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Collapse potential of partly saturated sandy soils from Mozal, Mozambique

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INTRODUCTION

There is an upsurge in development along the eastern coast of southern Africa that creates the need to re-examine the typical red sands which occur from northern Mozambique to the south coast of KwaZulu-Natal in South Africa.

A number of problems pertaining to these sands were highlighted by the work at Mozal, the aluminium smelter in Maputo, in the late 1990s (McKnight 1999). The primary geotechnical problem was perceived to be the high collapse potential of the red sands to depths of up to 15 m. The geotechnical investigation, which included conventional laboratory testing together with extensive pile testing, led to the decision that pile depths, particularly under the extremely settlement sensitive pot lines, should be significantly deeper than originally estimated. The foundation engineering decisions made at Mozal were based on the geotechnical information available at the time. However, there is a question as to whether the profession has a proper understanding and hence a rational practice for dealing with these red sands.

As part of the Mozal work, a separate investigation was conducted by the authors during which a number of unconventional laboratory tests were carried out. These include triaxial collapse tests, one-dimensional incremental collapse tests and pore fluid suction pressure measurements. The results do not agree well with those from the conventional tests conducted as part of the initial site investigation and this raises important questions regarding the validity of current practice. A constitutive collapse model is proposed which may shed light on the mechanisms involved.

COLLAPSE POTENTIAL

Collapse potential was first recognised, described and defined by Jennings and Knight (1957) and Knight (1961), and routine testing was developed to quantify soil collapse. Schwartz (1985) comprehensively reviewed this work in a state of the art paper.

Foundation recommendations for collapsible soil tend to be routine, that is, found below the layer with collapse potential, excavate and recompact the material, dynamically compact it and carry out various drainage precautions to prevent wetting. All these are possible and generally economical because the strata with collapse potential typically are not thick. These approaches have not changed significantly for many years, though our knowledge and understanding of the collapse phenomenon have improved in the recent past.

The workshop on the Genesis and Properties of Collapsible Soils (Derbyshire *et al* 1994) was convened for this purpose and the proceedings are essential reading. In broad terms the work deals with the genesis, identification, testing and engineering of collapsible soils with considerable emphasis placed on the definition of terminology. 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 may be present and the relative strength of the bond type will depend on the soil (Osipov & Sokolov 1994; Lefebvre 1994). Collapsible soils may generally be identified by simple tests such as particle size distribution, Atterberg limits, dry density, void ratio or degree of saturation and many regulatory authorities present criteria by which collapse may be recognised. These are generally empirical and relate to the area of that authority. Both one-dimensional consolidation and triaxial tests are used to assess the degree of collapse, but it is recognised that sample disturbance and the unrealistic wetting processes of the tests are severe limitations. Some authors (eg Houston *et al* 1994a) advocate field testing but again the unrealistic wetting presents a major difficulty.

In broad terms South African practice for potentially collapsible soils may be described as essentially qualitative in nature rather than quantitative and it needs to be asked whether this can still be regarded as satisfactory. For example, publications from the symposium on the Genesis and Properties of Collapsible Soils give both theoretical and empirical relationships which could be adopted for southern African conditions: Fredlund and Gan (1994) set out a derivation of collapse relationships from an approach analogous to consolidation theory and incorporating suction pressures, and Lin (1994) presents illuminating test results for loess emphasising the influence of age, overburden pressure, degree of saturation and suction pressure.

Schwartz (1985) pointed out that degree of saturation was a vital parameter in assessing collapse potential and suggested that it should be used as a determining factor for collapse estimates, that is, a critical degree of saturation exists for collapsible soils above which collapse would not occur. Unfortunately this concept would seem to have been largely ignored in practice, although McKnight (1999) reproduced the Schwartz data and added the information from the Mozal site, thus providing a useful illustration of a general relationship between critical degree of saturation and fines content.

Schwartz (1985) also stresses that the concept of effective stress must be taken into consideration in the formulation of any research

