Chapter 1

Review of expansive soil behaviour & statement of the research problem

Laboratory and field behaviour of unsaturated expansive soils are reviewed in this chapter. The features of expansive soil behaviour, which have been critically reviewed are soil structure, swelling potential (intrinsic expansiveness), swelling pressure, volumetric (swelling and shrinking) strains, shear strength, cyclic swelling strain and the associated effects of applied external load. The definitions and methods of determining these features are considered and their shortcomings are noted. In addition, the different heave prediction methods and soil models, published over the last 50 years are reviewed. The degree of empiricism or rationality of the models and methods are highlighted. Basing on the review, the chapter concludes by presenting the statement of the research problem, justification of the study, and an outline of the structure of the thesis.

1.1 Definition of expansive soils

Expansive or swelling soils are soils that, because of their mineralogical composition, experience large volume changes or volumetric strains when subjected to moisture changes. They swell on wetting and shrink on drying respectively. (Bolt, 1955; Jennings and Knight, 1957). These soils are commonly referred to in literature as active clays, swelling clays or volumetrically active soils. In this thesis, they are called expansive soils (Gromko, 1974; Gens and Alonso, 1992), reserving the term "active" to the highly swelling clay mineral involved. The ability of the clay mineral to adsorb and absorb water is its intrinsic property, which results from its mineral composition. Schreiner (1987) called it intrinsic expansiveness.

1.1.1 Origin

The origins of expansive soils are concisely summarised by Gromko, (1974), Mackechnie (1984) and Chen (1988). Essentially, the formation of expansive soils depends on a complex interaction of a number of controlling variables such as, parent rock type, weathering and erosion, prevailing climate, local topography and drainage. Expansive soils have a world-wide distribution; their occurrence is not climate specific though they are particularly widespread in arid to semi-humid climate, in which evapotranspiration exceeds rainfall for significant portions of the year. This is partly explained by the theory that lack of leaching in semi-arid zones helps the formation of montmorillonite (Mitchell, 1993). In the arid and semi-arid climates, expansive soils usually exist in an unsaturated state. However, in the wet climates, the soil is fully saturated and the problem manifests when the soil dries out, hence the term "desiccating soil". The term "unsaturated" is herein used to describe both partially saturated and dry states of the soil.

1.1.1.1 Clay-mineral structure

The swelling and shrinking phenomena are caused by the presence of clay minerals that have very large specific surface areas, and hence high water adsorption capacities (Mitchell, 1993). Clay minerals are complex silicates of aluminium, magnesium and iron. The two basic crystalline units, which form the clay minerals are (1) a silicon-oxygen tetrahedral, and (2) an aluminium or magnesium octahedron. The clay minerals are plate-like and very small, being measured in Angstrom (Å) units. However, they have very high specific surface areas. For instance, montmorillonite, an expansive clay mineral, has a specific surface area of up to 800 m^2g^{-1}, compared with $1x10^{-2}$ and $1x10^{-1}$ m^2g^{-1} for coarse and fine sands, respectively. Fookes and Parry (1993) demonstrated that the volume change depends on the clay particle size and the thickness of the absorbed water. In their study, they noted that the ratio between the absorbed water and the particle thickness was 40 for montmorillonite compared to 0.8 for kaolinite. This indicated a theoretical potential volume change for montmorillonite of about 50 times that of kaolinite from completely dry to saturation.

The structure of the different clay minerals is complex. However, schematic representations have been adopted to simplify the presentations (Mitchell, 1993). The clay minerals exist in two or three-layer systems called sheets. Figure 1.1 illustrates the symbolic structure of typical clay minerals.

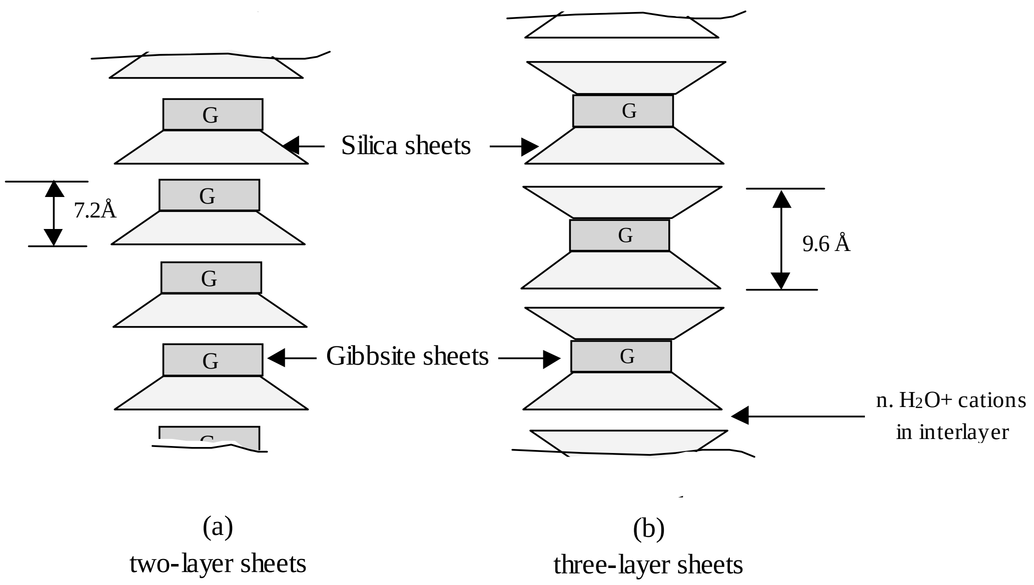

Figure 1.1 Symbolic structure of (a) kaolinite and (b) montmorillonite clay minerals (*after Mitchell, 1993*).

1.1.2 Occurrence

The problems associated with expansive soils are widespread across the world (Donaldson, 1969; Chen, 1988). Problems with expansive soils were reported in Australia, China, India, Israel, Jordan, South Africa, South America (particularly Brazil), Spain, United Kingdom, United States of America (particularly Texas and Colorado), Zimbabwe and several other parts of Africa.

1.2 Structure of expansive soils

1.2.1 Conceptual models for the microstructure level

Extensive study of the particle level behaviour of soils is reported in the literature. Bolt (1956), Oslen and Mesri (1970), Sridharan and Rao (1973), Collins and McGown (1974), and Gens and Alonso (1992) cover comprehensive reviews of the subject. Alonso, Lloret and Gens (1995) reported experimental behaviour of highly expansive double structure clay. A number of theories have been proposed to explain clay compressibility at the particle-water-cation level. The Gouy-Chapman double layer theory (Gouy 1910, 1917; Chapman, 1913) has been successfully applied by Mitchell (1976), Callaghan and Ottewill (1974), Sridharan and Jayadera (1982) and Jayadeva and Sridharan (1982). However, Low and Margheim (1979) and Low (1980,1991) contend that the double layer theory does not satisfactorily explain the experimental swelling results of montmorillonite. They have proposed an exponential empirical relationship that relates swelling pressure to inter-layer distance. Baveye, Verbug and Beilders (1991) and Tessier (1991) reported that direct mechanical effect of suction might significantly contribute to the volume-change behaviour. This is in addition to the osmotic and hydration effects that are addressed by the current concepts. Apart from this, the question of whether or not microstructural deformations are reversible has not yet been fully addressed. Warkentin, Bolt and Miller (1957) reported irreversible deformations on Sodium montmorillonite tests. Ormerod and Newman (1983), and Kraehenbehl Stoeckli, Brunner, Kahl and Muller-Vonmoos (1987) on the other hand, reported slight hysteresis in the water content-suction curves of an illite-montmorillonite clay mixture.

Several conceptual models of particle arrangements and behaviour have been proposed. The conceptual models vary depending on the formulations. In the main, volume changes in expansive soils are known to be a result of physical-chemical interaction phenomena at the clay-platelet level. Alonso Gens and Hight (1987) referred to microfabric as that which controls the conditions of the water in soils, specifically potential or suction. They suggested that the mineralogical composition affects the adsorption component of matrix suction, while the internal geometry controls the capillary component. However, their proposition was based on the results relating to non-expansive to moderately expansive soils. Gens and Alonso (1992) critically reviewed some of the fundamental aspects of soil microstructure in relation to the behaviour of expansive soils. They highlighted the important role played by the various phenomena occurring at particle level in response to external actions (load, chemical phenomenon, and suction changes). The review demonstrated a lack of consensus in describing the behaviour of the microstructural level, particularly for expansive soils. The complexity of modelling the interactions at particle level was evident.

1.2.2 Conceptual models for the macroscopic level

Brackley (1975a) modelled the unsaturated clay soil as a conglomeration of packets of soil particles. The packets are considered completely saturated and the inter-packet voids are filled with air. McGown and Collins (1975) and Collins (1984) proposed three basic microfabric features that make up the structural arrangements of soils namely, "elementary particle arrangements", "particle assemblages" and “pore spaces". They based their descriptors on observations from a study of microfabric features of a variety of natural soils using Scanning Electron Microscopy (SEM). They observed a general correlation between the microfabric of the soils and some engineering soil behaviour such as sensitivity, collapsing and expansiveness.

Schreiner (1987a) proposed a mechanical analogy between the swelling process under decreasing suction and a linear spring system. This analogy likens intrinsic expansiveness to the spring stiffness. He asserted that the intrinsic expansiveness does not change regardless of the changes in stress. This however, seems to contradict the observed results from cyclic swelling tests reported by several researchers. Chen (1965), Chu and Mou (1973), Chen, Lu and He (1985), Dif and Bluemel (1991), Popescu (1980), Osipov, Bik and Rumjantseva (1987) and Day (1994) reported that swelling potential changed to a limiting value, as the number of wetting and drying cycles increased.

1.2.2.1 Effect of history and stress path

Schreiner and Burland (1991) reported an alteration of the microfabric due to changes in applied stress and pore water pressure. They noted that changes in applied stress (stress path) and pore water pressure (suction) significantly altered the soil fabric.

Al-Homoud, Basma, Malkawi and Bashabsheh (1995) investigated the effect of cyclic wetting and drying on the microfabric and swelling characteristics of six recompacted expansive soils of liquid limit (LL) = 65-90 per cent and plasticity index (PI) = 40-80 per cent. After each cycle, the swell and swelling pressures of the soils were measured. Scanning Electron Microscopy (SEM) was used to study the soil microstructure before and after cyclic swelling. Reconstruction of the clay microfabric with each cycle was evident. Initially the soil had a turbulent fabric with a low degree of micro-aggregate orientation. After five cycles, the microstructure became uniform. The investigation indicated that there was a continuous rearrangement of particles during cyclic swelling. The same result was shown in the work of Hussein and Adey (1998). Rao and Satyadas (1987) noted particle aggregation. However, Day (1994) suggested that drying and wetting cycles changed the initially dispersive structure to a flocculated structure.

1.2.3 Summary

The important role played by the soil structure in expansive soil behaviour is clear. The influence of stress path on the swelling behaviour of expansive soils is apparent. This seems to indicate that the fabric of remoulded soil samples does not correctly model natural soils. This conclusion was emphasised by Janbu (1998) when he commented on how remoulding samples generally destroys the stress history of the soil, and hence fail to yield results of immediate practical relevance. Expansive soil behaviour is sensitive to stress path i.e., the effect of external load or whether the soil is initially in a wet or dry state. Therefore, use of laboratory prepared clayey specimens, which have different microfabric features, could lead to misinterpretation of the mechanical behaviour of natural expansive soils.

It is evident from the review that soil behaviour at the microstructural level is complex and not yet fully understood. In addition, the available concepts of the microstructure are contradictory in many respects. Accordingly, there is insufficient information to help develop soil models for expansive soils based on the behaviour at the microstructural level, as presently understood.

1.3 Laboratory investigation of expansive soils

1.3.1 Stress paths commonly investigated in the laboratory

In the last 50 years, a lot of laboratory experimental work was done on unsaturated expansive soils (Alonso, *et al*., 1987; Buisson and Wheeler, 2000). The work included mainly, oedometer testing (with or without suction control) and to a lesser extent, triaxial or direct shear testing. Different researchers used various test procedures. The variations were largely in (a) the initial state of the test sample (natural undisturbed or remoulded), (b) different initial surcharge load (*in situ* overburden, 1kgPa, 7kPa etc.), (c) initial water content (natural or optimum), and (d) stress path adopted (constant load, constant volume, swell-under-load etc.).

The volume change features of an unsaturated soil are commonly discussed in a two-dimensional stress space (p, s), where "*p*" is mean net total stress (or mean total stress), and "*s*" is the soil suction. In the case of oedometer tests, p is taken to equal the total vertical stress. Typical stress paths followed in the oedometer test (fig. 1.2) were reported by Alonso *et al.* (1987), and are explained below.

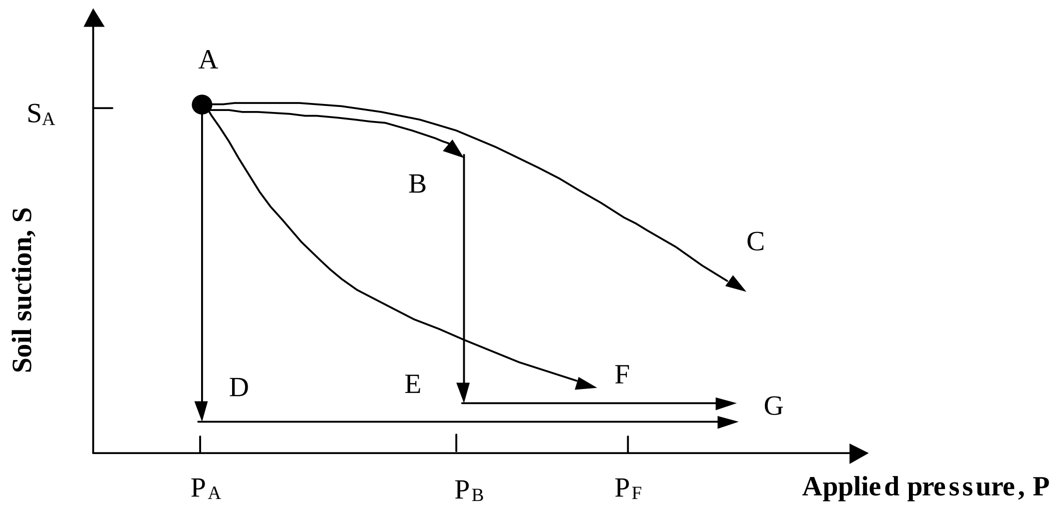

Figure 1.2 Typical stress paths in the (p, s) stress space (*after Alonso et al. 1987*)

The initial state (A) of the sample corresponds to a given initial suction value (S_A) and a low applied stress (P_A).

- Path (A-C) is followed by a sample undergoing consolidation (compression) at initial water content
- Path (A-D-G) involves a sample being saturated (A-D), followed by saturated compression (convention consolidation) along path (D-G).
- Path (A-B-E-G) involves compression of the sample at natural water content along (A-B) to stress (P_B), saturation along (B-E) under constant stress (P_B), and saturated compression (E-G).
- Path (A-F) is a swelling pressure test, with no volume change.

As indicated in fig. 1.2, the specific stress paths imposed on the test samples usually involved a suction decrease (wetting the sample), and different sequences of vertical loading. Stress paths involving constant or increasing suction were rare. Therefore, the effects of suction reversal on soil deformation can not be readily appreciated in most cases. Use of the oedometer cell invariably meant that one-dimensional soil behaviour was investigated, despite the three-dimensional nature of volume change. Several attempts were made to evaluate three-dimensional swell, with varying degrees of success.

1.3.2 Stress state variables

The effective stress principle proposed by Terzaghi (1936) has been remarkably successful in describing the stress-strain behaviour of fully saturated soils (Rendulic, 1936; Bishop and Eldin, 1950 and Skempton, 1961). This led to the belief that the effective stress principle governs the behaviour of soils over all ranges of degree of saturation, with full saturation being one boundary condition.

