extended, arguably, to include the submarine sand slopes of coastal Holland and the Beaufort Sea, which have suffered many failures ascribed to liquefaction; it may be argued that liquefaction is but one manifestation of collapse.

Materials that also fit the definition are compacted soils (Booth 1977).

Rogers (1995) suggested the following definition of collapsible soils: "A collapsible soil is one in which the constituent parts have an open packing and which forms a metastable state that can collapse to form

a closer packed, more stable structure of significantly reduced volume. In most collapsible soils the structural units will be primary mineral particles rather than clay minerals. The collapse process that occurs in these soils gives them geotechnical significance." However, Rogers points out that rather than have a definition *per se*, it is more useful simply to list the typical characteristics of a collapsible soil:

- an open structure
- a high void ratio
- a low dry density
- a high porosity
- geologically young or recently altered deposit
- high sensitivity
- low interparticle bond strength

The most common recognition test, other than visual assessment, is the single oedometer collapse potential test which results in the categories shown in Table 1 (Jennings & Knight 1975). The originators of the test, and Schwartz (1985), emphasised that the test was intended only as an indicator, not as the basis for a method of predicting the amount of collapse settlement.

A number of workers have attempted to predict collapse as a function of material characteristics such as density, porosity, clay content moisture content, soluble salts, etc. In the southern African context, Brink (1985) reproduced two sets of relationships between collapse potential index and dry

density for aeolian sands and soils of mixed origin, attributed to Schwartz and Pavlakis respectively. These relationships are represented by the following equations:

Aeolian sand:

$$CP = \frac{1672 - \rho_d}{22} \quad (1)$$

(coefficient of correlation = 0,73)
(Schwartz)

Mixed-origin soils:

$$CP = \frac{1590 - \rho_d}{18,9} \quad (2)$$

(coefficient of correlation = 0,77)
(Pavlakis)

- 1 Swell (positive strain) and collapse (negative strain) prediction (from El-Sohby et al 1995)
- 2 Typical result from triaxial collapse potential tests
- 3 Effect of pore fluid suction on effective stress

Cp (%)	Severity of problem
0 – 1	No problem
1 – 5	Moderate trouble
5 – 10	Trouble
14 – 20	Trouble
> 20	Very severe trouble