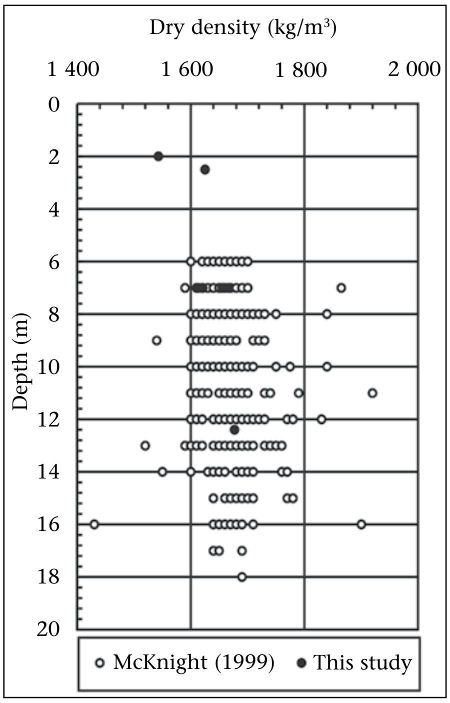

Figure 1 Dry density of the Mozal material

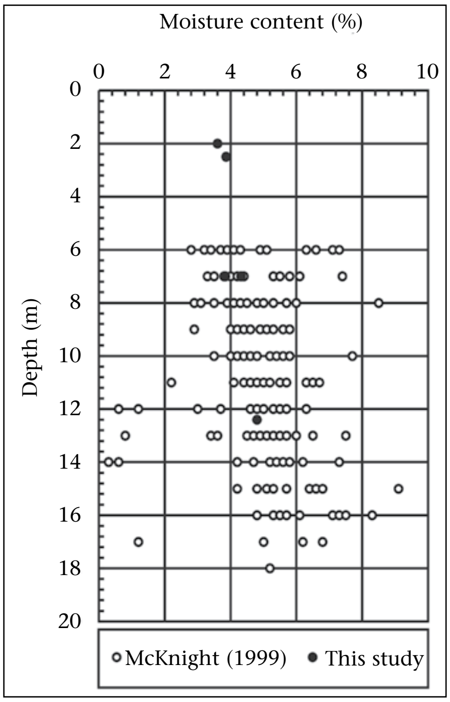

Figure 2 Moisture content of the Mozal material

work related to the collapse phenomena. As this requires accurate real-time measurement of soil suction it is currently, however, not realistic to expect the practising engineer or a commercial soils laboratory to adopt these requirements to correctly model a collapsible soil in terms of effective stress.

RED SANDS OF THE SOUTHERN AFRICAN EAST COAST

The engineering literature of the red sands of the eastern seaboard of southern Africa is less extensive than may have been expected considering the often problematic nature of the material and the extensive development particularly around Durban. Webb and Hall (1967) give a comprehensive description of the formation and properties of the Berea red sands and graphically illustrate the extreme heterogeneity

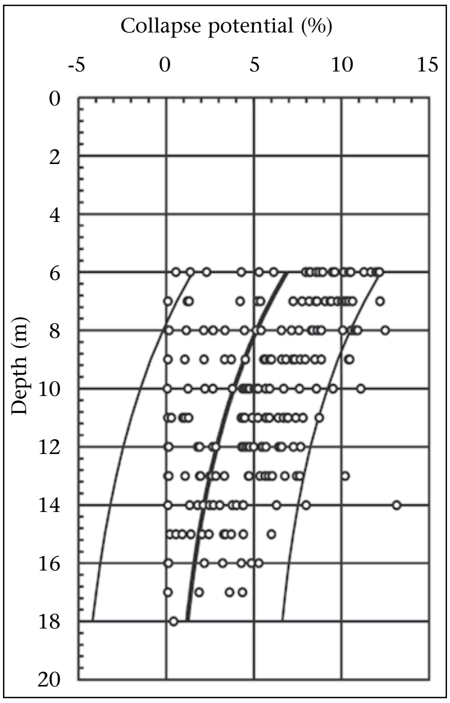

Figure 3 Oedometer collapse potential of the Mozal material (McKnight 1999)

of the material and in particular the wide variation in clay content from less than 5 % to greater than 40 %. They note that collapse has been recognised, particularly with low clay content, but that it has been rare below depths of 5 m.

The extensive investigation for the Mozal aluminium smelter at Maputo provides an example of a thorough, conventional geotechnical investigation of the red sands carried out according to best practice at the time (McKnight 1999). The data is reproduced to show dry density and moisture content against depth (figures 1 and 2) and collapse potential against depth (figure 3). The results indicate that the dry density has a minimum of about 1 600 kg/m³ and a maximum of about 1 850 kg/m³ but with most of the values between 1 600 and 1 700 kg/m³ and apparently no relationship of density against depth (figure 1).

The moisture content (figure 2) is somewhat more scattered but with about 60 % of the results between 4 % and 6 %. A few results are below 2 % moisture and these occur below 12 m depth, which seems unlikely since there appears to be a trend of increasing moisture content with depth.

The degree of saturation varies in the range of about 15 % to 40 % linearly from about 20 % at 6 m depth to about 30 % at 16 m depth. The void ratio averages about 0,55 with a standard deviation of 0,08 and does not appear to show any correlation with depth.

Collapse potential (figure 3) was measured during one-dimensional loading using the standard single oedometer method but with the pressure at inundation being 1 000 kPa. The reasoning for this was that piling was the selected founding method, therefore a high pressure laboratory test would better represent the in-situ conditions. The test results are remarkable in that some of the collapse potentials measured were very high even at depth despite high densities and low void ratios. These, together with soaked pile tests, led to the conclusion that piling through the sands to the underlying stratum was essential considering the extreme sensitivity of the structures to differential settlement.

Recognition of collapse potential by southern African criteria

This Mozal data may be compared with that for other recognised southern African collapsible sands. Brink, Patridge and Williams (1982) and Schwartz and Yates (1980) suggest typical dry densities of collapsible materials to be from 900 to 1 600 kg/m³, although the former warn against assuming higher densities necessarily exclude collapse.

Brink (1985) reproduces 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: $$CP = \frac{1\,672 - \gamma_d}{22}$$

(coefficient of correlation = 0,73)

Mixed origin: $$CP = \frac{1\,590 - \gamma_d}{18{,}9}$$

(coefficient of correlation = 0,77)

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.

Clearly the implication from the recorded evidence from southern African soils is that the Mozal soils, with densities that average 1 650 kg/m³, are unlikely to have significant collapse potential.

Recognition of collapse potential by international criteria

The data in the various papers from the *Genesis and properties of collapsible soils* (Derbyshire *et al* 1994) indicate a fairly large range of dry densities of collapsible soil but generally from about 1 100 to 1 400 kg/m³ for loesses of Eastern Europe and China. Whilst it would be unacceptable to draw a definite conclusion from the dry density information, there can be no doubt that the Mozal soils have an unusually high dry density for collapsible soil.