Relentless efforts to characterise unsaturated soils in terms of effective stress are recorded in the literature. The works of Aitchison and Donald (1956), Bishop (1957), Bishop and Donald (1961), Jennings (1961), Coleman (1962), Bishop and Blight (1963), Burland (1965), Matyas and Radhakrishna (1968) can be cited in this regard. Bishop and Blight (1963) and Burland (1965), proposed that the volume change in unsaturated soils could be independently related to the net total stress and suction stress variables. Fredlund and Morgenstern (1977) formally proposed that any pair of the following stress fields forms a suitable framework to describe the stress-strain-strength behaviour of unsaturated soils: (i) the net total stress, σ (ii) the effective stress, σ' and (iii) soil suction (U_a-U_w). In these expressions, U_a is pore air pressure and U_w is pore water pressure. Accordingly, unsaturated soils and expansive soils have since been characterised in terms of suction. Vanapalli, Fredlund and Pufahl (1999) referred to the soil-water characteristic curve as a conceptual and interpretative tool, by which unsaturated soil-behaviour can be understood.

The concept of soil water potential has been adopted to describe the effect of the forces acting on an infinitesimal body of water in the existing force field. The term “soil water potential” denotes the specific potential energy of soil water to that of water under standard reference state. The concept has been very useful in visualising soil water phenomena. For instance, it has helped in unifying the phenomena of water retention and movement. However, its focus on pore water seems to be a limitation in soil mechanics, where the focus by definition, is on the soil grains.

Not much work has been done regarding expansive soils (Alonso, Gens and Josa, 1992; Gens and Alonso, 1992). Frydman (1992) postulated that swelling pressure is equal to the internal effective stress of a swelling soil. However, he lacked sufficient data to validate the hypothesis. The approach has been to treat expansive soil behaviour as an extension of unsaturated, non-expansive soil behaviour.

1.3.2.1 Swelling pressure

Sridharan, Rao and Sivapullaiah (1986) reported of a generally agreed definition of swelling as "the pressure required to hold the soil at constant volume, when water is added." The definition seems to be satisfied by the three different test procedures for determining swelling pressure reported by Brackley (1973). However, Sridharan *et al*. (1986) noted that the three procedures that were reported by Brackley (1973), gave significantly different swelling pressure values for a given soil. The difference in swelling pressure values appears to stem from the different stress paths associated with the different testing methods.

1.4 Laboratory stress-strain behaviour

1.4.1 One dimensional free swelling

Sridharan *et al.* (1986) reported results of a comparison of the three basic test procedures for determining swelling pressure, and the relative influence of the factors affecting the swelling pressure of soils. The factors studied included time effects, effects of stress path, initial densities, water content and compactive energy employed in the specimen preparation. Remoulded samples of black cotton soils (80 per cent<LL<108 per cent and 44 per cent<PI<71 per cent) were used. Twenty-five specimens from eight different soils were tested. They noted that a rectangular hyperbola reasonably represented both the time-swelling pressure relationship for the constant volume test and the time-swell relationship. The effect of initial water content on swelling pressure was relatively less significant. This, they concluded, was consistent with osmotic pressure theory (double layer theory).

However, Brackley (1973) conducted free swell tests under a token load of 1kPa. The compacted samples were of weathered norite (LL = 89 per cent, PI = 57 per cent), from a site in South Africa. He concluded that swelling strain was dependent on original void ratio and that the final swell was strongly dependent on the original water content, and hence suction. Kassif, Baker and Ovadia (1973) published results that were obtained in an osmotic suction controlled oedometer cell. Remoulded samples of a high plasticity clay (LL = 72 per cent, PI = 48 per cent) were prepared at varying initial water content, and a common dry density of 14.7kNm^{-3}. The samples were then equilibrated at different suction and initial loads. The researchers concluded that the first stages in suction reduction induced small swelling strains compared with the final stages. Swelling along a suction reduction path takes place at an increasing rate. For a given void ratio, swell strain is inversely controlled by applied stress and directly controlled by suction. However, Justo *et al.* (1984) observed that the larger amount of swell tended to take place at low suction values. Richards (1984) and Josa, Alonso, Lloret and Gens (1987) reported similar results on suction controlled oedometer and on isotropic swelling tests respectively. Thus, Kassif *et al.* (1973), Justo *et al.* (1987) and Abduljauwad *et al.* (1993) reported a strong dependency of swelling pressure on initial suction.

Meanwhile, Yong (1973) realised that the volumetric strains of a swelling soil were a result of the mobilised internal pressure. He therefore developed an analysis, with a closed form solution, for predicting volumetric strains using internal pressure.

1.4.2 One-dimensional swell-under-load

Fig. 1.3 is a summary of the stress paths for the different ways of wetting an oedometer sample (Justo *et al*., 1984). Blight (1965b), Kormonik and Livnelo (1967), Escario and Saez (1973), Kassif *et al.* (1973), and Popesu (1979) and Justo *et al*. (1984) reported that externally applied vertical stress controlled the amount of swell experienced by the soil sample. Yevnin & Zaslavsky (1970) and Brackley (1980) reported similar results for both remoulded and undisturbed samples. Pidgeon (1987) carried out swell-under-load tests on several undisturbed samples and concluded that the relationship between percentage swell and the logarithm of the applied pressure was linear. He proposed that the relationship be considered universal.

While these findings are useful in showing the general soil behaviour under applied load, they are not explicit in handling suction in relation with the applied external stress. However, the results of Habib *et al* (1992a) and Habib *et al.* (1993) are sufficiently comprehensive to rationally establish the trends in stress-strain behaviour of loaded soil. During the tests, the soil samples were subjected to various loading and unloading stress paths of both suction and external pressure, while the lateral swelling pressure was measured. The researchers made the following observations. (1) Void ratio change was influenced more by intensity of vertical stress, while water content was influenced more by suction changes and (2), there was merit in plotting the vertical strain versus effective mean stress. Lastly, both vertical and lateral swelling pressures attained maximum values before complete water saturation. It is pointed out though, that their analysis did not attempt to reconcile swelling pressure and applied loads in terms of developing a rational framework.

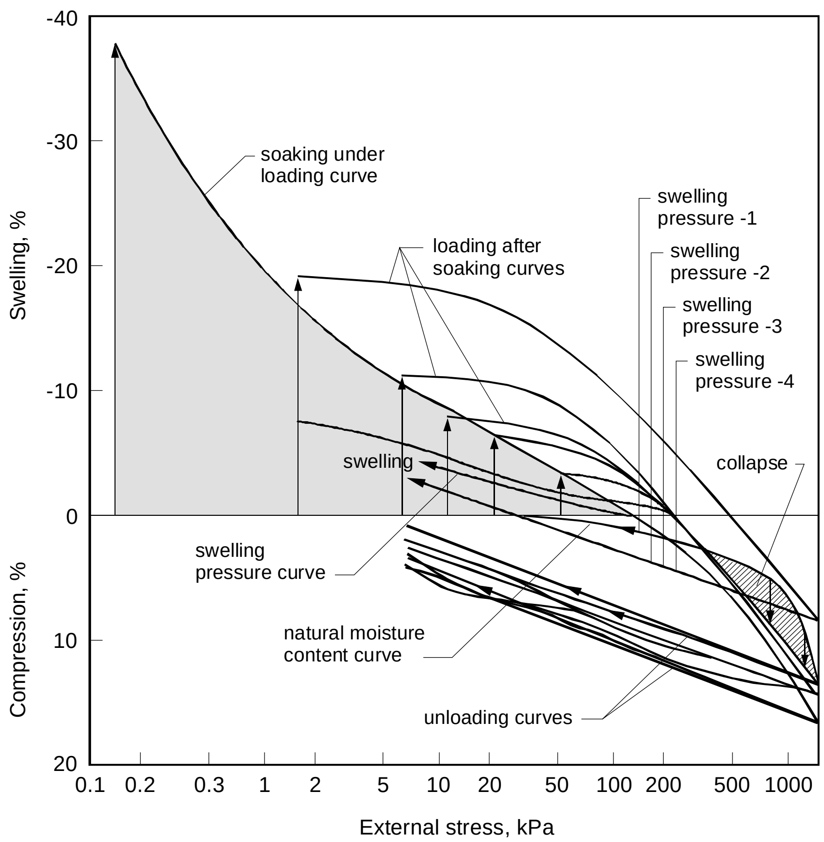

Figure 1.3 External stress - strain plot (*after Justo et al. 1984*)

1.4.3 Shrinking strain

Sridharan and Rao (1971) broadly explained the shrinkage phenomenon as being initiated by the increase in capillary forces due to the surface tension of the pore fluid, with the resultant volume reduction being dependent on the shear resistance offered by the soil. The shear resistance is a function of (i) normal forces acting between particles and at the particle contacts, (ii) the frictional properties, and (iii) the electric attractive and repulsive forces. Volume decrease continues for as long as the capillary forces are larger than the internal resistance generated by the soil. The pressures generated during the shrinking process are enormous, as evidenced by the high density attained from drying, which cannot be obtained by any usual compacting force in the laboratory (Rao and Satyadas, 1985).

The factors that affect shrinkage magnitude, as summarised by Rao and Satyadas (1985), are percentage of clay in the soil, type of clay mineral, mode of geological deposition, particle arrangement or fabric, overburden pressure, degree of weathering and exchangeable cations, orientation of soil fabric, and initial water content. Hyenas and Stirk (as referenced by de Jong and Warkentin, 1965) reported that the shrinking process involves four distinct stages: structural, normal, residual and no shrinkage.

1.4.3.1 Shrinking path

Rao and Satyadas (1985) investigated the shrinkage of an expansive black cotton soil. Compacted soil specimen, at different initial water contents and cured under controlled temperature and humidity, were used in the experiments. The soil had LL = 97 per cent, PL = 32 per cent, SL = 8.6 per cent. The results indicated that volumetric shrinkage has a unique shrinkage path in terms of water loss, but is independent of water content or rate of shrinkage. In contrast, the linear shrinkage paths are dependent on conditions under which shrinkage takes place.

1.4.3.2 The shrinkage limit

Williams and Sibley (1992) investigated the possible links between the shrinkage limit and distinct changes in other properties of a clay soil undergoing drying. The properties considered are volumetric air content, heat of wetting, tensile strength, total suction and thermal resistivity of the soil. Undisturbed soil samples were used. The soil had an average LL = 73 per cent, PI = 51 per cent and SL = 12 per cent. They observed a distinct change in the trends of the properties of the soil at the shrinkage limit, when plotted against water content. They linked the observed changes to possible changes in structure of the soil solids and the disposition of the pore water within that structure. Their major conclusion was that the shrinkage limit marks a fundamental change in the behaviour of the soil.

1.4.4 Discussion

The literature survey has revealed no record of rationally measured stress-strain behaviour, in terms of the internal soil response, save the work of Habib *et al* (1992a) and Habib and Kurube (1993). The apparent lack of agreement by researchers on the measurement of swelling pressure seems to suggest a lack of rational understanding of expansive soil behaviour. The test procedure that involves continuously increasing the external pressure provides results that show the general trends in the soil behaviour. For instance, the works of Sridharan *et al* (1986) produced the following useful qualitative observations that have been recorded by other researchers. The variation of swelling pressure with time, for a flooded soil sample, has a rectangular hyperbolic shape. The swelling pressure depended on the initial water content (suction), with high suction giving high swelling pressure. Kassif *et al*. (1973), Justo *et al.* (1987) and Abduljauwad *et al*. (1993) reported a strong dependency of swelling pressure on initial suction.

On soil shrinking, the work of Williams and Sibley (1992) appears to be fundamental. It points to the proposition that the classification data could be linked to the internal effective stress of expansive soils. However, this has not been done yet.

1.5 Soil models for expansive soils

Expansive soils can be modelled with respect to total heave (short-term), long term heave (time-related) and differential heave. Total heave-prediction received most attention in the last 50 years. Differential heave prediction has received the least attention, despite the fact that it, rather than total heave, is generally responsible for major structural damages (Pidgeon, 1987). The efforts made toward modelling of expansive soils are reviewed in this section, starting with empirical methods and moving on to advanced constitutive models.

1.5.1 Total heave prediction

1.5.1.1 Empirical methods

Many empirical methods have been proposed to correlate swell with soil properties such as plasticity index, shrinkage limit, colloidal content and clay fraction (Holtz and Kovacs, 1981). These prediction methods have met with varying degrees of success. The major limitation of these empirical methods, is that soil heave is stress related and not soil-type related. Classification data may not be used to predict strains because they do not involve stress changes in any way (Schreiner, 1987). He noted that in their simplicity, most of these models have negated the fundamental requirement that strain changes is a result of stress changes. Thus, most of these methods are of relatively little value except under very specific stress and suction or water content conditions. Any attempts to use them may lead to false predictions.

1.5.1.2 Semi-empirical methods

The inclusion of water content in heave prediction models is considered a partial improvement to the classification data approach (Schreiner, 1987a). Most of these methods are based on the use of the oedometer. Fredlund, Hansan and Filson (1980) reviewed the available methods. Pellissier (1991a) summarised 35 different methods in his state of the art report. Fredlund and Rahardjo (1993) listed thirteen methods that utilise oedometer test results. They listed the definitions of the volume change indices with respect to suction changes, as reviewed by Hamberg (1985).

1.5.2 Differential heave prediction models

The prediction of differential heave has not received justice, given its significant role in damaging structures built on expansive soils. For a long time, the only published recommendations were the works of Templer (1957), Jennings and Kerrich (1962), and Donaldson (1969, 1973), where total heave is multiplied by an empirical factor to get differential heave. In general, total heave was taken to be twice differential heave by the rule of thumb commonly used to predict differential settlement. Table 1.1 reproduced from Williams, *et al.* (1985) gives recommended differential heave for structures with length to height (L/H) ratios of 4 to 5 (Pidgeon, 1979).

Table 1.1 Types of construction for various heave magnitude (Williams, et al., 1985)

Type of construction	***Estimated total heave (mm)***	***Corresponding maximum deflection ratio***	***Estimated additional cost increase (%)***
Normal-continuous brick walls on strip footings	0-6	1:4 000	0
Modified normal high fan-lights reinforced footings and lintels	6-12	1:2 000	1-3
Split construction with reinforced brickwork	12-50	1:480	5-10
Piles to limited depth with split construction and reinforced brickwork	50-100	-	20
Underreamed piles with suspended floors	100+	-	30+
Stiffened raft foundations	-	-	7-15

One of the most recent reviews on differential heave are the works of Pidgeon (1987) and Jennings (1988). He recommended that differential heave be taken as the difference between the total heave at the centre and that at the edge. In each case, the total heave should be calculated using soil suction values in the appropriate equations, or the suction is converted to the corresponding change in moisture content.

1.5.3 Advanced conceptual models

Gens and Alonso (1992) presented what they called a "first proposal" of the framework for describing the behaviour of unsaturated expansive clays. Alonso, Gens and Gehling (1994) developed it into a constitutive model. The model is based on the suction concept. Buisson and Wheeler (2000) presented a qualitative framework for unsaturated soils, which allows hydraulic hysteresis. It builds on Alonso *et al.*'s (1987) model for unsaturated, non-expansive soils.

Earlier, Frydman (1992) proposed an effective stress model based on the concept that the effective stress in the initially unsaturated soil is equal to the swelling pressure of the swelling clay under completely constrained conditions. The initial effective stress would then be obtained by carrying out a swelling test, in which the confining stress is continuously adjusted in order to keep the sample dimensions constant. He presented a limited amount of laboratory data, which appeared to support the model. However, the model did not accurately predict the laboratory results obtained by Holtz and Gibbs (1956).