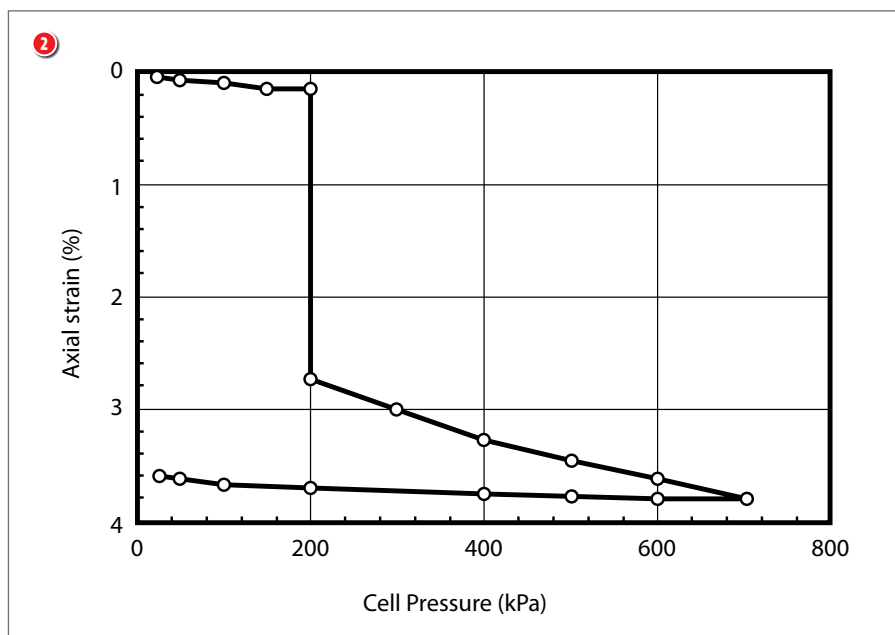
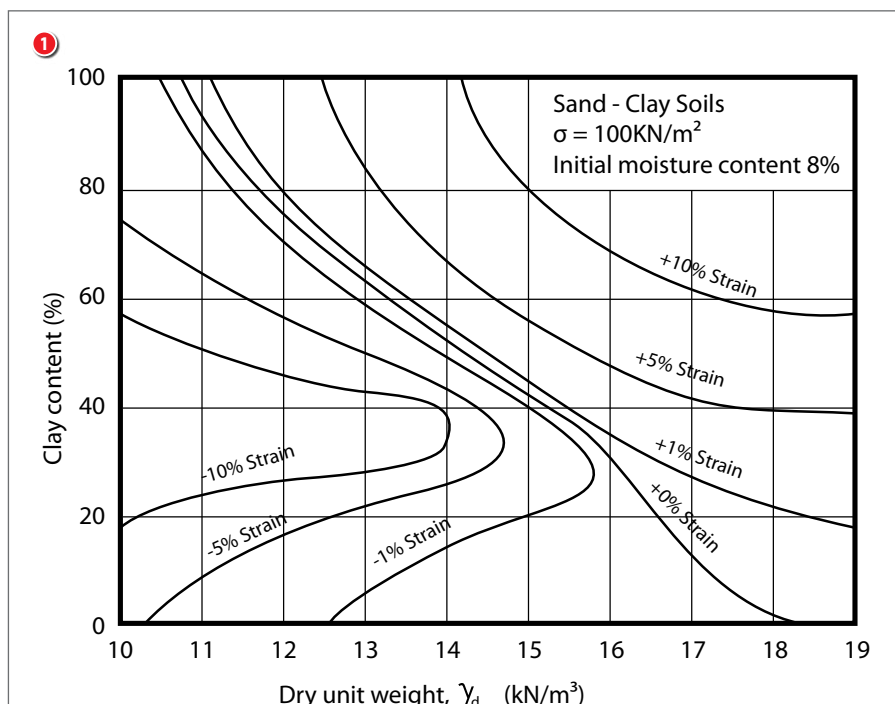
The equations imply that aeolian sands with dry densities greater than 1 672 kg/m³ and mixed-origin soils with dry densities greater than 1 590 kg/m³ are generally not collapsible. The coefficients of correlation 0,73 and 0,77 are not high, but could possibly be improved with more data. It would, however, be simplistic to assume that such a single-function model, relying only on density, would provide the optimum correlation of multi-functional collapse potential with basic soil parameters.

Figure 1 is taken from El-Sohby et al (1995). It represents an amalgam of swell and collapse predictions based on numerous predictive methods representing worldwide best practice. The authors give two similar diagrams: one for silt-clay and one for sand-clay, and it is the latter that is reproduced here. It clearly shows that soils with a dry density of 1 600 kg/m³ would not be expected to have collapse potentials of greater than 1%.

SAMPLING AND TESTING OF COLLAPSIBLE SOILS

High-quality sampling is required for conducting collapse-potential tests in the laboratory. Hight et al (1992) showed that block sampling produces samples of the highest quality compared with other sampling techniques. However, Heymann & Clayton (1999) highlighted disturbances that may occur as a result of moisture change during storage. Even small changes in moisture content can change the matric suction, and hence the effective stress, by much more than the loading due to an engineering structure. They recommended that samples be covered with numerous layers of aluminium foil and polyurethane film (cling film) to protect them against moisture change.

Rust et al (2005) pointed out some shortcomings of the oedometer test for measuring collapse behaviour. In particular, these include the bedding errors that occur due to surface irregularities at the interface between the soil and the top and bottom porous discs. The inaccuracy due to bedding errors becomes more pronounced as the height over which the sample collapse is measured becomes smaller, such as in the case of a one-dimensional oedometer. These errors can be significant and in some cases the bedding error can exceed the collapse settlement of the whole sample. Samples containing sand-sized particles are particularly prone to bedding errors.