Figure 4 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 which is reproduced here. It clearly shows that soils with a dry density of 1 650 kg/m³ and clay content less than 30 % would not be expected to have collapse potentials of greater than 1 %. This is deliberately expressed in a conditional sense, rather than as categorical, since the models on which the diagram is based would not be expected to deal with extreme conditions. Nevertheless the weight of evidence contained in the diagram certainly indicates that the Mozal soils at depth would be expected, at the worst, to have a small collapse potential. Schreiner and Popescu (1995) discuss both swell and collapse and in particular the concept of truly collapsible and conditionally collapsible soils which are

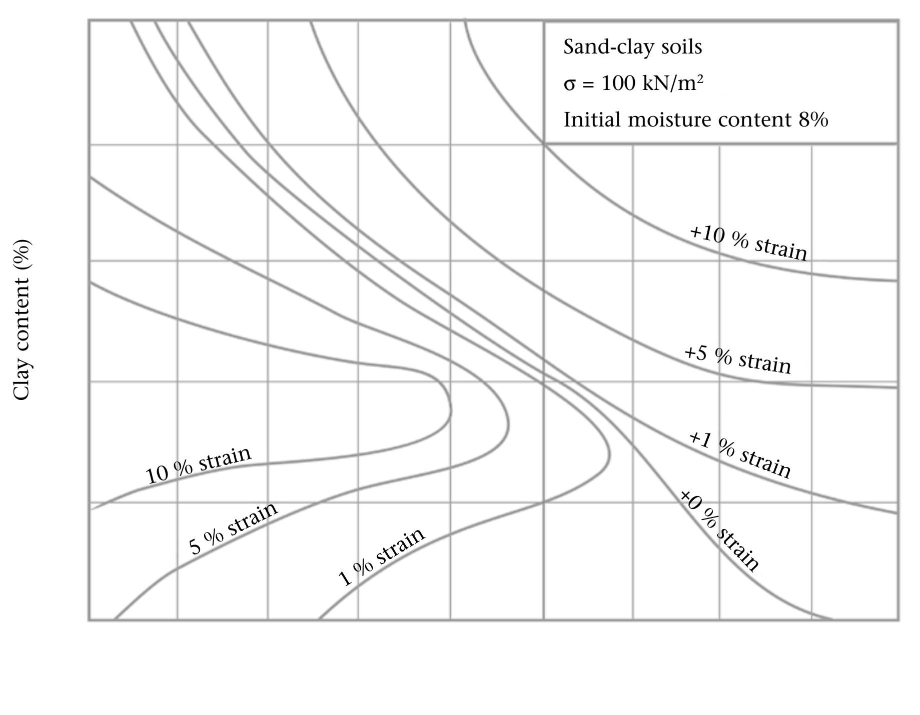

Figure 4 Swell (positive strain) and collapse (negative strain) prediction (from El-Sohby et al 1995)

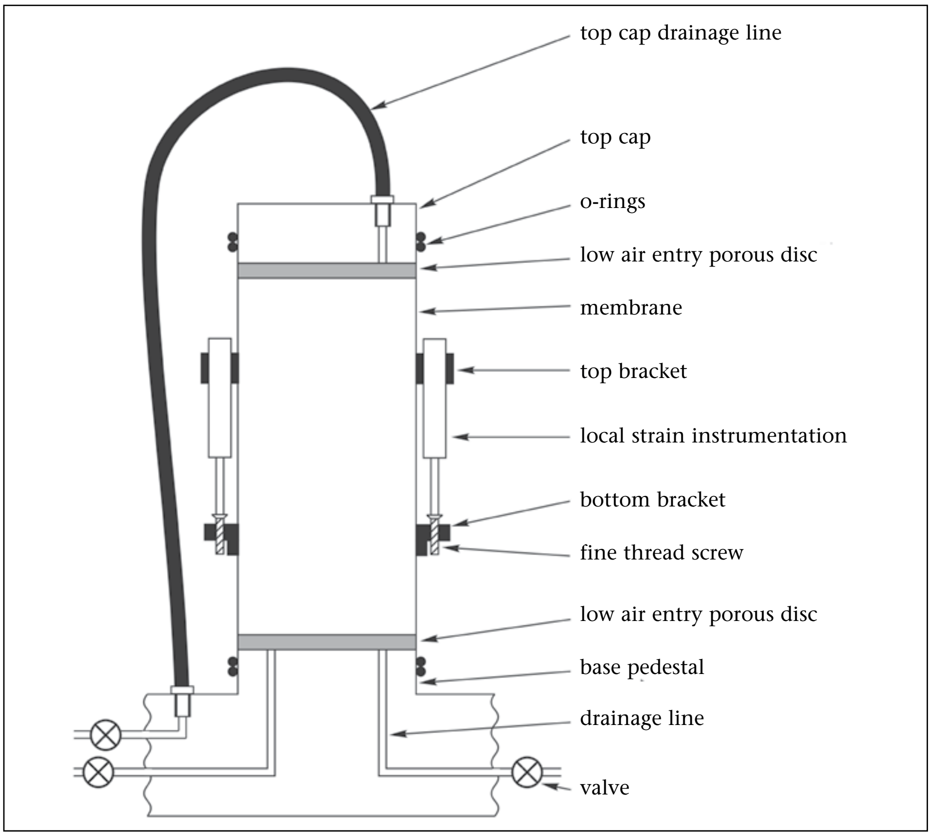

Figure 5 Triaxial collapse potential test

defined in terms of limiting pressures beyond which the rate of change of deformation increases rapidly. They conclude that 'testing for collapse can and should be done using the double oedometer' and it would be interesting to know whether they have experienced problems similar to those reported herein.