1.5.4 Discussion

Gens and Alonso (1992) noted that their conceptual model did not allow for the possibility that the effects of a microstructural volume change on the macrostructure could be different from that due to loading or collapse. In addition, the existence of particle bonding could also significantly contribute to the irreversibility of strains caused by microstructural swelling. Apart from this, there remains the task of determining a model suitable for calculating volumetric strain at particle level. Accordingly, Alonso *et al.*'s (1994) constitutive model inherited the same limitations inherent in the conceptual model.

Buisson and Wheeler (2000) on the other hand, proposed a framework, which centres on the degree of saturation. However, the emphasis on voids may be a problem to swelling soils, whose voids fill-up with adsorbed water during hydration.

Apart from this, the model may be subject to similar limitations as those for the models by Gens and Alonso (1992) and Alonso, Gens and Gehling (1994).

Frydman’s (1992) attempts to characterise expansive soil behaviour to the effective stress appears rational because he makes reference to swelling pressure, a quantity which arises from the swelling phenomenon and has units of effective stress. The inconsistencies with the work of Holtz and Gibbs (1956) appear to stem from the uncertainty surrounding the interpretation and measurement of swelling pressure, as noted by Sridharan *et al.* (1986).

1.6 Summary of the literature review

The pertinent conclusions regarding the foregoing literature review are as follows.

- The real picture of the interaction mechanisms at particle level appears more complex than envisaged. Any reliable interaction model for the microstructural behaviour should be amenable to verification at the macrostructural level. Such a model could form a rational base for modelling the macroscopic behaviour of expansive soils.
- Physical-chemical interactions at the clay mineral level play an essential role in the volumetric behaviour of expansive soils.
- The generally investigated stress paths involved wetting of the sample. However, the stress paths do not seem to provide a coherent understanding of the swelling soil behaviour during the wetting process.
- There is no agreement on the laboratory determination of swelling pressure. In addition, there is no evidence to show that horizontal swelling pressure is accounted for in the current formulations. Consequently, the state of stress of the soil remained unknown.
- The available empirical and semi-empirical heave-prediction methods are limited to the respective geographic settings, from which they were deduced. Several researchers have expressed reservations to their use.
- There is no soil model for expansive soils, which focuses on the mechanics of the soil grains. The authors of the only constitutive model (Alonso, Gens & Gehling, 1994) earlier on acknowledge the too-simplistic nature of their framework (Gens and Alonso, 1992). In addi-

tion, the model is based on the existence of theoretical models of the particle phenomenon, which is a subject of much debate. Accordingly, Buisson and Wheeler's (2000) formulation may be similarly limited.

1.7 Statement of the research problem

1.7.1 Problem

Civil engineering structures built on expansive soils often suffer damages in the form of cracks and distortions. The structures often affected include aircraft runways, light buildings, railway lines, retaining walls, roads, shallow underground service lines, concrete canal linings and swimming pools. The volumetric strain induced in the soils by seasonal suction changes cause differential movements of the superstructure at the soil-structure interface. This in turn stresses the whole structure leading to development of cracks and distorts, thus rendering the structures unusable (Aitchison *et al.*1965; Yoshida *et al*., 1983). Not only are the damages costly and unsightly, but they adversely affecting the performance of the structures. For instance, doors and windows jam, buried pipes burst, railway lines move out of alignment and pavements become uncomfortable to drive on.

Meanwhile the high capital costs associated with the current durable foundation solutions are beyond the reach of the majority of the world's population. For instance, there is a pressing need, particularly in developing countries like Zimbabwe, to build many low-cost houses to reduce the housing backlog, a problem emanating from rapid urbanisation. This, together with the need to optimise land use, inevitably results in the utilisation of sites with such problem soils.

1.7.1.1 Attempts to address the problem

Over the last four decades, relentless efforts were made to understand and solve the problems associated with engineering on expansive soils. Initially the philosophy was to identify the expansive soils using simple field indicators and laboratory tests, and to avoid them as much as possible. However, as it became apparent that the problem of expansive soils was widespread, efforts to understand and address the problem gathered momentum. Both full-scale field tests and laboratory experimentation were employed in this regard. Notable contributions to this work were cited in the relevant sections of the literature review. However, the published methods have met with different levels of success, largely because of their empirical or semi-empirical nature. The outcome of the extensive work was several empirical and semi-empirical design approaches for stiffened rafts. Pidgeon (1979) reviewed foundation options for expansive soils. However, Pidgeon (1986) noted that of the 21 methods he compared, only two adopted, in some way, a rational approach. Thus, the published methods have met with different levels of success, largely because of their empirical or semi-empirical nature. Their lack of rationality presents a limitation in application to different environments.

Clearly, the question of engineering on expansive soils is not yet fully addressed. Soil characterisation, particularly the stress-strain relationship, remains a prerequisite to sound geotechnical engineering. There still remains the need for fundamental studies of expansive soil behaviour as leading to developing rational design and construction methods for structures build on expansive soils.

1.7.2 Justification and validity of the study

From the literature study, two things are clear. Firstly, the problems associated with civil structure built on expansive soils are diverse. In addition, the financial commitments to addressing or averting the problems are very high. Jones and Holtz (1973) estimated damages and losses to buildings due to expansive soils in the United States of America to be about US$2,2 billion, coming second and tie with hurricane wind/storm surge. Wiggins, Slosse and Krohn (1978) reported damages due to expansive soils as one of the six major natural hazards in the United States of America, the other five being earthquake, landslide, hurricane, tornado and flood. According to Krohn and Slosson (1980), the United States of America spent US$7 billion per year on costs associated with damage to all types of structures built on expansive soils. Williams and Pellissier (1991) reported that in South Africa over R100 million is spent annually on effecting remedial works on buildings on expansive soils, citing R126 million in 1988 and R194 million in 1989. Snethen and Huang (1992) inferred that expansive soils, then in third place behind riverine flooding and hurricane wind/storms surge damage, would probably be surpassed by only hurricane wind/storm surge by the year 2000. Cardoso, Bueno and Lima (1992) estimated that 30 per cent of partial and total failure of civil engineering structures in Brazil, mainly those related to pavement construction and slope stability, are induced by expansive soils.

Secondly, it is asserted that a lack of a clear understanding of the mechanical behaviour of expansive soils during wetting is central to the failure to rationally deal with the problem, from an engineering point of view. For instance, there is presently no rational method for the analysis and design of raft foundations on expansive soils. The available design methods have significant shortcomings, both in their theory and degree of empirical simplification (Pidgeon, 1980). That is, they are dependent on geographical setting. Accordingly, knowledge of the soil's mechanical response to water flow is prerequisite to sound engineering on expansive soils.

1.7.3 Objectives and scope of this study

The primary objectives of this research work were outlined as follows:

- To develop a new concept for visualising the soil swelling phenomenon. The concept places emphasis on the soil solids rather than the soil water.
- To characterise the effective stress-strain behaviour of an unsaturated expansive soil in terms of the effective stress.
- To define and determine the intrinsic soil property and develop an effective stress model for the investigated unsaturated expansive soil
- To establish the link between the internal effective stress and soil structure and thus rationalise the consistency limits.

1.7.3.1 Scope of work and limitations

The scope of the work was limited to the study of one type of unsaturated expansive soils. The soil studied is of Zimbabwean origin. It was sampled from Avondale stream within the University of Zimbabwe campus. The study was carried out on undisturbed soil samples undergoing wetting. The research focused on mechanical characterisation of the expansive soil as an engineering material. Accordingly, the soil was tested with out the influence of external load.

In view of the complexity of the swelling phenomenon, the study was restricted to the effects of water flow on the stress-strain behaviour and change in soil structure. Time effects, cyclic swelling and soil shrinking were not covered in the study. Likewise, water flow in the swelling soil was not fully treated. It was covered to the extent that it relates to the change in soil consistency. Accordingly, unless specified, all rate effects were with respect to the change in soil structure or internal effective stress. In order to simplify the conceptions the tests were carried out in a split ring oedometer, which provides data for one-dimensional analysis. Nevertheless, the findings were are valid for the three-dimensional case.

1.8 Organisation of the Thesis

A comprehensive review of the mechanical behaviour of expansive soil is given in ***chapter one***. The review covers the important features of expansive soil behaviour namely, microstructure, laboratory stress-strain behaviour and heave predictive models. An outline of the relevant stress state variables, the stress paths commonly investigated in the laboratory, precedes the review of test results and the heave-prediction models. The empirical nature of the heave-prediction models and current design methods is demonstrated in this chapter. Following is a summary of the literature review, from which the statement of the research problem and justification for the study become evident. The chapter concludes by outlining the structure of the thesis.

Presented in ***chapter two*** is a new conceptual framework for understanding expansive soils. the concept forms a basis for characterising expansive soils in terms of effective stress. The effective stress principle for expansive soils is presented as a hypothesis, which is validated later.

The important features of volumetric soil behaviour noted in chapter one form the basis of the laboratory test programme designed and presented in ***chapter three***. Modifications made to the test equipment to accommodate suction measurement are presented. There is a presentation of the method used to analyse the experimental data. The resistance concept is adopted as the method of analysis. Its merits over the popular classic theory, which uses logarithmic scales and void ratio are demonstrated in the context of consolidation settlement of fully saturated soils. The chapter concludes by outlining the diffusion concept, which is used to characterise water flow.

The benefits of adopting the resistance concept in the analysis become clear in ***chapters five and six***, where the experimental data is analysed. The analysis in chapter five seeks to validate the new concept, while the effective stress hypothesis is validated in chapter six. Chapter six concludes by defining a conceptual model for conveniently handling the internal effective stress in a manner that is amenable to constitutive modelling.

Chapter seven focuses on the analysis of the effective stress-strain behaviour in terms of the tangent modulus. It is shown that the change in the internal effective stress underlies the changes in soil structure and strain.

The link between the internal effective stress and soil water content is established in ***chapter eight***. The internal effective stress of the soil at the shrinkage limit is unique, thereby rationalising the shrinkage limit. It is shown that the link defines the intrinsic soil property. The intrinsic soil property is defined and rationalised. The chapter concludes by expressing the intrinsic soil property in terms of soil parameters, which are obtained from routine laboratory test results.

The application of the effective stress principle to expansive soils is demonstrated in ***Chapter nine.*** Following this is the formulation of the appropriate expressions of the effective stress principle and the determination of the respective input soil parameters. Thereafter, the effective stress model is used to simulate the changes in the internal stresses during soil wetting. Summary, conclusions and recommendations for further research are presented in ***chapter ten***.

Chapter 2

Development of a new concept for visualising particle-level phenomena

According to the literature study, the soil water potential (suction) concept was adopted to characterise the mechanical behaviour of unsaturated and expansive soils. However, the suction concept places emphasis on the water phase and not the solid particles in general and expansive soils in particular. As such, it indirectly addresses the soil mechanics of unsaturated soils. The concept is well suited for applications, where the movement of water is of primary concern. Maybe this explains why its origins are in the discipline of soil science and agricultural engineering (Buckingham, 1970). This probably explains the limited success in applying the effective stress principle to unsaturated soils, despite commendable efforts (Burland, 1965).

In view of this, a new concept was developed and is presented in this chapter. It places emphasis on the soil skeleton and thus facilitates a mechanistic visualisation of the particle level phenomena. The concept formed the basis of the hypotheses, which relates the soil potentials (particle-level forces) to effective stress. The hypothesis is also presented in this chapter. The starting point was a consideration of the major forces that exist in a clay-water-electrolyte system. The work presented in this chapter is original.

2.1 Clay-water-electrolyte system of an expansive soil

The major forces that act in a clay-water-electrolyte system are (Ghildyal and Tripathi, 1987):

- Gravity
- Hydrostatic forces associated with pressure gradients
- Osmotic forces related with solutions
- Adsorptive forces (long range electric forces and the short-range London Van der Waals' forces)
- Temperature forces

For simplicity, we assume isothermal conditions and that there are no osmotic forces due to solutions. In addition, Yong and Warkentin (1975) pointed that clay minerals fall in the class of colloids (molecular size of between 0.001mm and 10^{-6}mm). For colloids, the adsorptive forces are dominant at molecular level and the influence of gravitational forces is small. Consequently, the forces that are significant to a swelling soil are the adsorptive or surface forces.

There are two types of adsorptive forces or force fields in the system, namely the attractive and repulsive forces. Attractive forces can be broken down to (i) forces of adhesion or adhesive force and (ii) forces of cohesion or cohesive forces (Ghildyal and Tripathi, 1987). The maximum distance between two molecules at which the force of cohesion acts is called their ***molecular range***. The particle-level phenomena can be viewed as an interaction of potential energy or 'force' fields. For convenience, the adsorptive forces are herein referred to as ***'soil potentials'***.

2.2 Development of a new concept - The induction concept

The induction concept was conceived by the writer and is presented here for the first time. It builds on the characterisation of the soil potentials, a view that is also original.

2.2.1 Characterisation of soil potentials

The term “cohesion” has been used invariably to refer to the attraction between hydrated clay minerals. A hydrated clay mineral is composed of the clay mineral and adsorbed water. However, cohesion has not been characterised in terms of the constituent elements of the hydrated clay minerals. Specifically, there is no clarity on the nature and extent of the contribution of the solid clay minerals to cohesion.

It is proposed herein for the first time to view cohesion as a product of two components arising from the constituent elements of the hydrated clay minerals. That is, the hydrated clay mineral is decomposed to (i) the solid particles (clay minerals) and (ii) the adsorbed water layers.

The first component of cohesion is the attraction between the solid clay-mineral particles in their own respect. The attraction is considered an intrinsic property of the clay mineral. It underlies the hydrated clay-mineral attractions, in spite of the same polarity carried by the adsorbed water layers around the clay minerals. It is therefore a primary potential. It is herein called ***soil attraction***, in correspondence with Janbu’s (1973) definition. Janbu defined attraction as *the isotropic tensile strength of a Coulombian material.*

The second component arises from the adsorbed water layers. It is herein called ***soil cohesion*** in line with the conventional usage of the term. Now, the same polarity on the water layers would normally cause repulsion. Yet, the water layers stick together and give rise to soil plasticity. It is asserted that soil attraction facilitates the mobilisation of soil cohesion, and in turn mobilises potential repulsion by pulling the solid clay minerals together. As such, both soil cohesion and repulsion are secondary potential. The proposed relationship between the soil potentials is illustrated in fig. 2.1.

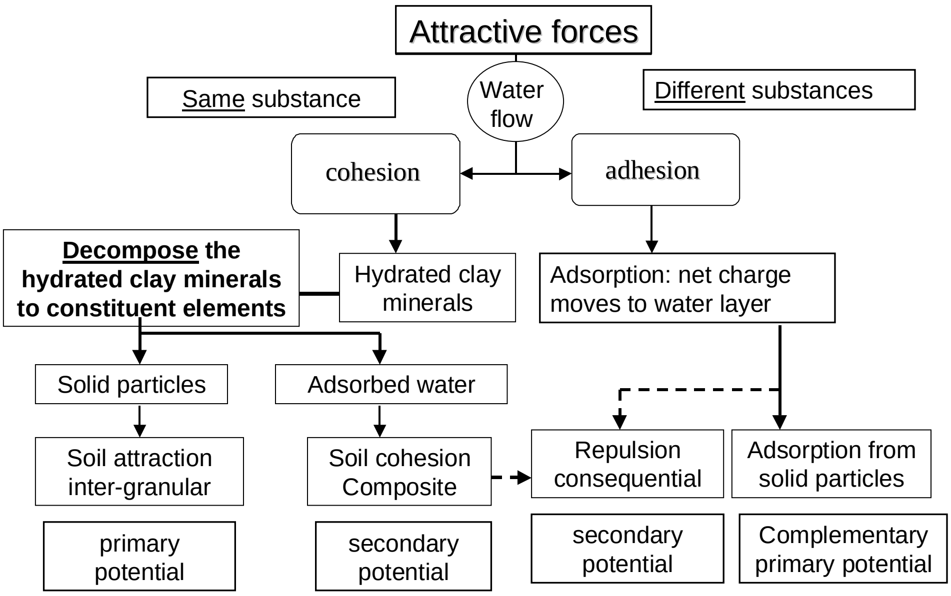

Figure 2.1 Characterisation of the soil potentials

From the above, a closer look at the origins of repulsion shows that it is a product of the attractive forces between the clay minerals. Adhesion attracts and leads to water adsorption, while cohesion brings the clay surfaces together, in spite of the potential repulsion. Consequently, repulsion is considered a secondary potential, being dependent on the potentials that cause hydration and bring the clay surfaces close, respectively. Similarly, soil cohesion is a product of the simultaneous change in soil attraction and soil adhesion. Accordingly, it is considered a secondary potential. Therefore, the soil attraction is the primary soil potential and it influences the particle level phenomenon.