Heymann (2000) and Rust et al (2005) described a test for measuring the collapse behaviour of soil in a triaxial apparatus. An integral part of the test is local strain instrumentation which is fitted directly onto the specimen away from the top and bottom porous discs. Conducting a collapse test in a triaxial apparatus allows a larger specimen to be used than in the oedometer but, more importantly, the use of local strain instrumentation entirely excludes errors due to bedding.

The specimen is placed in the triaxial at the in situ moisture content. The cell pressure is increased in increments and at the required cell pressure, the bottom drainage line is opened and the specimen is inundated with de-aired water. The response of the local strain instrumentation is monitored as collapse takes place. The loading is subsequently continued followed by an unloading cycle. Figure 2 shows a typical result of a collapse test conducted in the triaxial apparatus on soil from the Mozal Aluminium Smelter site in Mozambique (Rust et al 2005).

The result in Figure 2 indicates some important phenomena that occur during soil collapse. The first is the sudden axial strain of 2.6% during wetting as the particles are rearranged into a denser state. The second phenomenon is less obvious, but an important indicator for understanding the mechanism that governs the collapse behaviour of unsaturated soil; this relates to the reduction in volumetric stiffness of the material during wetting. Assuming isotropy, the stiffnesses before and after wetting can be calculated as 114 and 11 MPa respectively, indicating a ten-fold reduction in stiffness on wetting. Rust et al (2005) argued that this was due to a reduction in matric suction and therefore a reduction in effective stress. They further investigated the mechanism of suction and showed that a threshold moisture content exists where the matrix suction suddenly changes. For the Mozal soil they observed that the suction reduces by between 7 and 18 MPa when the moisture content goes above the threshold. This indicates that the change in effective stress in the soil due to the suction pressure changes can be many times the stress applied to the soil due to loading by engineering structures.

They also statistically compared two data sets of results from oedometer and triaxial collapse potential tests on material from the same site. They concluded that results from the triaxial collapse

potential test were much more reliable and suggested that this was due to the fact that bedding and confinement errors are present in the oedometer tests and are avoided in the triaxial test.