A similar argument for the recognition of collapse from dry densities can be advanced on the basis of the void ratios. Typical data from the literature indicates void ratios of 0,8 or greater for collapsible loesses of China (Lin 1994). Also, in the experience of the authors the well-known collapsible Kalahari sands typically have void ratios of about 0,8. It can be seen that the Mozal soils, which have void ratios of about 0,55, fall well below this range for collapsible soils.

Another dataset of importance when considering collapse potential is penetration testing, both standard penetration tests (SPT) and cone penetration tests (CPT). At Mozal these both typically show high values of SPTs in the range 30–70 and CPTs of greater than 30 MPa indicating by normal interpretation very dense and certainly not collapsible material.

Clearly there are major inconsistencies in the data which require resolution. The empirical interpretation of the indirect indicators of collapse, that is density, void ratio and penetration resistance, appears to show that the material has little or no collapse potential. On the other hand, however, there is the apparently supportive data for collapse from the laboratory collapse potential tests and from plate-bearing and pile tests – both of which showed vertical displacements of about 35 mm on soaking.

SAMPLING AND TESTING OF MOZAMBIQUE RED SANDS

During the investigation at the Mozal site the opportunity arose for the authors to sample the red sands carefully and conduct laboratory tests. Hight *et al* (1992) have shown that block sampling produces higher-quality samples than other sampling techniques. The samples used during this testing programme were block samples retrieved by hand from a large diameter auger hole using the methods described by Heymann and Clayton (1999). The samples were covered in numerous layers of aluminium foil and polyurethane film (cling film) to protect them against moisture change. The tests described were conducted on specimens cut from these block samples.

Three types of laboratory tests that are currently not regarded as standard practice were conducted on the Mozal material. These are triaxial collapse tests, pore fluid suction measurements and incremental collapse tests.

Triaxial collapse potential test

A test was developed in which the collapse behaviour of the partially saturated sandy material from the Mozal site was measured in a conventional triaxial apparatus. An important feature of the new test was the use of local instrumentation to measure the axial strains over the central part of the specimen remote from the ends. Numerous instruments suitable for local strain measurements have been described in the literature (eg Heymann 2000). For the tests described, submersible LVDTs with a range of ± 5 mm and a gauge length of 55 mm were fitted to 76 mm diameter specimens. These local instruments avoid bedding errors that occur due to surface irregularities at the interface between the soil and the porous discs. The inaccuracy due to bedding errors becomes pronounced as the length over which the sample collapse is measured becomes small, such as in the case of a one dimensional oedometer with a typical height of about 20 mm. Bedding as small as 0,5 mm on each end of the specimen will result in a bedding error of 5 %, which may well be in excess of the soil collapse. Conducting a collapse test in a triaxial apparatus allows for a larger specimen to be used, but, more importantly, the use of local strain instrumentation entirely excludes errors due to bedding.

During the test the specimen was placed in the triaxial apparatus at the in-situ moisture content. Dry, low air entry porous discs were placed on both ends of the specimen and the specimen covered with a latex membrane in the conventional way. The schematic arrangement of the experimental set-up is shown in figure 5. The cell pressure was increased in increments whilst noting the response of the local strain instruments