2.2.2 Formulation of the induction concept

For an unsaturated expansive soil element, the soil potentials, soil water content and soil structure exist in unstable equilibrium, with respect to water flow. When water flows, it offsets the equilibrium and triggers a dynamic equilibrium, which is sustained until the soil starts to flow. The characteristic feature of the dynamic equilibrium is that it is physically admissible at specific water content points. The relationship between the soil potentials and water content is then reflected in the change in soil consistency. The dynamic equilibrium is a response to the change in soil attraction, which is an induction phenomenon. The induction phenomenon is illustrated in fig. 2.2 and discussed in below in detail. The water content points denoted as A', B, C, C', D and F are arbitrary. In view of their link to the soil structure, the arbitrary water content points have physical significance.

The induction phenomenon

Consider an unsaturated soil element with water content at point A. Water flow causes hydration, which causes the water-layers to thicken and thus increases the inter-solid particle distance. This increases the inter-solid particle distances. As such, both soil attraction and soil adhesion decrease. The effect of adhesion translates to the adsorbed water as a net charge and hence potential repulsion or swelling pressure. A significant water content is attained (point A'), when the inter-hydrated particle distance is within the molecular range for soil cohesion to mobilises. Accordingly, soil cohesion mobilises. In other words, the simultaneous decrease of soil attraction and soil adhesion (A-B) induces soil cohesion.

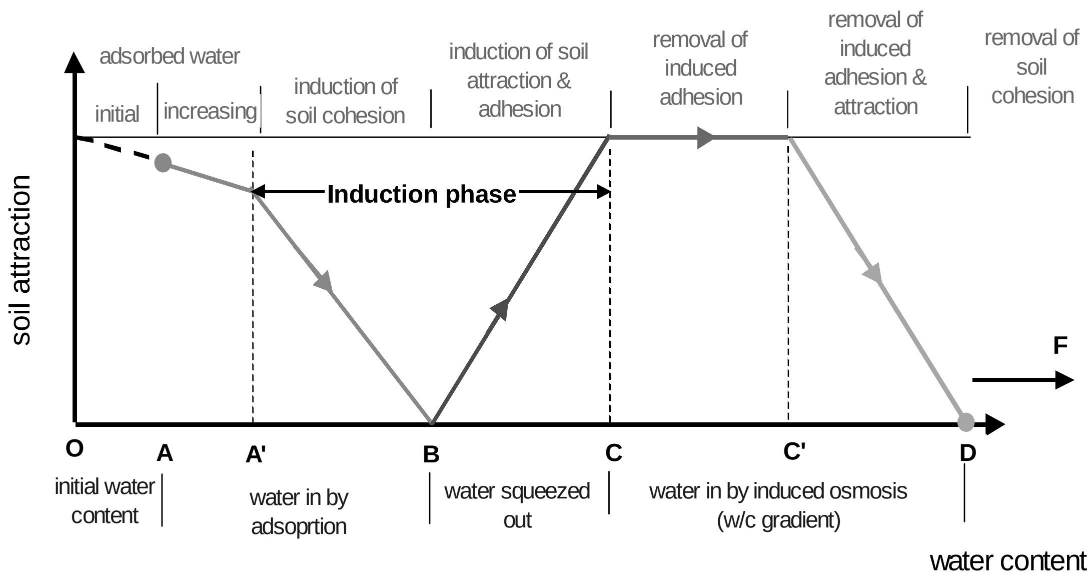

Figure 2.2 Illustration of the induction concept in terms of soil attraction

Point B is a critical water content point for two reasons. Firstly, both soil attraction and adhesion are exhausted and manifest externally as soil cohesion. As such, the soil element is internally saturated at point B. Secondly, soil cohesion is a maximum. It is proposed that the compression of maximum soil cohesion reduces the inter-clay mineral distance to within the molecular range of the solid clay-minerals. This induces the mobilisation of soil attraction.

At the same time, soil attraction mobilises and presses the adsorbed water layers and squeezes out the loosely bound water. This creates an ionic concentration (water content) gradient in the soil. It is also referred to as osmotic potential (Mitchell, 1993). Now, osmotic potential has the capacity to suck in water much the same as soil adhesion during hydration. Accordingly, osmotic potential is considered ***induced adhesion*** however, under saturated conditions. Like adhesion, it is in potential form and has a high affinity for water. Thus, the decrease of soil attraction and soil cohesion (A-B) induced soil cohesion, which in turn induces soil attraction (B-C), and the induced soil attraction simultaneously induces adhesion however, under saturated conditions. Thus, the phenomenon is an induction phenomenon, hence the term ***"induction concept***".

The soil condition at point C has high osmotic potential. Accordingly, the squeezed out water is assimilated back in the soil under an osmotic-gradient to restore water-content equilibrium (Mitchell, 1993). Inflow of water increases inter-particle distances and reduces soil attraction. Thus, internal limiting equilibrium in the soil is reached at point C', when the effect of the induced adhesion (osmotic potential) balances the remaining soil attraction. The diffusion of the squeezed out water does not lead to a change of the soil element in terms of structure or stiffness. This is because the water is immediately decomposed into the cations and anions, which are tightly packed in the diffuse double layer. Accordingly, the soil stiffness at point C', is equal to that at point C.

Point C' is an internal yield point beyond which soil attraction rapidly decreases. At point D, the soil element has zero soil attraction but maximum soil cohesion. The induced adhesion is likewise zero. Thereafter, removal of soil attraction would require forced water flow. The soil element eventually flows when the liquid limit is attained (point F).

2.2.3 Soil structure and the induction concept

The coupling between the soil potentials and soil structure is such that the change in soil structure during water flow reflects the change in soil attraction. This is illustrated in fig. 2.3 and described below.

An unsaturated expansive soil element (point A) has initial resistance due to soil attraction. When water flows in the soil element, the loss of soil attraction causes the soil element to soften and change its initial structure. Meanwhile, soil cohesion increases and keeps the clay minerals together. In addition, it causes the swelling particles to fill the void space and prevents water flow.

The condition of maximum soil cohesion (B) is of physical significance to the soil structure and water flow. According to the induction concept, cohesion pulls the hydrated clay minerals such that the soil is plastic. In addition, the void spaces are completely blocked such that water flow is by diffusion. Consequently, soil cohesion is directly related to the soil's resistance to flow of water. It seems that the induction of soil attraction at point B is an attempt to restore initial soil conditions, following the successful resistance of mechanical water flow, which tended to deform the soil element.

Soil attraction presses the adsorbed water layers together. Accordingly, the water layers are hard-pressed between the clay mineral surfaces. This phenomenon increases the soil stiffness and plasticity. Accordingly, the soil element mobilises the maximum possible stiffness, under saturated conditions, at point C. However, the soil stiffness that corresponds to point A is close to point C', when the soil element attains limiting equilibrium in terms of water flow, however under saturated conditions.

The subsequent change in soil structure is best considered in the context of the dynamic change in soil structure during soil wetting. This is discussed in section 2.3 below.

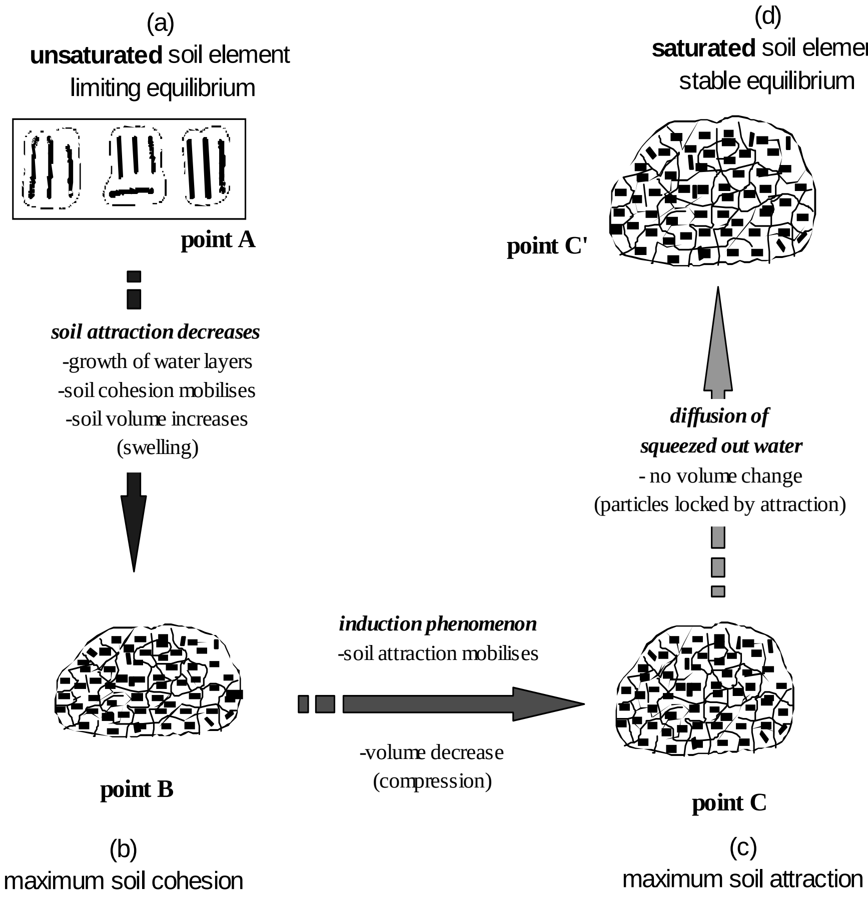

Figure 2.3 Illustration of the induction concept

2.3 A new model for the dynamic (changing) soil structure

A new model for the changing soil structure of a swelling soil was conceived by the writer and is presented in this section. In order to cover the full range of water content from the dry phase to the liquid phase, an initially dry soil with initial water content well below the shrinkage limit of the soil was considered. In addition, it was necessary to model the soil particles in the various water content ranges, where the soil structure is significantly changed. Thereafter, the induction concept was used in conjunction with the soil particle model, to model the changing soil structure during soil wetting.

2.3.1 The soil particles model

It is proposed that three distinct types of 'soil particles' adequately describe the structure of an unsaturated expansive soil, between the dry and the wet soil states. The first type is the *clay mineral*. It is the basic unit. Secondly, the clay minerals can stack up, forming *clay platelets*. A clay platelet is made up of two or more basic units. Lastly, the clay platelets can be in clusters forming what is called "*clay particles*". A 'clay particle' is an aggregation of clay platelets. The main features of the model are (1) the degree of bonding (mobilised soil attraction) within a particle, (2) the defined water content ranges over which the different soil particles are predominant and, (3) the mode of disintegration of the soil particles to a lower level particle. Thus, the presence of the three types of soil particle in a soil depends on the soil water-content. Fig. 2.4 illustrates the soil particles.

2.3.1.1 Feature 1: Particle bonding

Soil attraction underlies the degree of particle bonding and hence the formation of the different particles. For the clay minerals, the particles are electronegatively charged and can not freely exist without adsorbed water. As such, a film of water is retained around the particles. However, because the inter-clay mineral distance is small, it is invariably less than the molecular range of the clay minerals. Therefore, the clay minerals mobilise soil attraction and stick to one another, despite the presence of the surface charge. They form clay platelets. Therefore, the ***bonding with clay platelets is considered primary bonding***. It is considered primary bonding because it is between the basic units and the inter-particle distances are such that maximum soil attraction can be easily mobilised.

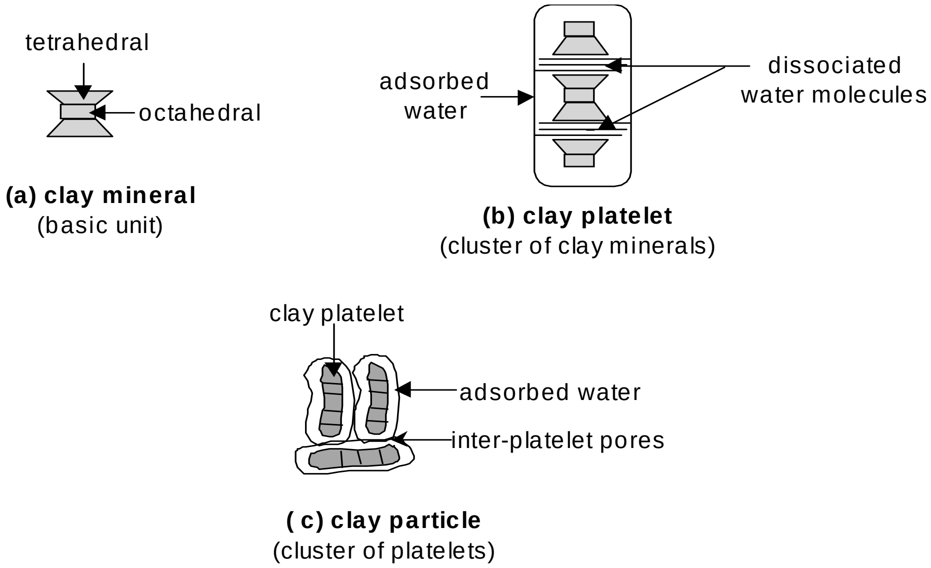

Figure 2.4 Illustration of the soil particle model

In general, the clay minerals stack and form clay platelets of different sizes. In addition, the stacking and alignment is considered random. Due to the random distribution of the clay platelets, it is possible that some clay platelets are sufficiently close to mobilised soil attraction between themselves. This leads to an aggregation of clay platelets and formation of ‘clay particles’. However, the bonding is relatively weaker than that within the clay platelets. Therefore, ***the bonding within the clay particles is considered secondary bonding***.

2.3.1.2 Feature 2: water content range

Since clay minerals exist in hydrated form, it follows that they are predominant in wet soils. For an initially wet soil, the development and clear formation of clay platelets increases with the drying of the soil. That is, soil drying shrinks the adsorbed water layers and thus increases soil attraction. It can be said then, that there exists upper limit water content, below which the clay platelets are well defined. It it is also the lower limit for clay minerals to exist as the predominant soil particles. As such, the soil should be sufficiently wet for clay minerals to be defined. This means that the soil has a continuous adsorbed water phase or is still plastic. Meanwhile, the inter-clay platelet distances get shorter as the soil dries. This is because the continuous water-phase link is weaker. It follows that there exists a water-content, below which attraction between closely positioned clay minerals causes them to cluster and become “granular” particles.

From the above, it seems to suggest that the water content limits implied here are the plastic and shrinkage limits respectively. That is, clay minerals may exist at water content above the plastic limit, clay platelets dominate the water content range between the plastic limit and the shrinkage limit, and the clay particles exist below the shrinkage limit. The validity of this proposition becomes clear in subsequent chapters.

2.3.1.3 Feature 3: Mode of particle disintegration

In view of the three types of soil particles, it follows that there are two levels, at which the change in soil structure is significant. Since the phenomenon involves destruction of the initial soil structure, it is herein called ***soil destructuration***.