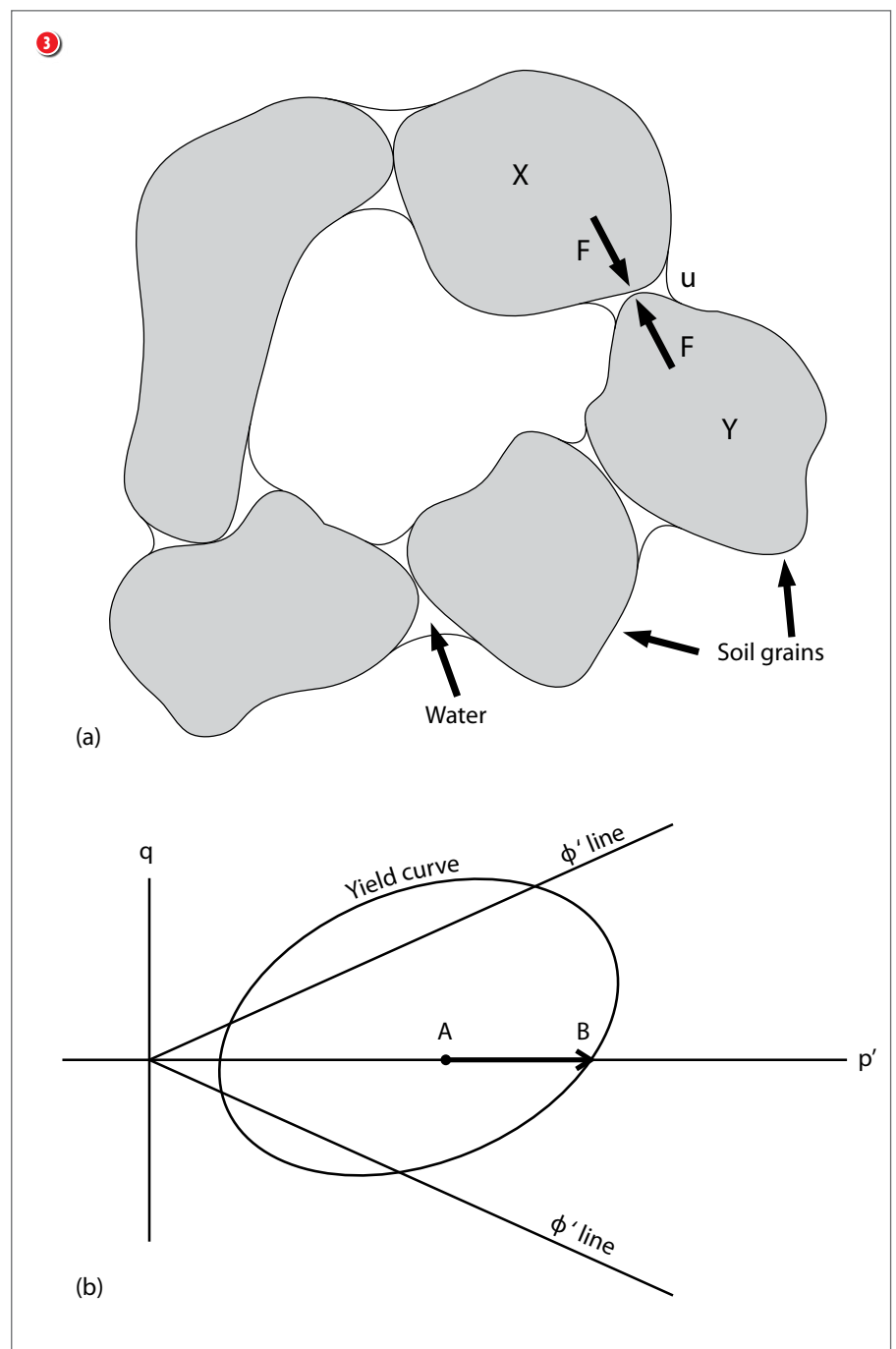
YIELD MODEL FOR COLLAPSING SOIL

Rust et al (2005) developed a conceptual yield model in terms of effective stress theory. This yield model for collapsible material is broadly based on the critical state model and specifically on the yield model for structured soils and weak rocks as presented by Leroueil & Vaughan (1990). At this stage the model is conceptual and no attempt was made to quantify any of the parameters. It is suggested that future research could be aimed at con-

firmed the validity, or otherwise, of the proposed model.

It is generally agreed that collapsible soils comprise a mixture of coarser soil grains held together by finer material which permit intermolecular, electrostatic, capillary and chemical bonds to develop, although not all of these bonds may be present and the relative strength of the bond type will depend on the soil and moisture content.

For a saturated material, applying a total stress to the soil or a suction of similar magnitude to the pore fluid has the identical effect on the effective stress of the material. For a soil with a low degree of saturation, a change in pore fluid suction and a change in total stress may not necessarily



have the same effect on the behaviour of the soil. The contribution of the capillary, or suction component of the intergranular forces is shown in Figure 3(a).

If the self-weight of this soil is ignored, the intergranular force (F) between grains X and Y is a function of the fluid suction (u) and the area over which it works (see Figure 3(a)). Changing the suction between the grains X and Y changes the intergranular force between the grains, but does not change the resultant internal force within grain X or Y , or for that matter anywhere else in the soil skeleton. This would happen between each pair of grains within the soil mass as it becomes more unsaturated, resulting in an increase in effective stress and

strength. Compare this with a change in the total stress on the sample that would, say, result in exactly the same increase in the intergranular force. This could only be done if the internal stress within grains X and Y changes.

One way to demonstrate the consequence of this difference is shown in Figure 3(b). A relatively dry sample is at an isotropic stress A and has a yield surface as shown. The sample is then subjected to an increase in isotropic stress to point B where it yields. At this point large volumetric strains occur (the sample collapses) and the collapsible structure is lost. Compare this with the same relatively dry sample at stress point A being dried out further. The suction pressures may increase to point B as before, but the soil skeleton will have no tendency

to yield and the suctions could be increased beyond point B with no yielding taking place. The reason for this is that during drying the yield surface has increased in size because of the increase in the strength of the bonds between the grains due to suction. This demonstrates the difference between the effects of these two components of effective stress. The suction forces act like bonding, with the bond strength being dependent on the moisture content or degree of saturation. This could be seen as suction-induced bonding.