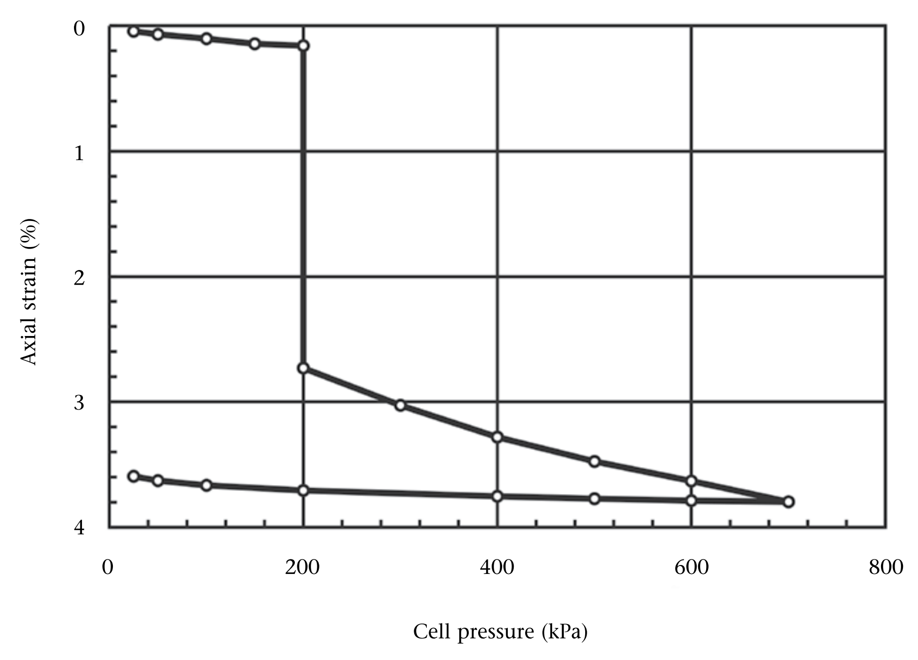

Figure 6 Typical result of triaxial collapse test on Mozal material

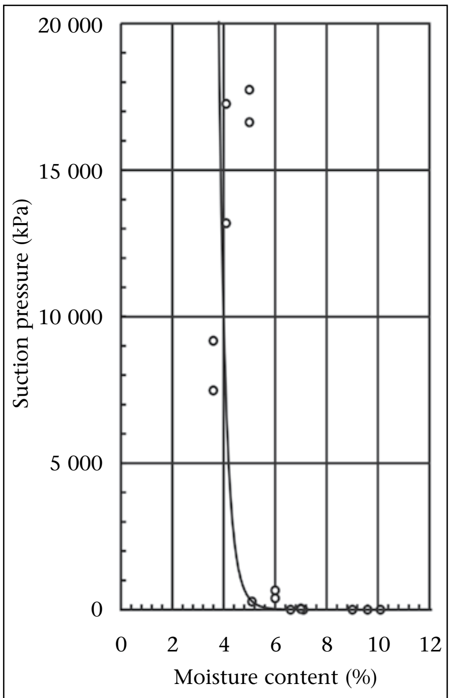

Figure 7 Pore fluid matric suction of Mozal material

for each increment. Sufficient time was permitted between cell pressure increments to allow the creep rate to subside to acceptable levels (< 0,25 % per hour). At the required cell pressure the specimen was inundated with de-aired water at a pressure of 15 kPa. The water entered the sample through the bottom drainage line to allow the displaced air to escape through the top line. The response of the local strain instrumentation was monitored as collapse occurred.

Once creep associated with the collapse event decreased sufficiently, loading was continued by increasing the cell pressure in increments to the maximum pressure required, again noting the response from the local strain instrumentation for each increment. This was followed by an unloading cycle where the cell pressure was decreased in increments, with cell pressures typically corresponding to those used during the loading cycle. The time required for creep to subside at each pressure increment was generally a few minutes prior to inundation, increasing to a few hours after inundation.

The main advantage of this triaxial collapse potential test is the avoidance of bedding errors, which should improve the repeatability of the results. However, the test has some shortcomings relating to the stress condition applied and the strains measured. In general the stresses exerted by a foundation consists of a combination of isotropic and octahedral stresses. The laboratory test imposes only isotropic stresses on the specimen and measures only axial strains. This does not allow a comparison of stress and strain increments on the bases of established constitutive relationships unless isotropy of strains are assumed.

Figure 6 shows a typical collapse test result on the material from Mozal. It is clear that the introduction of water to the soil had an effect on its behaviour and axial strain of 2,6 % was observed during inundation. This magnitude of collapse is within the range of 1 % to 5 %, which was classified by Schwartz (1985) as indicating 'moderate trouble'. However, the difference in volumetric stiffness of the material before and after inundation was much more dramatic. Assuming isotropy, the stiffness before and after wetting was calculated as 114 MPa and 11 MPa respectively, indicating a ten-fold reduction in stiffness on wetting. It was presumed that such a dramatic change in stiffness may be due to a reduction in pore fluid suction and therefore a reduction in effective stress. This mechanism was investigated by conducting pore fluid suction measurements on additional samples from the Mozal site.

Pore fluid suction pressure measurements

Matric suction measurements were conducted using the filter paper method (Chandler & Gutierrez 1986; Houston *et al* 1994b). Specimens were prepared by cutting suitably sized soil samples and adding increasing quantities of water to the various specimens. Whatman no 42 filter paper was placed in contact with the specimens and wrapped with aluminium foil and polyurethane film. The specimens were then left for ten days to allow equilibrium of the suction pressure in the specimen and the filter paper to be reached. The measurements were duplicated by using two pieces of paper for each sample and no significant differences were observed between the measurements made by each pair of filter papers. The suction pressure was taken as the average of the two measurements and the results are shown in figure 7. The figure indicates that the suction pressures in the specimens at the in-situ moisture content were high and reduced dramatically as the moisture content increased. The results also show that a critical moisture content exists at about 6 % above which suction pressures become negligible. The notion of a critical moisture content was further explored by conducting incremental collapse tests.

Incremental collapse tests

Collapse tests were carried out in a special one-dimensional oedometer which allowed the incremental addition of small quantities of water to the specimen via a pinhole at the bottom of the apparatus. The apparatus suffers from bedding effects similar to those present in the conventional oedometer apparatus but allows the collapse phenomenon to be observed as the moisture content increases. Figure 8 shows the results obtained at a vertical total stress of 600 kPa. The initial moisture contents of the three samples varied between 4 % and 5 % and it may be seen that a slight increase in moisture content above these levels triggered collapse settlement. It may also be observed that at a moisture content above about 5,5 % the rate at which collapse increased for an increase in moisture content reduced significantly. This moisture content of 5,5 %, and the 6 % above which pore fluid suctions become negligible, corresponds closely with the critical moisture content of 6,3 % identified by McKnight (1999) above which collapse does not occur for the Mozal material.

In order to advance the argument of the collapse behaviour of the soil as observed above, a conceptual yield model was developed in terms of effective stress theory. This is consistent with Blight (1963), who inferred on the basis of a small data set that the phenomenon of soil collapse is consistent with the principle of effective stress.