Soil destructuration is defined as a condition, in which the bonding between previously joined constituent elements of the soil structure is progressively destroyed by shearing. The cause of the destruction can be internal or external. The first use of the term "destructuration", to mean the post-yield disruption of the natural structure of clay, was by Leroueil *et al*. (1979). The issue is discussed in detail by Burland (1990) and Leroueil and Vaughan (1990). Earlier, Janbu (1963) had reported the same phenomenon. The first suggestion of how destructuration might be incorporated within elasto-plastic constitutive models was by Gens and Nova (1993) and subsequent papers that develop these ideas include Rouainia and Muir Wood (2000) and Baudet and Stallebrass (2001). The term is adopted herein for the first time, concerning the change in soil structure due to a change of the internal stress of the soil. In this regard, mobilisation of soil cohesion constitutes a shearing process.

2.3.2 Formulation of the dynamic soil structure model

The structure of an unsaturated expansive soil undergoes at most two distinct levels of change before attaining internal saturation. The disintegration of the initial soil structure is called soil destructuration. The main features of the model are (1) soil destructuration and (2) removal of soil potentials. The physically admissible stages in the dynamic soil structure model are illustrated in fig. 2.5 and described as follows. The arbitrary water content points defined in the induction concept are shown.

2.3.2.1 Soil destructuration

Consider an initially unsaturated soil element at point A, well below the shrinkage limit. As such clay particles are dominant.

That is, the clay particles are held together by the mobilised soil attraction and are relatively spaced to be "independent" of each other. Hydration leads to the growth of adsorbed water layers. The subsequent mobilisation of soil cohesion shears the clay particles and reduces them to the constituent elements namely, clay platelets. Accordingly, the destruction of the initial soil structure to clay platelets is herein called ***1st-level soil destructuration***. The process is completed at water content point A'.

Now, 1st-level soil destructuration (point A') enhances the mobilisation of soil cohesion because it multiplies the elementary clay particles by division and further reduces the inter-particle distances of the resultant particles. Thus, continued hydration causes growth of the water layers. This increases the distances between the solid clay minerals within the clay platelets on the one hand. On the other hand, it increases soil cohesion, which then shears and hence disintegrates the clay platelets. Consequently, the disintegration of clay platelets to clay minerals is herein called ***2nd-level soil destructuration***. It is completed at point B, when soil cohesion is a maximum and soil attraction completely removed.

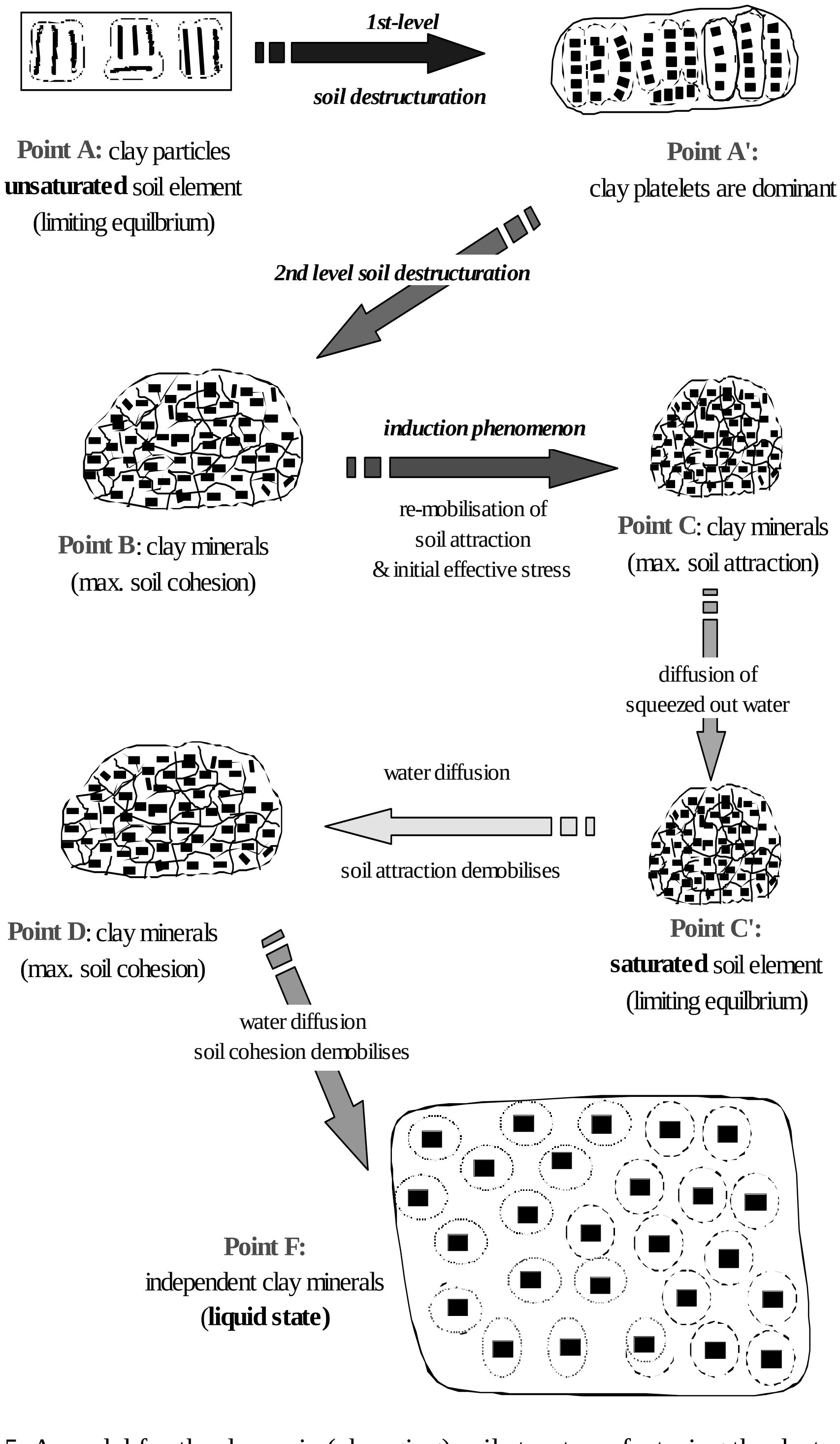

Figure 2.5 A model for the dynamic (changing) soil structure, featuring the destructuration and induction phenomena.

The subsequent induction phenomenon (B-C) does not change the composition of the soil particles. Clay minerals remain the dominant particles. However, the adsorbed water is pressed and squeezed by soil attraction. This increases the soil stiffness up to point C, where it is a maximum as discussed earlier.

2.3.2.2 Removal of the soil potentials

The diffusion of water in phase (C-C') leads to a simultaneous decrease of soil attraction and the induced adhesion (osmotic potential). The soil stiffness only changes after attainment of limiting equilibrium (point C'), which is an internal yield point. This is the case because until then, the diffusing water is not stored in liquid form, a consequence of the high osmotic potential. After point C', limited water exists in liquid form and occupies significant space. Sufficient volume of water comes between the clay minerals and increases the interparticle distance beyond the molecular range of the respective soil particles. This explains the relatively fast decrease in soil attraction in fig. 2.2 (C'-D).

Point D is characterised by the restoration of maximum soil cohesion. Since the effect of soil attraction is removed and induced osmotic potential neutralised, the hydrated clay minerals are no longer compressed. It is reasonable to say that internally the soil is under atmospheric conditions and the soil water exists in liquid form. Therefore, the soil is in stable equilibrium. Thus, ***point D is the atmospheric saturation water content***.

Beyond point D, forced water flow is required to remove soil cohesion. This is the case for two reasons. Firstly, there is induced osmotic potential to cause water diffusion. Secondly, the electrochemical bonding associated with the adsorbed water (soil cohesion) requires some energy to reverse it. The soil structure takes a new form at zero soil cohesion. Thus, another limiting water content is attained, above which the clay particles act independent of each other. Under conditions of continued water flow, the particles can flow. In other words, the soil has attained the liquid state and is denoted point F. Thus, the soil has destructured from the solid phase to the liquid phase. Therefore, the soil has six water content points, at which the change in soil structure is significant.

2.3.3 The physical significance of the arbitrary water content points

The physical significance of the water content points is summarised in table 2.1 below.

Table 2.1 Significance of the arbitrary water content points

water content point	***physical condition***	***physical significance***
A	‘granular’ particles	initial water content
A’	adsorbed water layers of adjacent particles start to interact	shrinkage limit
B	adsorbed water layers are a maximum (zero soil attraction)	plastic limit (internal/suction saturation point)
C	maximum soil attraction & swelling pressure	pressure saturation point
C’	limiting soil attraction	internal yield point
D	zero soil attraction and swelling pressure at atmospheric value	atmospheric saturation water content (external yield point)
F	zero cohesion	Liquid limit

2.4 Mechanistic view of the particle level phenomenon

Soil attraction and soil adhesion are characteristic of the clay minerals and are complementary. Soil attraction has an internal effect, while adhesion is external (it causes water adsorption). Soil attraction is intergranular and hence characteristic of the soil grains or clay minerals. Since soil attraction is intergranular, it gives rise to effective stress. That is, the clay minerals have a unique property that they can mobilise effective stress internally.

2.4.1 Effective stress hypothesis

The soil potentials can be expressed in terms of effective stress as follows.

(a) Soil attraction is the internal effective stress. It is intergranular and is capable of resisting unsaturated expansive soil deformation caused by a mechanical action.

(b) Soil adhesion is potential internal effective stress. Adhesion is only mobilised when the external action to be resisted is water (moisture) flow. In this case, it immobilises the water molecules leading to the mobilisation of soil cohesion.

(c) Cohesion is the mobilised internal effective stress. The mobilised form of internal effective stress that is capable of resisting deformation by water flow. Maximum soil cohesion reduces mechanical water flow to a diffusion process.

(d) For drier than shrinkage limit conditions, ***the initially adsorbed water film is the residual (mobilised) effective stress***. The effective stress associated with the adsorbed water is mobilised after maximum soil cohesion, hence residual. The rationality of the terminology is shown in section 7.3.1.2 of chapter seven.

(e) Swelling pressure (repulsion) is excess negative pore water pressure

2.4.2 Swelling pressure hypothesis

Swelling pressure is a water pressure and is therefore isotropic. By virtue of its coupling with the soil potentials, it reflects the change in soil attraction. As such, it is a measure of the internal effective stress.

2.4.3 Storage chamber hypothesis

The changing soil structure models a natural chamber that stores the mobilised internal effective stress. The chamber is eventually destroyed as the effective stress is removed.

The storage chamber hypothesis is conveniently handled in the form of a physical soil model. Thus, a physical soil model traces the development of a stress storage-chamber and fully accounts for the changes that take place in soil structure. Such a model was formulated and is presented in the next section.

2.4.4 Significance of the hypothesis

2.4.4.1 Effective stress

Soil attraction presses the clay minerals together, while soil cohesion offers resistance to shear deformation. Accordingly, the significance is that the hypothesis and induction concept enable decomposition of the internal effective stress of an unsaturated expansive soil into (1) an isotropic stress (soil attraction) and (2), shear stress (soil cohesion). The decomposition agrees with Janbu (1973).

2.4.4.2 The swelling process

The swelling process is considered a rearrangement of the internal effective stress of an expansive soil in order to effectively resist soil deformation by water (moisture) flow. After resisting the external action, the soil tries to return to its initial condition by the induction phenomenon.

2.5 Development of a physical soil model for an expansive soil

Consider an unsaturated expansive soil element, with initial condition drier than the shrinkage limit. The soil element is laterally confined and is wetted from the bottom. The water flow in the unsaturated soil can be defined by an advancing wetting front moving upwards. The sample can swell one-dimensionally in the vertical direction. Swelling pressure is measured in the horizontal direction.

2.5.1 Formulation of the model

An advancing wetting front of mechanical water flow in an unsaturated expansive soil element causes the wetted soil to adsorb water and swell, and it evolves into a Swelling Boundary Surface (SBS). The evolution of the SBS models the mobilisation of soil cohesion. Therefore, when fully developed, the SBS is plastic. The Swelling Boundary Surface models the storage chamber and is herein called the SBS chamber. Its main feature is to store the mobilised swelling pressure, which is a reflection of the mobilised internal effective stress. The induction of soil attraction strengthens or reinforces the SBS chamber. The accompanying volume increase is modelled as the motion of the SBS. It models the increase in storage capacity to match the mobilisation of the effective stress.

The removal of the effects of soil attraction and cohesion is simultaneous with the dissipation of swelling pressure. Accordingly, this is modelled as demolition of the defunct SBS storage facility, following the release of the mobilised effective stress. Thus, the development, motion and destruction of the Swelling Boundary Surface characterise the swelling phenomena at macroscopic. The elements of the SBS are discussed below, with reference to fig. 2.6.

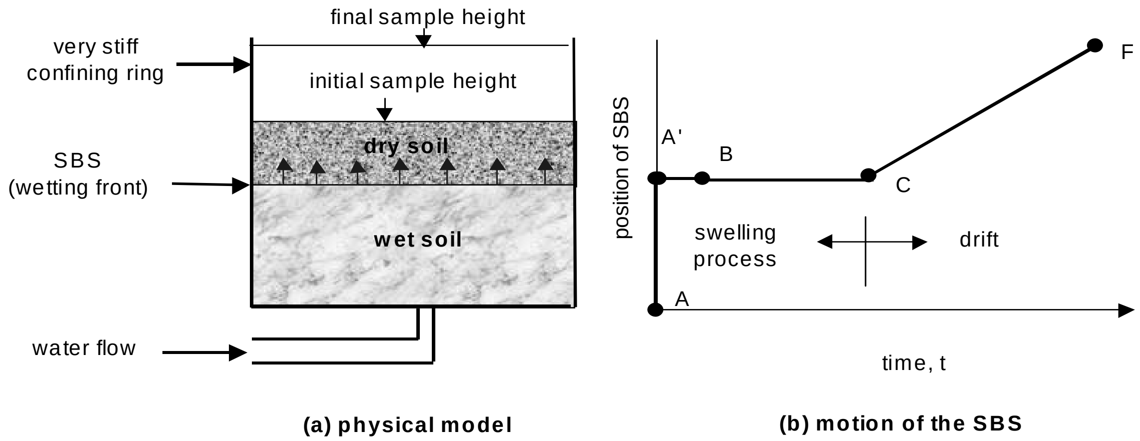

Figure 2.6 The development of the Swelling Boundary Surface (SBS)

The SBS divides the soil into two distinct soil types with respect to soil stiffness and mode of the swelling process. The soil above the boundary surface is effectively dry since it has no direct contact with water and is called “dry” soil. The soil within the SBS is in contact with water and is called “wet” soil (fig. 2.6a). The swelling process generally decreases in intensity with distance from the SBS, depending on the degree of soil confinement. The swelling process within the wet zone is completed at point C.

The development of the SBS is considered to take place relatively quickly, before the development of the repulsion-related swelling pressure (mobilisation of effective stress). In addition, it is associated with the mobilisation of soil attraction, which restrains swelling. Therefore, the development and motion of the SBS is largely not sensitive to the degree of soil confinement. Consequently, significant volume increase and the effect of degree of soil confinement come into play after the completion of the swelling process at point C.

2.5.1.1 Development of the SBS

As the water flows, the clay-particles within the wet zone of an advancing wetting front adsorb water and swell (A-A'). The increase in particle size is initially within the pore space and the change in soil structure is internal. The swelling particles start to reduce the pore space at point A' and increasingly retard the mechanical flow of water. In addition, they mobilise soil cohesion. Thus, the swelling process overtakes water flow at point A'. Consequently, the wetting front slows and transforms into a homogenous, impermeable, paste-like boundary surface, effectively sealing off the soil above it from that below it. Thus, the wetting front evolves and becomes a fully developed and stable boundary surface at point B. Though the sides and base of the soil element are confined, they similarly define the boundary of the wet surface. Consequently, the surface is three-dimensional and envelops the homogenous and paste-like wet soil. The boundary surface is herein called the ***"Swelling Boundary Surface" (SBS)*** because it is born out of the swelling of clay minerals. In accordance with the interaction soil model, the change in soil structure of the wet soil (A'-B), is a consequence of the mobilisation of soil cohesion. Therefore, the evolution of the SBS (A'-B) models the mobilisation of soil cohesion, while the re-mobilisation of soil attraction (B-C) is the mature phase of the SBS.