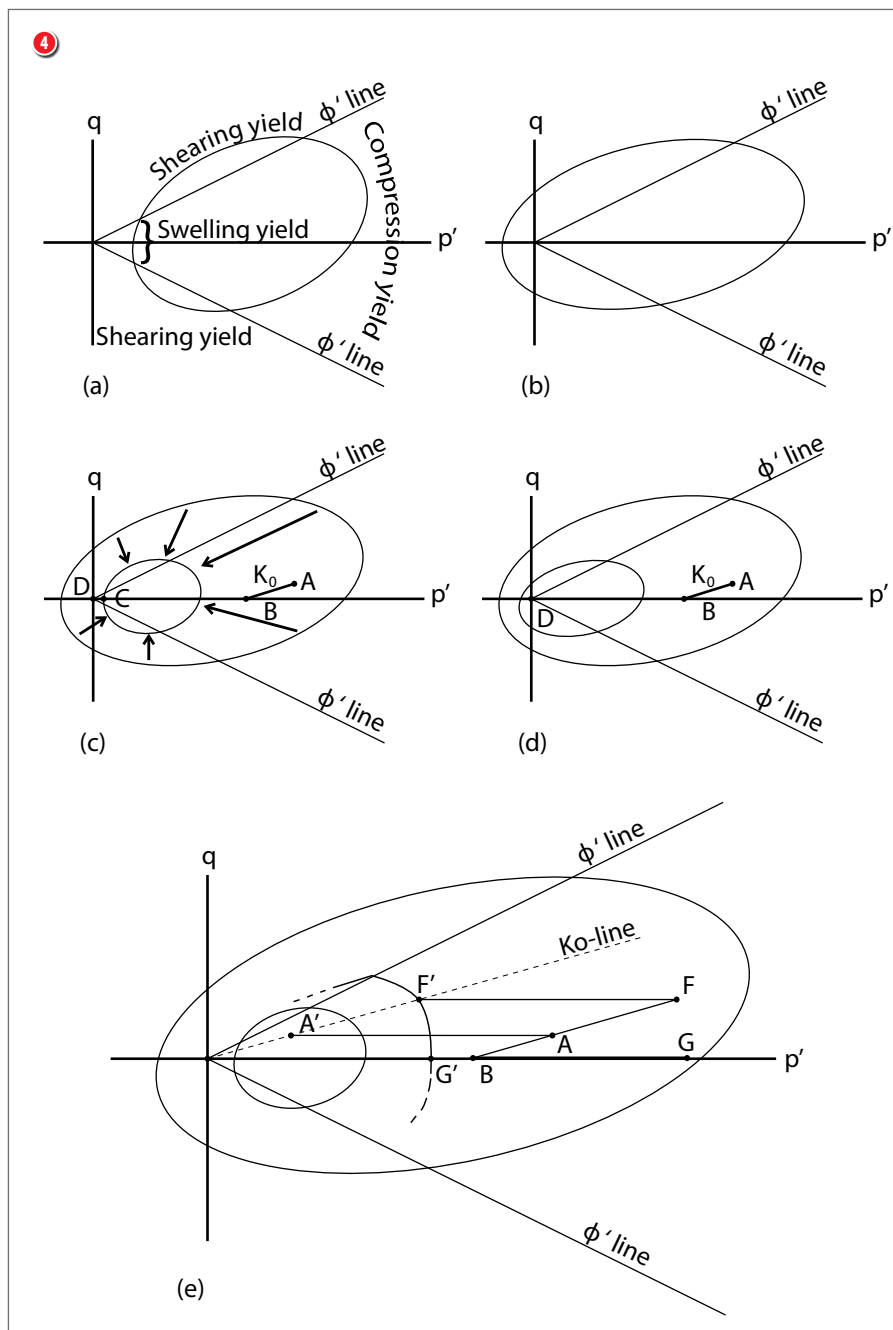
Collapsible soils can be seen as “structured” in their undisturbed state. This structure can be destroyed by excessive strain or remoulding, as shown in Figure 4(a). Three parts of the yield curve may be identified: shearing yield, compression yield and swelling yield. Shear yielding occurs in the vicinity of the Φ' -lines.