YIELD MODEL FOR COLLAPSING SOIL

The yield model for collapsible material presented in this paper is broadly based on the critical state model and specifically on the yield model for structured soils and weak rocks as presented by Leroueil and 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 confirming 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

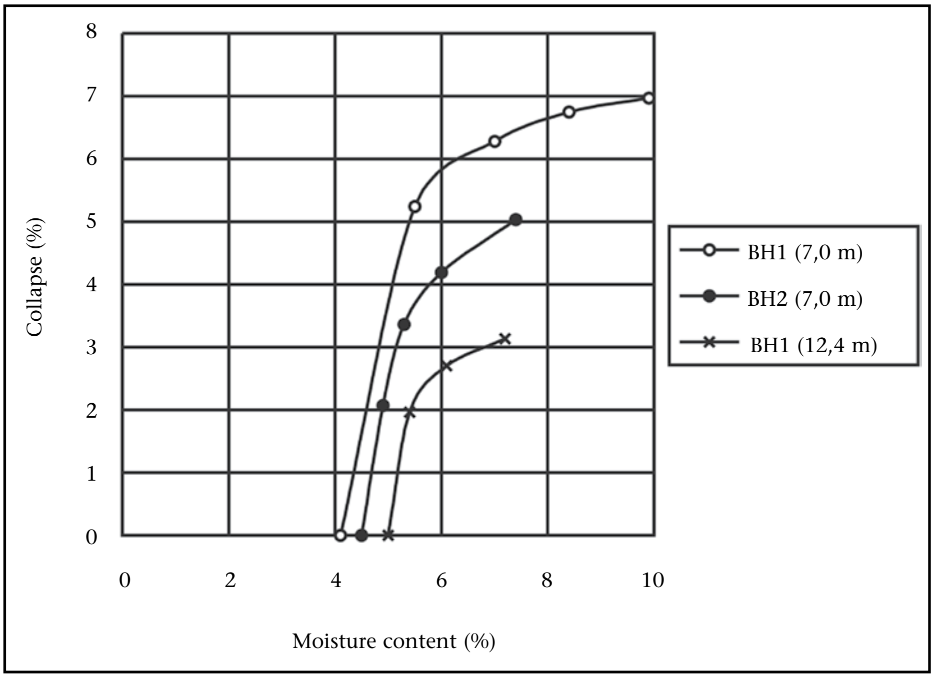

Figure 8 Incremental oedometer collapse results for Mozal material

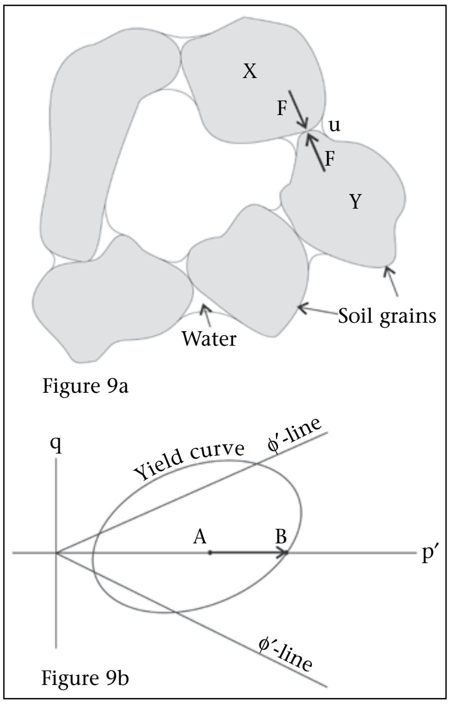

Figure 9 Effect of pore fluid suction on effective stress

these 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 low degree of saturation a change in pore fluid suction and 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 9a. 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. 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 to a change in the total stress on the sample that would, say, result in exactly the same increase in intergranular force. This could only be done if the internal stress within grain X and Y changes. One way to demonstrate the consequence of this difference is shown in figure 9b. 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 the large volumetric strains occur (the sample collapses) and the collapsible structure is lost. Compare this to the same relatively dry sample at stress point A being dried out further. The suction pressures may increase to point B as before. However, 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 due to the increase of the strength of the bonds between the particles 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 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. Leroueil and Vaughan (1990) presented a general yield model for structured soil shown in figure 10a. Three parts of the yield curve may be identified: shearing yield, compression yield and swelling yield. Shear yielding occurs just before shear failure 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 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 10b, rather than during positive p' for a weakly bonded material, as shown in figure 10a.

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 10c the suction is represented by D-B and the overburden pressure by B-A. K_o 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 10c. Wetting the sample will reduce the suction, resulting in a decrease of the isotropic stress 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 10c 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 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 only take place under tensile conditions along this stress path, as shown in figure 10d. In this case the sample will not disintegrate when placed in water. Both of these conditions were observed at the Mozal site. Generally the shallow samples disintegrated while the deeper samples stayed intact.

A general stress path for the in-situ material is shown in figure 10e. 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 resulting in a stress represented by point A'. Since the land surface is at a stable state no collapse of the profile will take place. The metastable soil structure is now supported by the chemical bonding and possibly some remnant suction pressure only.

The stress path of the one-dimensional collapse potential test is also shown in figure 10e. The stress path starts at the in-situ stress point A, after sampling the overburden is removed and the stress 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 K_o line. At this point collapse will occur, that is, large deformations will take place and the soil will be de-structured. The yield surface at point F' now represents a de-structured yield 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.