2.5.1.2 *The main feature - SBS storage chamber*

The SBS models the mobilisation of soil cohesion. According to the interaction model, repulsion is dependent on and lags behind the mobilisation of soil cohesion. Soil cohesion and soil-attraction pull the clay-particles together and thus fuel swelling pressure development. Soil attraction “locks in” the swelling pressure, causing it to remain as potential repulsion until after maximum soil attraction is mobilised. Accordingly, cohesion (internal and mobilised) acts as storage for the swelling pressure. The simultaneous mobilisation swelling pressure and development of the storage facility shows that the soil provides sufficient storage for the swelling pressure all the time, during the swelling process. Since the SBS models the mobilisation of cohesion, it follows that it models the storage of the mobilised effective stress.

2.5.1.3 *Motion of the SBS*

The motion of the SBS is with reference to fig. 2.6b. The interaction model reveals that the time associated with the swelling phase (A-A’) is very short because it involves mechanical water flow. In addition, the period (A’-B) is relatively short because of the link between 1st and 2nd-levels of soil destructuration. When compared with phase (B-C), which is diffusion controlled, the time for phase (A’-B) can be considered “instantaneously small”, thereby making points A’ and B one physical point. Accordingly, the development of the SBS takes place, while the wetting front is in one physical position. At the same time, the swelling process in phase (B-C) is controlled by the mechanism governing the flow of water. Flow of water is by diffusion, which makes the motion of the SBS painstakingly slow. In addition, soil attraction is induced and further reduces the diffusion process. From a macroscopic point of view, the Swelling Boundary Surface can be said to be stationary between points B and C. It is then reasonable to say that points B and C are the same physical position, making positions A’, B and C coincide or be at one level. As such, the Swelling Boundary Surface develops and remains stationary during the swelling process. The physical position of A’ is fixed by the initial steady position of the wetting front. Therefore, the swelling boundary surface develops and remains stationary, at the same position fixed by the water level following the mechanical flow in the initial phase (A-A'). The SBS is passive because it is stationary throughout the swelling process and does not interfere with the swelling process.

The post-swelling process (C-F) is simultaneous with the swelling process in the "dry soil" above the SBS. Accordingly, the motion involves three mechanisms. Firstly, the SBS stretches as water increases the volume of the wet soil. Secondly, the SBS eats into the 'dry' soil above, with the swelling process concentrated on a narrow band immediately above the SBS. The water diffuses and wets the "dry soil". Lastly, the SBS carries the 'dry' soil above as the volume of the wet soil increases. However, the load decreases with time, given that the SBS 'eats' into the dry soil above. Now, water flow is by diffusion being under a water-content gradient across the SBS. Consequently, the three mechanisms of the post-swelling motion are governed by water flow. Therefore, the Swelling Boundary Surface motion is a drift at a rate governed by the change in the water content gradient. Physically, the motion is tied to the increase in soil volume.

2.5.1.4 Demolition of the SBS chamber

The removal of the SBS models the progressive offsetting of soil attraction and subsequent removal of soil cohesion by continued water flow. Water flow forces water between the clay particles and eventually leads to significant volume increase. However, volume increase is very slow, being dependent on water flow by diffusion.

It can be said that volume increase is caused by the removal of swelling pressure, since water flows to neutralise the diffuse double layer. The increase in soil volume is equivalent to the amount of water required to neutralise the net charge and hence the swelling pressure. Thus, volume change is related to change in swelling pressure. Thus, the release of swelling pressure excess of atmospheric is responsible for the significant volume increase. However, the action of swelling pressure is passive in the sense that it is its removal, which causes volume increase. Accordingly, the SBS storage chamber is destroyed as the stored product is released. The process continues until the soil attains the liquid state (point F). Therefore, the demolition is considered a drift towards an equilibrium water-content condition (liquid limit).

Thus, the development of the storage chamber is concurrent with the build up of swelling pressure. Similarly, the removal of soil attraction and soil cohesion in the post-swelling stage is a removal of a progressively defunct SBS chamber.

2.5.1.5 Effect of initial soil water-content on the storage chamber

The development of the SBS is well defined for initial water content below point A’. Above point B, the soil structure is that of the SBS. The swelling-pressure and soil volume-change decrease with increase in initial water content.

2.6 Summary

The pertinent points from this chapter can be summarised as follows.

- By characterising the soil potentials, it was shown that soil attraction is the principal potential and is central to the swelling phenomena.
- The soil water potential concept is mainly suited for analysis of water movement in the soil. Accordingly, an alternative concept that is readily amenable to soil mechanics analysis was formulated and presented. It is considered rational because it focuses on the clay mineral solids and not the soil water.
- Three hypotheses were postulated: (i) The soil potentials can be expressed in effective stress terms. (ii) Swelling pressure is isotropic and is a measure of soil attraction, and (iii) the physical change in soil structure during swelling is such that the soil becomes a storage chamber for the mobilised internal effective stress. The chamber develops with the mobilisation of effective stress and self-destructs with the removal of effective stress.
- Swelling process is a re-arrangement of the internal effective stress to effectively resist deformation by water (moisture) flow.

Chapter 3

Laboratory equipment, testing programme and methods of analysis

This chapter details the laboratory test equipment and testing programme adopted in this research. This is followed by a description of the expansive soil that was investigated. The chapter concludes by outlining the methods used in analysing the test results.

3.1 Test equipment: The split ring oedometer

The split-ring oedometer is one test-equipment that can be easily adapted to the study of expansive soils. Though it was originally designed to study saturated soils, it has several features that are amenable to the study of swelling soils. Firstly, it is possible to measure horizontal pressures to 1000kPa with very good repeatability. Secondly, the equipment offers flexibility in mounting a soil-specimen with minimum disturbance. Thirdly, the initial horizontal contact pressure can be defined by adjusting the ring segments. Lastly, a gap between the ring and the specimen can be introduced to study the influence of lateral deformation. The last feature is particularly significant where the effects of cracks on swelling pressure and water flow are investigated. With modest modifications, it was possible to study the important features of expansive soil behaviour.

3.1.1 Description of the oedometer

The split-ring oedometer was designed and developed at the Department of Geotechnical Engineering of the Norwegian University of Science and Technology (NTNU), Trondheim. Senneset (1982, 1989) reported details of the equipment. The oedometer ring is made up of three equal segments, hence the name "split ring". The three parts of the ring can be simultaneously moved in and out by a precision lathe chuck. Each part has a 1mm-thick steel membrane lining on the inside, which overlaps on one side to cover the slits between adjacent ring parts when they are fully clamped in, giving a smooth finish on the inside of the ring. The inside diameter of the ring is perfectly circular at a diameter of D = 54.3mm (23.15cm^2 specimen area). The maximum height of a trimmed specimen that can be tested H = 20mm, giving H/D = 0.37. A detailed cross section is given in fig. 3.1(a), while fig. 3.1(b) shows the split ring oedometer.

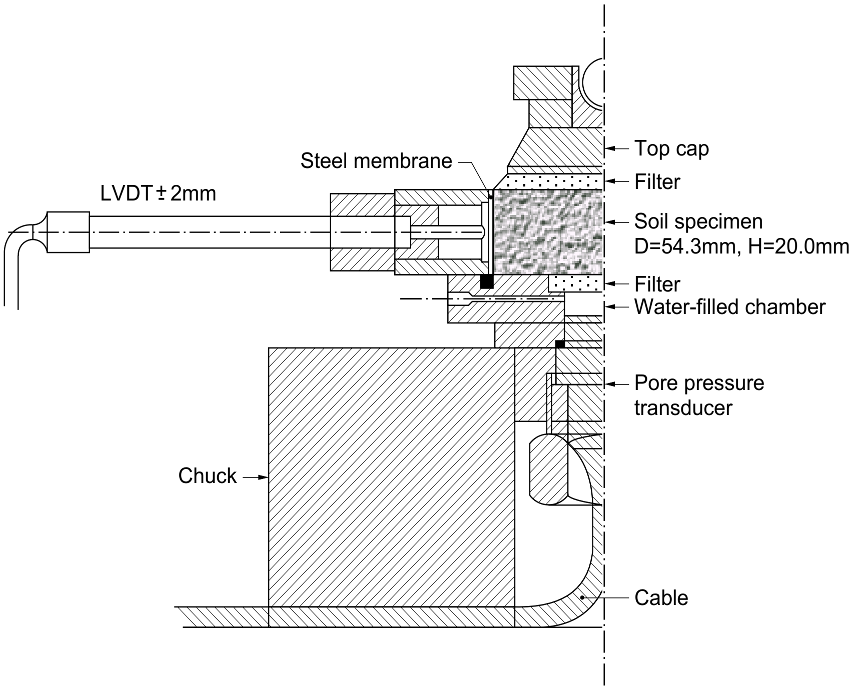

Figure 3.1 (a) Simplified cross-section of the split-ring oedometer (*after Senneset, 1989*)

The three parts of the split ring are very stiff, with the exception of the steel-membrane lining on the inside. The steel membranes' deflection in relation to the pressure exerted by a sample in the ring is measured by a Low Voltage Displacement Transducer (LVDT), from which the horizontal pressure is indirectly determined. The radial deformations measured outside the ring, as the overall system deformed, were approximately 8μm for a horizontal stress of 800kPa. This corresponds to a 16-μm increase in diameter. Thus, the measured eigendeformation of the split-ring oedometer was negligibly small. The ring was therefore, considered sufficiently stiff to ensure constant specimen volume for up to 800kPa horizontal pressure.

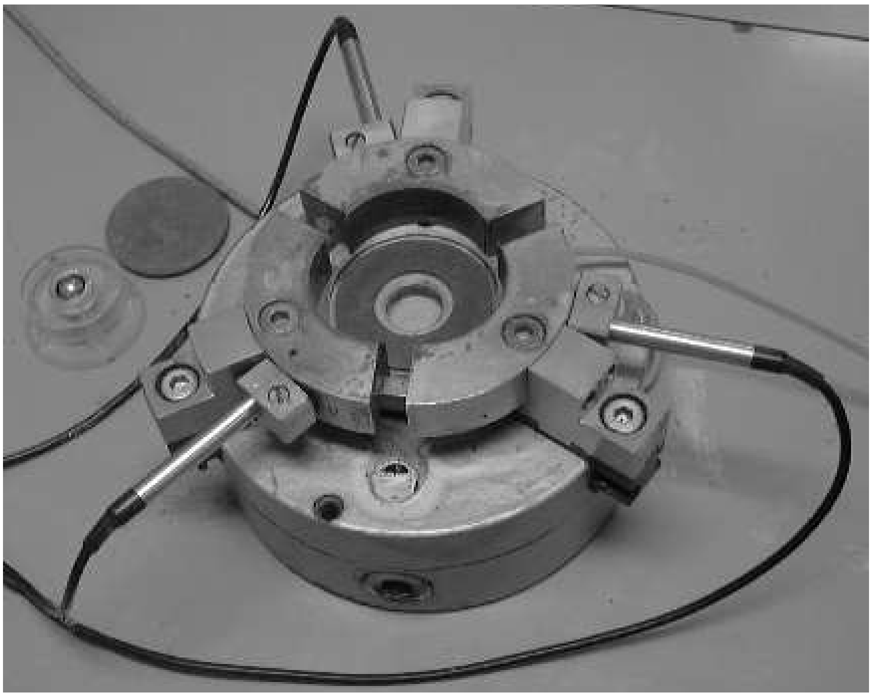

Figure 3.1 (b) The split-ring oedometer

By adjusting the ring segments inwards, it is possible to clamp a soil specimen of a given dimension with a controlled contact pressure. The largest sample "diameter" that can fit in the ring is 56mm, when the overlapping steel membranes are on edges. However, the sample would be oval-shaped.

3.1.1.1 LVDT specifications and calibration

The LVDT pressure transducers installed on the split ring to measure horizontal stress have a measuring range of 200 PSIG. The LVDT were calibrated by applying water pressure to the membranes with a dead weight pressure calibrator, to a pressure of 800kPa. The membrane deflections were measured against the increasing horizontal pressure and the relationship is plotted in Fig.3.2

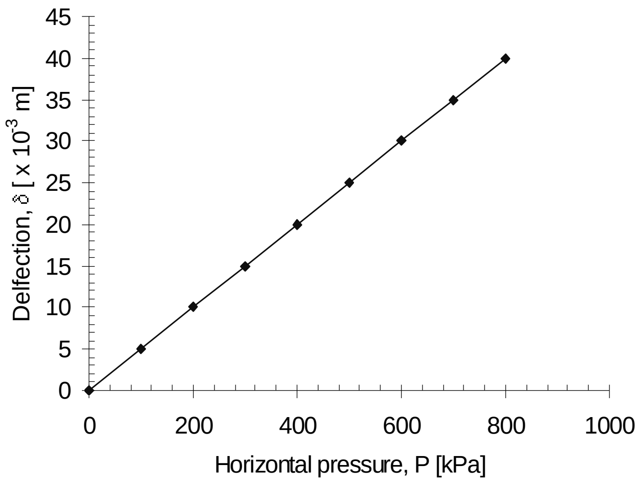

Figure 3.2 Membrane deflection with increase in horizontal pressure

A high degree of linearity and insignificant hysteresis between increasing and decreasing pressure were observed and are evident in the figure. An average lateral displacement of 38.4μm for the three membranes was measured for an applied vertical pressure of 800kPa. This corresponded to a volume change of the soil in contact with the membranes of 0.18 per cent, of the total sample volume. The deflections of the steel membranes were considered too small to affect the volume of a soil sample during a test. Therefore, the apparatus was considered stiff enough for constant volume tests with respect to horizontal displacement, up to a horizontal pressure of 800kPa.

At the base of the ring is attached a pore water pressure measuring transducer. It is the AB Model-type transducer from Data Instruments, with a measuring range of 1400kPa absolute pressure. However, its readings were not of significance given that pore water pressure in unsaturated soils is negative.

3.1.1.2 The vertical load and displacement transducers

The split ring oedometer was used in conjunction with either a continuous-load loading frame or an incremental-load loading frame, depending on the particular test as discussed in the test programme section below. The locking mode of the former frame was used to confine a sample in the vertical direction with a pre-defined or no vertical displacement. The latter on the other hand, was used in a different test together with a displacement measuring devise, to measure vertical displacement of a swelling soil sample.

The continuous-load loading frame has a load transducer with a vertical loading capacity of 10kN. In its locked mode, the eigendeformation of the loading ram and the whole set-up was negligibly small for the 10kN load. Therefore, the combined system was sufficiently stiff to handle constant volume tests with respect to both vertical and horizontal displacements. The incremental load-loading system does not experience significant vertical loads during the test, save those that simulate overburden or a superstructure. In this case, the samples freely swelled in the vertical direction. As such, the compliance of the apparatus is considered very low.

3.1.1.3 Data acquisition system and software

The Tesa System was used to convert the measured signals into physical quantities. The system provided a measuring range of +/-0.2mm, a gain error of 0.3 per cent at 20^{o} C. The zero drift per degree Celsius is 0.01 per cent. Repeatability and hysteresis error was 0.01mm, with a maximum measuring range of 100mm.

This system was connected to a 486 PC computer, through which the test was controlled. The tests were run and controlled using a specially designed computer software programme called Split Ring Programme Application Menu (SRPAM), developed in the department of Geotechnical Engineering, Norwegian University of Science and Technology, Trondheim NTNU. The programme has a windows interface to input initial test conditions and monitor the progress of the test on the screen.