Compression yielding occurs between the two Φ' -lines due to increasing mean effective stress (p'). Swell yielding also occurs between the two Φ' -lines but is due to a reduction in mean effective stress (p'). It is possible that swelling yield may occur at negative p' (tension) if the bonding is sufficiently competent, as shown in Figure 4(b), rather than during positive p' for a weakly bonded material, as shown in Figure 4(a).

The in situ stress at depth is due to overburden pressure plus lateral pressure, as well as the isotropic stress component due to suction. In Figure 4(c) the suction is represented by $D-B$ and the overburden pressure by $B-A$. K_0 conditions are assumed for the overburden pressure.

Consider the following stress path. Cutting a sample from an unsaturated profile will remove the overburden pressure, leaving the sample at point B in Figure 4(c). Wetting the sample will reduce the suction, resulting in a decrease in the isotropic stress and moving the stress towards point C . This wetting will simultaneously reduce the size of the yield surface due to the weakening of the suction-induced bonds and the yield surface will change to the position shown in Figure 4(c) as the stress approaches point C . At point C the sample will yield in swell. This can be seen when a sample is placed in water and it completely disintegrates. It is also possible that the swell strains will not be sufficient to yield the chemical bonds under zero effective stress conditions and that yielding will take place only under tensile conditions along this stress path, as shown in Figure 4(d). In this case the sample will not disintegrate when placed in water.

4 Yield model



A general stress path for the in situ material is shown in Figure 4(e). The in situ sample will be at point A as discussed earlier. During a wet period the moisture content may rise above the critical moisture content, reducing the suction to zero and resulting in a stress represented by point A'. Since the land surface is in a stable state, no collapse of the profile will take place. The metastable soil structure is now supported by the chemical bonding and possibly by some remnant suction pressure and friction from the overburden stress. Point A' is still inside the reduced yield surface (Figure 4(e)).

The stress path of the one-dimensional collapse potential test is also shown in Figure 4(e). It starts at the in situ stress point A. After sampling the overburden is, of course, removed and the stress is at point B. The sample is placed in the oedometer and loaded to point F. Water is added and the suction reduced to a stress state represented by position F'. At the same time, the yield surface contracts, resulting in yielding of the sample in compression on the Ko line since F' now

falls outside the contracted yield surface. At this point collapse will occur, i.e. large deformations will take place and the soil will be de-structured. The yield surface at point F' now represents a de-structured classical state boundary surface.

The stress path of the triaxial collapse potential test is represented by the isotropic loading from stress point B to point G. The sample is then inundated, resulting in a stress at point G' and yielding in compression as with the previous case, but under isotropic stress conditions.

CONCLUSIONS

The brief literature review illustrates that, depending on the definition of collapse, a very wide range of soils can under some conditions be potentially collapsible. This range can, for example, extend from compacted road pavement materials to saturated soft clays. The generally accepted southern African potentially collapsible soils are the Natal coast Berea Red Sands, the Highveld granitic soils and the Kalahari Sands. These may be considered as being in the classical or typical range,

having low densities, high void ratios and being partially saturated sandy silts to silty sands with a little clay.

The triaxial collapse potential test illustrates two fundamental aspects that are essential in a proper understanding of collapse and that is not easily observed in oedometer testing. These are, firstly, that suction forces dominate the behaviour, and secondly, that the changes in suction pressures result in major changes in the stiffness of the material before and after wetting.

The yield model described in terms of effective stress and yield surfaces, which takes account of suction and other forces, demonstrates that the collapse process can indeed be explained by normal soil mechanics principles. Because this is so, it should be expected that collapse behaviour in the field can be predicted in the same way that consolidation testing and theory allows the prediction of consolidation settlement with considerable reliability.

NOTE

The list of references is available from the editor. □