DISCUSSION

In reviewing the collapse potential data from the one-dimensional oedometer tests (figure 3) on the material from the Mozal site, the first observation is that the scatter of results is very large. Between depths of 6 m and 10 m the collapse potential varies from 0 % to

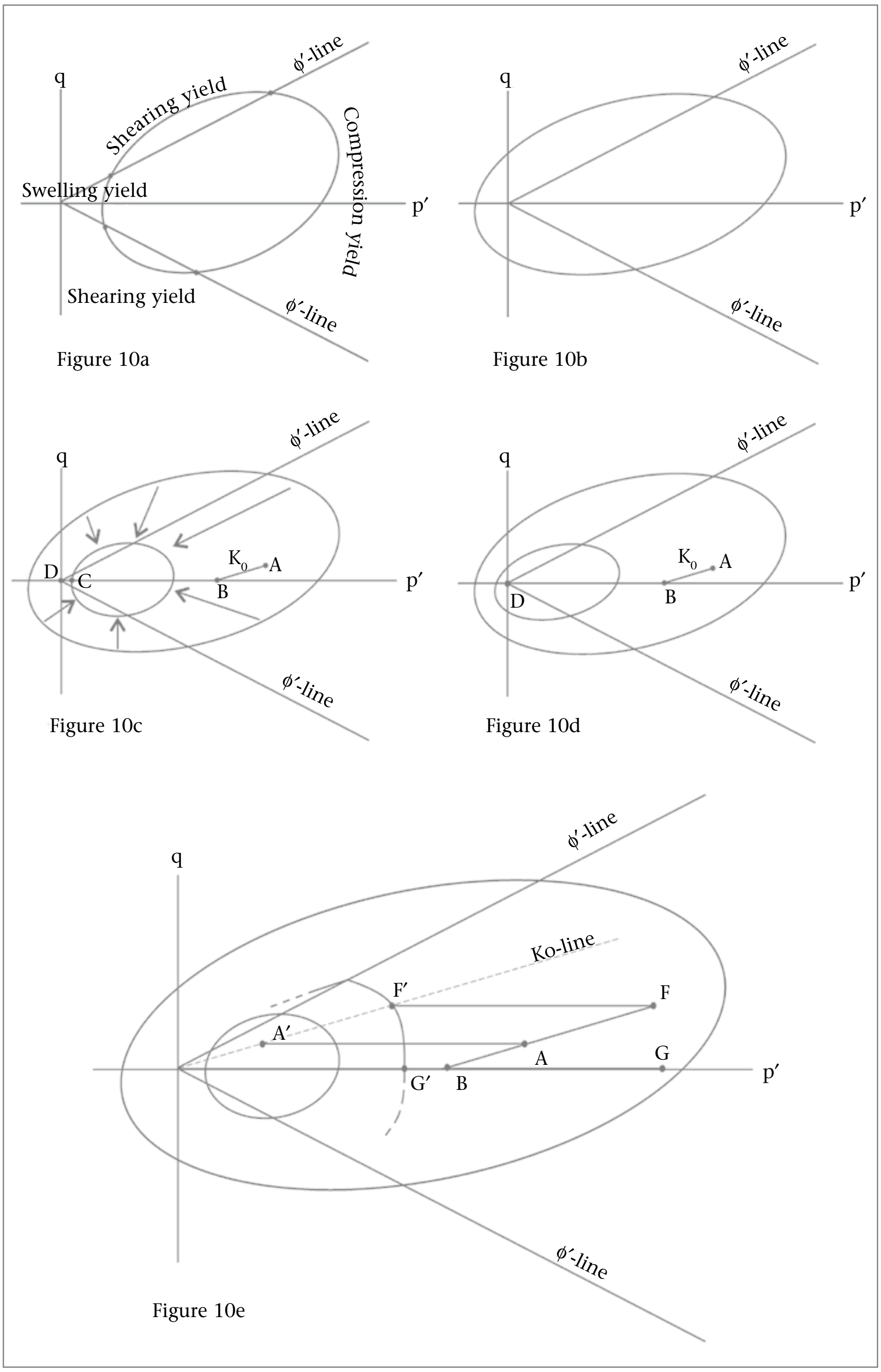

Figure 10 Yield model

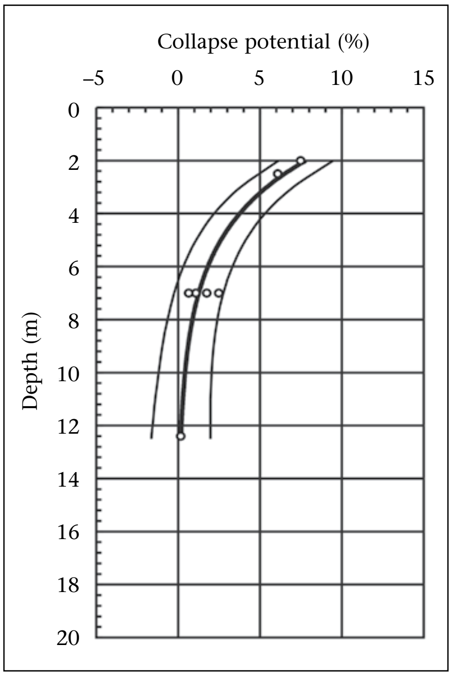

Figure 11 Triaxial collapse test results for Mozal material

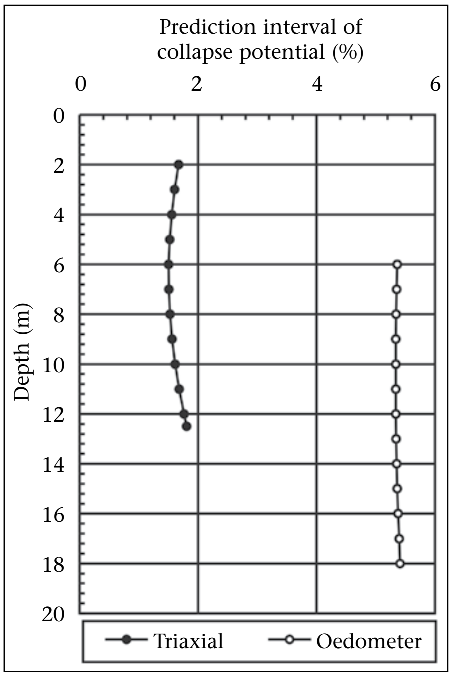

Figure 12 Comparison of triaxial and oedometer prediction interval

12 % and although the samples at the same depth across the site should not necessarily be expected to give similar parameters, this extreme variation gives cause for concern. It was noted before that the collapse potential tests conducted in the oedometer are prone to bedding errors. These errors depend on the asperities at the interface between the soil and porous discs and it could therefore be expected that the errors may vary significantly from one test to another. However, a high bedding pressure prior to inundation, as used for the Mozal testing, should reduce this effect. Also, samples placed inside the one-dimensional oedometer have to be cut meticulously to fit firmly inside the confining ring to provide lateral support to the specimen. If the sample does not fit well inside the ring the Poisson effect will cause lateral strains upon loading and inundation resulting in vertical strains unrelated to one-dimensional loading. Such confinement errors will lead to further inaccuracies in collapse measurements.

In order to investigate the effect of bedding and confinement errors on the variability of the data, a statistical analysis was conducted to compare the scatter of the dataset obtained from the oedometer with that of the triaxial collapse dataset. Figure 3 shows the best-fit trend line as well as the 90 % prediction interval for the oedometer data using the student-t distribution. The prediction interval shows the boundaries between which the result of the next test will lie with a probability of 90 %. For the large number of data points in this dataset (208) the student-t distribution converges to the normal distribution, and the negative values for the lower boundary is a mathematical incongruity as a result of the distribution being symmetrically spread about the trend line. Figure 11 shows the trend line and 90 % prediction interval for the seven results from the triaxial collapse test. The student-t distribution fully accounts for the different dataset sizes, and the prediction intervals from the two test methods may be directly compared, as shown in figure 12. The figure shows that the scatter in the data sets is markedly different, with the spread of the triaxial results approximately three times less than that of the oedometer tests.

Clearly the conclusion must be made that the large scatter indicated by the oedometer test was not only due to the soil behaviour but also an artefact of the test procedure. The procedure includes sampling techniques, sample storage, sample transportation as well as the characteristics of the test itself. Due to the fact that all samples were subjected to disturbance during sampling, storage and transportation, the authors are of the opinion that the most likely reason for the difference in scatter of the data is the inclusion of bedding and confinement errors in the oedometer collapse tests and the exclusion of these errors in the triaxial collapse test.

CONCLUSIONS