One significant feature of the software is the flexibility in changing the logging time interval, even when a test is running. The smallest possible logging interval is five (5) seconds. The raw data is automatically stored in a data file, which can be read by most spreadsheets. Figure 3.3 shows the general set up of the apparatus, with the continuous-load loading frame being on the far right end.

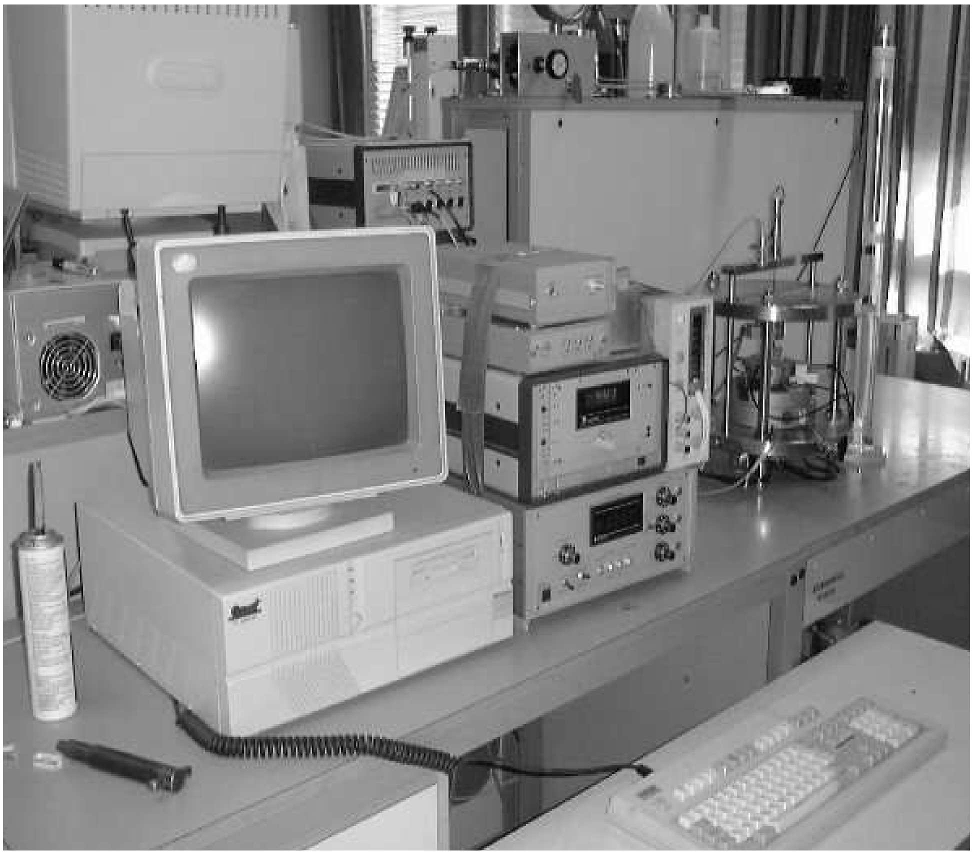

Figure 3.3 General layout: Split-ring oedometer equipment at NTNU

3.1.2 Modifications to the split-ring oedometer

The split ring oedometer was designed to measure the horizontal stress developed during compression test of saturated soils. Modifications were made to facilitate measurement of water flow during wetting or drying cycles and measurement of soil temperature during a test.

3.1.2.1 Water inflow (wetting cycle)

The outlet point at the base of the oedometer, normally used for drainage during consolidation tests, was used as the inlet for the water-supply system. The water-supply system consisted of an LVDT operated floating burette, with a 1mm-internal-diameter plastic pipe connecting to the drainage point. The burette was calibrated to enable measurement of water uptake. The calibration graph is presented in fig. 3.4.

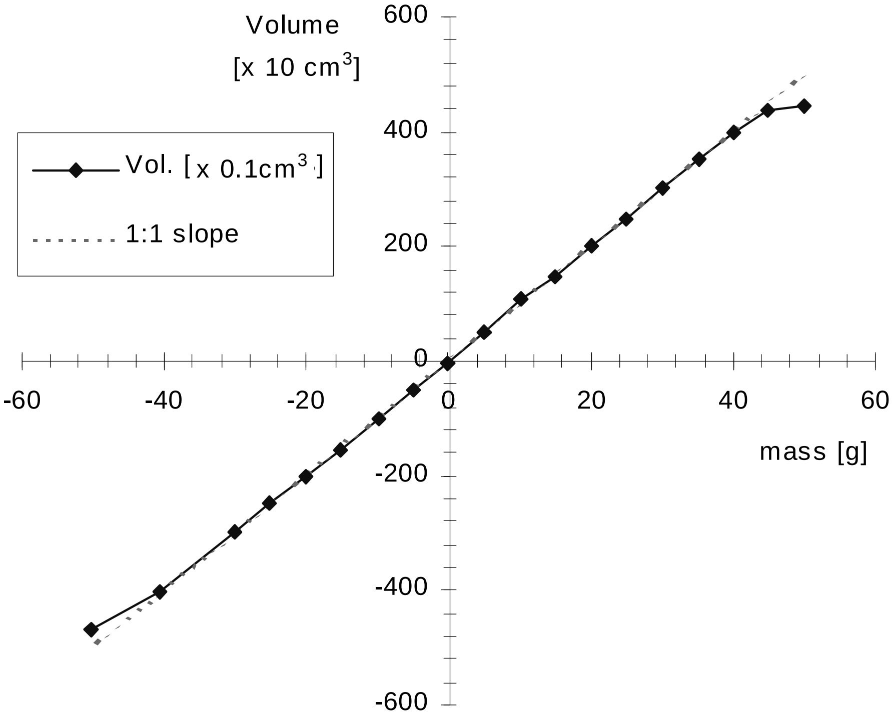

Figure 3.4 Calibration curve for the electronic burette

Water uptake could be measurement to accuracy of 0.1g. The connecting plastic-pipe had a stop-valve to regulate the amount of water flow to the soil sample as necessary. In addition, the original porous disk at the base covering 29 per cent of the sample base-area was replaced by one which covered 100 per cent of the sample base-area, to increase the contact area between the sample and the water. The coefficient of permeability of the coarse porous stone was 10^{-2} ms^{-1}.

3.1.2.2 Water outflow (drying cycle)

The other modification was to adapt the equipment to measurement of decrease in water content of the soil sample. Soil drying can be achieved by driving out the water from the sample using air-pressure. Therefore, a rigid steel cap was specifically designed and fabricated, to tightly fit the top end of the oedometer ring. When clamped together, the oedometer chamber enclosing the sample become airtight and is capable of withstanding 600kPa of air-pressure. The steel cap had an inlet point to allow air pressure. In addition, it had a built-in loading cap, which required 10kPa-pressure to overcome friction and make contact with the top end of the sample. The set-up could then be connected to a regulated air-pressure supply system.

3.1.2.3 Soil temperature measurements

A self-contained FLUKE-54 series type thermometer was used to measure the temperature of the soil sample and its immediate surroundings. The thermometer has two sensors and is capable of simultaneously taking two temperature readings. It can store 500 sets of temperature data. Logging time intervals are built-in, but can also be user-defined. In this arrangement one temperature sensor T1, was embedded in the bottom third of the sample to measure soil temperature. The other sensor was placed against the top cap of the oedometer set up to measure the outside temperature, i.e. the temperature immediately outside the oedometer ring. A kaylite box cover was used to cover the whole apparatus to reduce fluctuation of the outside temperature.

3.1.3 Reliability of the equipment

The split-ring oedometer has been used in fundamental research work on saturated soils at NTNU, since 1984. Senneset and Janbu (1994) successfully used the split ring oedometer to study the influence of initial lateral contact pressure on the preconsolidation stress, and the effective stress ratio on fully saturated soils. The results were in remarkable agreement with theory (Brooker and Ireland, 1965; Janbu, 1985). This gives credibility to the results obtained by this apparatus.

3.2 Laboratory test programme

The test programme was designed to provide sufficient information to characterise the expansive soil. The features of the expansive soil behaviour that were studied are swelling pressure and its mobilisation, axial strains, water flow and the effects of temperature. These features were studied under different wetting and confining conditions to simulate different in situ field conditions. The test programme recognised the need to closely simulate soil behaviour under field conditions.

The test programme was carried out in the Department of Geotechnical Engineering, Norwegian University of Science and Technology, Trondheim (NTNU), where the suitable apparatus is found. However, routine tests and determination of the soil-moisture characteristic curves were carried out in the Departments of Civil Engineering and Soil Science and Agricultural Engineering, University of Zimbabwe, respectively.

3.2.1 Test programme

Following a preliminary study of the versatility of the equipment and the possible soil response, it was concluded that an appropriate test programme be designed around three factors that affect soil swelling. The factors are (a) degree of soil confinement, (b) initial soil water content and

(c) method of wetting the soil or wetting condition. The degree of soil confinement was used as the basis of all the test series. Two test series were identified to simulate completely confined and completely unconfined (free swelling) soil conditions.

The tests were carried out on undisturbed expansive soil samples. The samples were sampled and tested in the vertical direction, at different initial water content. The range of initial water content covered was from eight to 42 per cent, the atmospheric saturation water content. Three different methods of soil wetting were used for each initial water content condition. The three wetting conditions that were adopted are (1) wetting under a fixed pressure gradient, (2) wetting under a suction gradient and (3) wetting by flooding. Pressure gradient is the sum of hydrostatic pressure and suction gradients. In all cases, the samples were wetted upwards. The effect of gravity was considered insignificant given the small maximum sample height of 20mm. Table 3.1 summarises the test programme. The different tests and testing procedures are described in the following sub-sections sections.

Table 3.1 Summary of the test programme

Main goals	***Description of test***	***Loading frame used***	***Test series***	***No of tests***
To study: - development & nature of swelling pressure - nature & character of the swelling process - confined water flow	confined swelling test	continuous-load	SP-series	27
To study - internal stress-strain behaviour - effect of swelling strain on swelling pressure & swelling process - unconfined water flow - relationship between internal stress & water content	free swelling test	incremental-load	FS-series	15

3.2.1.1 General testing procedure

In all tests, an undisturbed soil sample at given initial water content was used. Its initial mass and dimensions were measured. Loctite Silicone Sealant was generously applied to the ends of the ring segments. The sample was then placed on a porous disc at the oedometer base, while the three ring parts were sufficiently separated to avoid sample disturbance (fig. 3.5).

The ring segments were then adjusted towards the sample until they were just in contact with the sample. Simultaneously, the slits between the three ring parts were sealed off, ensuring a water- and pressure-tight chamber. A 1mm-diameter hole was drilled through the soil sample to let in a temperature sensor, T1. A coarse porous stone was placed on top of the sample before placing the loading top cap. The porous stone and top cap had a similar sized hole to let through the sensor cable. The sensor was embedded in the bottom third of the sample to measure soil temperature during the test. The other temperature sensor T2 was placed against the top cap to measure the temperature immediately outside the ring (outside temperature). The set-up was then connected to the water supply system and placed either in the continuous-load loading frame or incremental-load loading apparatus, depending on the particular test as detailed in the test programme in table 3.1. A kaylite box to reduce fluctuations of the outside temperature covered the whole set up. The water inlet valve and the SRPAM programme were simultaneously turned on to start the test.

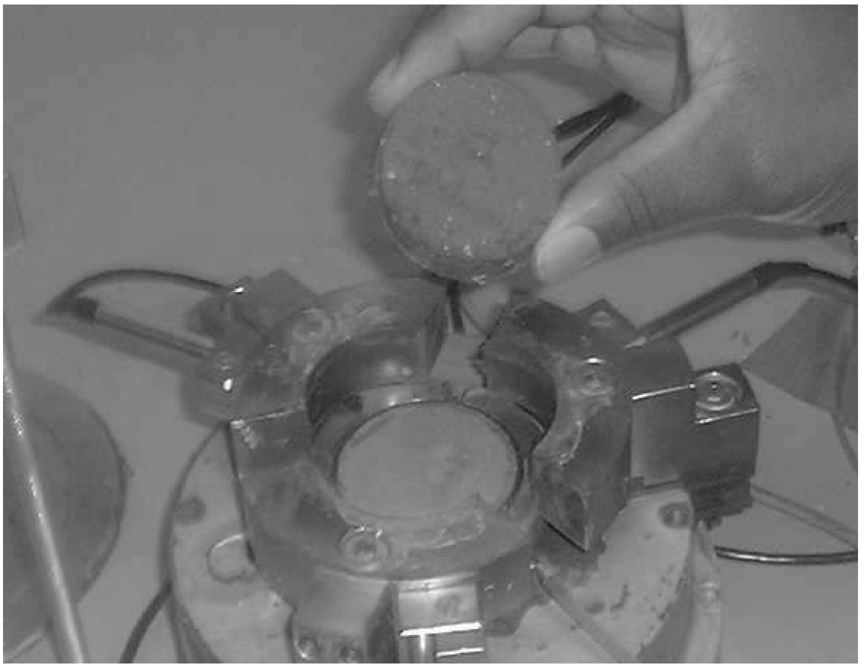

Figure 3.5 Placing of the soil sample in the split-ring oedometer

The SRPAM programme was programmed to log data every five or ten seconds for the first five minutes of the test, and changed to every 120 seconds thereafter. Each test was run for at least 72 hours or until there was no discernible change in swelling pressure, whichever was longer. At the end of each test, the logger and water inlet valve were switched off and the apparatus disassembled. The sample was carefully retrieved from the ring and had its wet mass and dimensions quickly taken before oven drying, for water content determination. The data was loaded onto a disc for subsequent analysis. The different wetting conditions used in each test series are as follows:

Wetting under a pressure gradient. The water level in the burette was kept at the same level as the top end of the oedometer ring. Any intermediate level could have been used. This level was adopted because it fixed a practical reference in terms of establishing the influence of the driving head on water flow. This condition simulated a steady rise in water table.

Wetting under a suction gradient. This wetting condition involves keeping the water level in the burette at the same level as the bottom of the sample so that only the bottom of the sample is in contact with the water. Water uptake is a result of suction only. (The effect of gravity was considered marginal given the small sample height). The condition simulates the field condition when moisture migrates under capillary forces. This can also closely relate to horizontal flow of moisture under a suction gradient.

Wetting by flooding. The wetting condition was achieved by rapidly forcing water in the sample through the base until it came up through the top of the sample. Additional water was introduced at the top of the sample to keep the top surface fully covered with water. In this procedure, it was not possible to measure water uptake with time. Therefore, only the initial and final water content of the soil was known. This condition simulated flooding or a rapid rise in water table.

3.2.2 Confined swelling test

3.2.2.1 Rationale

Confined swelling test involves swelling the soil under complete confinement. Keeping the volume of soil constant reduces the variables and hence simplifies the analysis of the swelling process. For instance, it becomes possible to study the influence of swelling pressure on the swelling process and flow of water in the soil independent of volume change. At the same time, results from tests at different initial water content can be used to study the swelling pressure-volume change relationship, in a similar way to incrementally loaded oedometer compression test on a fully saturated soil. In this case, the change in initial water content is analogous to the load increments. Lastly, the test brings out the nature of the swelling process in two respects. Firstly, the coupling between soil response to water flow and the swelling pressure and secondly, the mobilisation of swelling-pressure in the vertical and horizontal directions.

Complete confinement is one limiting case of expansive soil behaviour, in which the soil experiences no volume change. Absence of volume change means that the maximum possible swelling pressure can be mobilised. By prefixing the maximum vertical or horizontal strain, it is also possible to relate residual swelling pressure to the measured swelling strain.