The phenomenon of collapse is well known both in southern Africa and internationally and has been extensively documented by Brink (1985) and Schwartz (1985) for the former and more recently internationally in Genesis and Properties of Collapsible Soils (Derbyshire *et al* 1994). The development along the southern African east coast has to a large extent refocused attention on the potential collapse properties of the red sands, and the recent work at Mozal has necessitated a reappraisal of our understanding of collapse and the appropriate testing to determine the probable magnitude of collapse settlement.

There appears to be a major inconsistency in the Mozal data in that the standard single oedometer testing indicates high but erratic collapse potential, whereas all the conventional empirical indicators of collapse, such as dry density, void ratio and porosity, using both local and international models, suggest that the collapse potential should be minimal. Both SPTs and CPTs give very high penetration values but, on the other hand, plate load and pile test showed apparent collapse on inundation during testing. Special triaxial testing with local strain measurements was carried out on very carefully obtained undisturbed samples to resolve these anomalies. This showed, albeit on relatively few samples, a consistent pattern of moderate collapse decreasing with depth with little scatter in the results. Most significantly, however, the testing showed an extremely large, one order of magnitude decrease in stiffness on wetting with collapse of only 2 %, which was ascribed to the reduction in pore fluid suction and shown to be so by further testing. Incremental oedometer tests were also conducted which showed large increases in strain at a well-defined moisture content corresponding to the point where pore fluid suctions become negligible.

A yield model is postulated based on that suggested by Leroueil and Vaughan (1990), which provides some explanation of the observed behaviour. This model should be developed and tested to confirm its validity.

We conclude on the basis of the test results and on the empirical evidence, both locally and internationally, of collapse potential indicators that the large collapse potentials reported for Mozal are misleading in that the soils tested in fact have moderate collapse potential. We suggest the problem lies in the testing procedures of the one-dimensional oedometer collapse test. It is absolutely necessary that meticulous preparation of samples should be carried out to ensure a near perfect fit in the oedometer rings and to minimise top and bottom bedding errors.

An implication of this is that if critical foundation design decisions are to be made based on potential collapse in these materials, then not only should meticulous oedometer testing be carried out, but more sophisticated triaxial testing should also be considered.

A further major implication of the work is that in these materials relatively high penetration test results could be misleading in that small increases in moisture could, by significantly reducing suction pressures, cause a large reduction in stiffness and hence settlement of foundations.

The following rational design approach should therefore be adopted for these materials:

- Model the soil behaviour in terms of changes in parameters with increase in moisture.
- Predict probable changes in moisture and possibly load in structure life.
- Design foundations accordingly.

Ironically perhaps, although the authors are suggesting a new perspective with regard to the behaviour of these soils, in many cases the foundation solution may well be similar to that which would have been appropriate before. The real problem remains that of assessing the risk of significant increases in moisture and it is in this field that we should recognise that our knowledge is often inadequate.

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