3.2.2.2 Test procedure

The oedometer set-up, with the sample horizontally confined was placed in the continuous-load-loading frame (fig. 3.6). The frame was moved upwards until it was just in contact with the top cap. By monitoring the vertical load and displacement readings on the computer screen, the contact between the loading frame and the sample was refined until there was no vertical movement and yet not imposing a vertical load on the sample. The frame was then "locked" in this position, thereby completely confining the sample in both the vertical and horizontal directions. Water was added to the sample through the base as per the different wetting conditions discussed above. The raw data captured in this test was water uptake (cm^3), vertical load (N), horizontal swelling pressure (kPa), soil and outside temperatures (^oC) and time (seconds). There was no change in the vertical displacement since the sample was confined. Mawire and Senneset (1999) reported results obtained by adopting the test procedure.

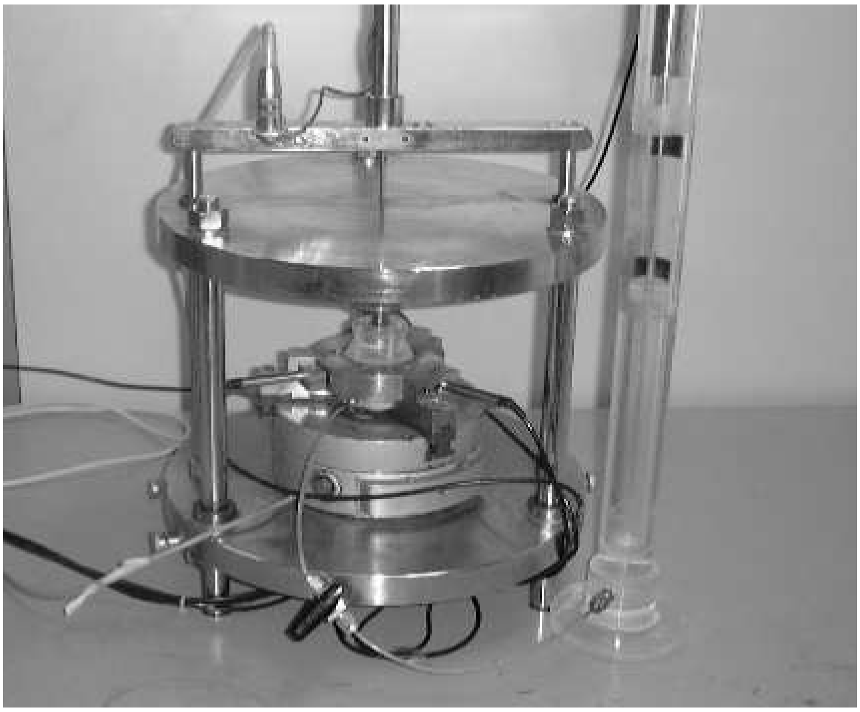

Figure 3.6 Oedometer set up for confined swelling tests

3.2.3 One-dimensional free-swelling test

3.2.3.1 Rationale

Free swelling test presents another limiting case, when the soil is only confined in the horizontal direction. It attains the maximum possible one-dimensional volume increase in the vertical direction. Just as confined swelling test is analogous to step wise loaded oedometer compression test, free swelling test is equivalent to continuous-load oedometer compression test. It offers the possibility to understand the relationship between swelling strains and the mobilised horizontal pressure, during and as result of water flow. It also simulates the soil behaviour close to the surface, where the overburden is a minimum. By introducing gaps in the horizontal direction, it is possible to simulate the effects of cracks.

Apart from this, the test offers insight to water flow under unconfined soil conditions and hence the influence of soil confinement on water-flow. It provides the upper limit of soil swelling with maximum volume change. Furthermore, the test offers the soil straining response during swelling, without the interference of external loads. Any subsequent loading would then be studied in the light of this response.

3.2.3.2 Test procedure

The oedometer set-up, with the sample horizontally confined was placed on an incremental-load loading apparatus. The vertical displacement-measuring device was screwed onto the oedometer ring. By monitoring the vertical displacement reading on the computer screen, the contact between the loading frame and the sample was refined until it was firm but not imposing a load on the sample. The sample was thus confined in the horizontal direction only, but free to swell vertically (fig. 3.7). With no vertical load applied the test was started, with water-uptake being through the base as per the wetting conditions discussed above. The raw data captured in this test was water uptake (cm^3), vertical displacement (mm), horizontal swelling pressure (kPa), soil and outside temperatures (^oC) and time (seconds). There was no record of the vertical load since the sample was not vertically loaded.

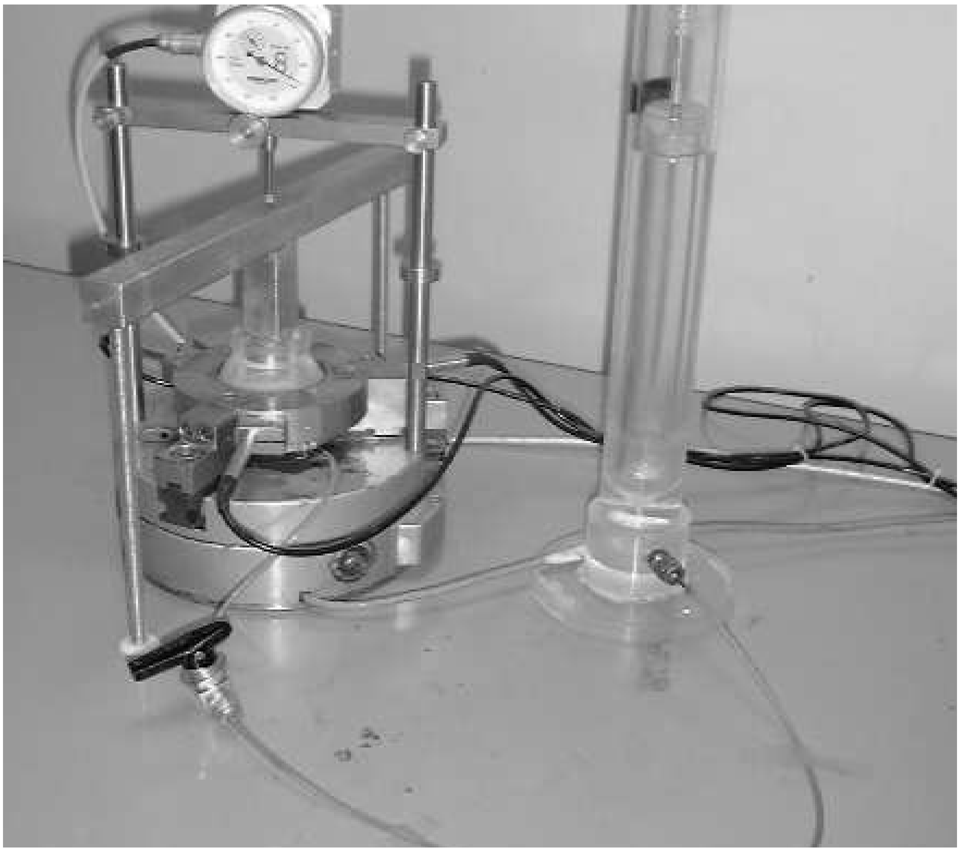

Figure 3.7 Oedometer set up for free swelling tests

3.3 Description of the expansive soil investigated

The expansive soil investigated was taken from a site within the University of Zimbabwe campus in Harare. The site was chosen for several reasons. Firstly, the soil on this site is representative of the most notorious soils in Zimbabwe, upon which massive low-cost housing development schemes are in progress. Thus, the findings from this research have immediate practical relevance. Secondly, the site is secure. Security was important because an instrumented field experimental structure, which forms part of the broad study of expansive soils in the department, had to be located in a secure place. Data from the field experiment would be used to realistically validate soil and interaction models, as necessary. Lastly, the site is located within 2km of the Department of Civil Engineering building, which is conveniently accessible for continuous study of the soil and long-term monitoring of the field experiment.

The soil samples tested were taken from a depth of 0.9 to 1.0 metres. This depth was considered within the zone of seasonal moisture fluctuation and influence of shallow foundations such as rafts. The field structure had a raft foundation.

3.3.1 Field description

All tests were conducted on an expansive clayey soil deposit called Avon clay. Avon clay is located along the Avondale stream, site within the University of Zimbabwe campus in Harare.

3.3.1.1 Geology of the site

It consists of approximately a 2m-thick layer of decomposed residual phyllite, overlying a dense to very dense, gravely parent rock.

3.3.1.2 Water table

The in-situ water content of the soil fluctuates with the seasonal water table, which varies between ground level in the rainy season and 1.5m-depth in the dry season. Typical in situ water content profiles during wet and dry seasons are illustrated in fig. 3.8. Surface cracks of the soil up to 100mm wide and 500mm deep are common in the dry season.

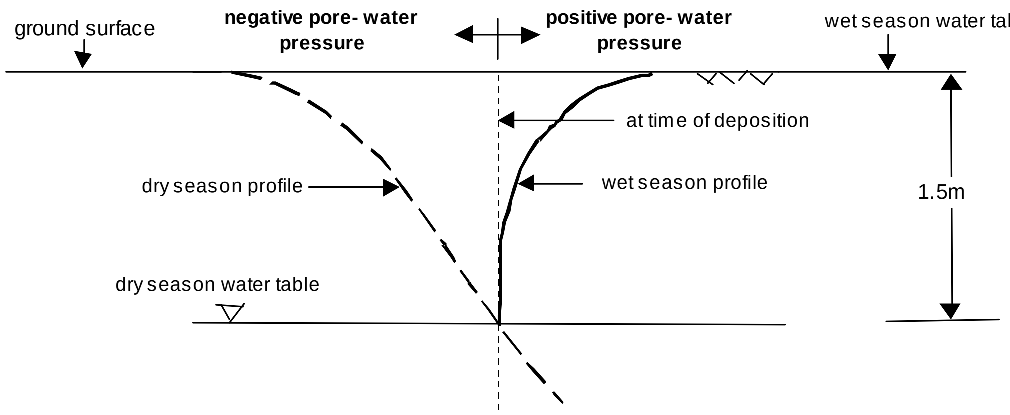

Figure 3.8 Seasonal variation of in situ water content profile for Avon clay

3.3.2 Sampling and transportation

3.3.2.1 Sampling

Undisturbed samples were block-sampled in accordance with Standards Association of Zimbabwe procedures, SAZS No 185: Part 2: (1977). The samples were taken from the sides and bottom of trial pits between 0.9 and 1.0m-depth. Specially designed PVC sampling tubes of 80mm-diameter and 60mm-height, with a sharp cutting edge, where gently pushed into the block samples. At the time of sampling, the soil was sufficiently wet to manually push the sampling tubes in the soil. In addition, the mobilised soil cohesion was such that minimum soil disturbance occurred. The ends of the samples were trimmed with a sharp edged spatula and immediately sealed with wax. The samples were wrapped in plastic bags and packed in 300mm x 400mm x 200mm-deep boxes, lined with thick cotton to cushion the samples from mechanical disturbance during transportation.

3.3.2.2 Transportation

The packed boxes were transported to Norway by DHL, an overnight express delivery service, for subsequent testing. Sample disturbance during transportation was considered insignificant for two reasons. Firstly, the samples progressively dried out during transportation and hence increased in stiffness. Secondly, the dimensions of the samples were much bigger than the size of the tested sample-size. A lot of the outer material was removed during sample preparation. Therefore, if any significant disturbance took place during transportation, it affected the peripheral soil, which was subsequently removed. Possible soil drying during transportation was actually a positive development. On arrival, the samples were further dried to various initial water contents before testing. Sample drying during transportation was therefore considered immaterial. However, few samples were taken at a time to avoid long periods of storage in the laboratory and hence excessive drying.

3.3.3 Laboratory sample preparation

3.3.3.1 Initial water content

The samples were removed from the packaging boxes and carefully extruded from the tubes. Two 20 to 30mm-high samples were obtained from each sampling tube. Each sample was weighed to determine the approximate water content. Care was taken to ensure that the sample retained the same orientation as existed in situ. The samples were then placed in a 100mm-diameter by 50mm-high plastic containers without lids, and left at laboratory temperature to slowly dry to lower water contents. The samples were periodically re-weighed to check the approximate water content. Drying at laboratory temperature proved to be sufficiently slow to avoid undue sample cracking. The exact initial water content was determined at the end of the test.

3.3.3.2 Trimming the samples

When the water content of a given soil sample was close to a specified initial water content, the sample was trimmed to a diameter of 54mm using a specially designed cutting ring, so as to tightly fit in the oedometer ring. The sample was trimmed to a height of between 18 and 19mm. A shorter than maximum possible specimen height (20mm) meant that the top loading cap would be within the ring walls and thus improve the pressure-tightness at the top. The sample was weighed accurate to

0.01grams and the dimensions taken to an accuracy of 0.05mm using a vernier calliper. An average of at least six readings was used as the sample height. The sample mass was measured just before the sample was placed in the oedometer ring, to reduce the possible change of sample mass due to evaporation losses. The sample was then subjected to different test conditions as discussed above. The specimen mass, diameter and height were also measured after the test and after oven drying, in order to determine the density, water content, void ratio and degree of saturation at the various stages of the test.

3.4 Methods of analysis

In soil mechanics, the two methods available for analysing consolidation test result for saturated soils are the classical approach and the resistance concept. In this work, the resistance concept was adopted because it is readily adaptable to unsaturated soil conditions without making any assumptions. As such, the concept is outlined in this section and its merits over the classical method are given. Since it is applied herein for the first time to unsaturated soils, new terms were introduced and are appropriately defined and explained. In the case of water flow, the diffusion concept was adopted.

3.4.1 The resistance concept

The resistance concept was the approach adopted for analysing the test results. A brief historical background of the concept is presented followed by the definition. The concept is first presented in the context of saturated soils, before extension to unsaturated expansive soils. The merits of the concept over the classical approach are highlighted. The resistance concept is used herein for the first time with modifications, to study unsaturated expansive soil behaviour. Specifically, new moduli are introduced in correspondence with the variables in a swelling soil.

3.4.1.1 Historical background

Janbu (1998) reported that Terzaghi's pioneering work on stress-strain behaviour in soil compression was based on the resistance concept. By closely reviewing Terzaghi's Hauptstiick III in Erdbaumechnik (1925), Janbu noted that Terzaghi used a soil modulus of elasticity E, defined by $E = \frac{dp}{de}$, where p = intergranular pressure and e = void ratio. Terzaghi called it ".... a fact of fundamental importance." On analysis of several tests using the resistance concept, Terzaghi obtained linear stress-strain relationship. He then concluded that the stress strain behaviour of soils was almost as simple as that for solid granular bodies. Janbu (1998) further pointed that Cassagrande (1932) used the resistance concept to accurately predict the preconsolidation pressure of undisturbed Laurentian clay.

The concept was revisited by Janbu (1963, 1985) and fully developed into a unifying framework in soil mechanics. He established that for engineering purposes, one could adequately cover the variations in compressibility of different types of geological materials from rock to soft clay, by means of one relatively simple resistance-based formula. The time effects are similarly characterised by a simple time-resistance formula. The resulting formulae for predicting stress- and time-strain behaviour can be expressed in terms of simple dimensionless numbers with well-defined mechanical meaning. The parameters are easy to explore by laboratory tests. The concept is the basis of geotechnical engineering in Norway for more than 30 years, including offshore geotechnics (Janbu, 1963).

3.4.1.2 Definition of the resistance concept

The resistance of a medium or part of it, to a forced change of an existing equilibrium condition, can be determined by measuring the incremental response to a given incremental cause.

$$Resistance = \frac{Incremental\ cause\ (given)}{Incremental\ effect\ (measured)} \qquad \text{(Equation 3.1)}$$

Thus, the concept relates the action on a body or medium to its response to the action. It is rationally defined in familiar engineering and mathematical language. It has been widely used in other fields of engineering, where action-reaction systems require analysis. Examples of its usage in some fields of engineering are electrical resistance (R or r), elastic resistance (E), dynamic resistance (mass), hydraulic resistance (k^{-1}), and heat resistance (C). The concept remains valid even for non-linear processes without a change in the definition. For most soils the response is non-linear and is generally defined as the tangent to the action-response curve. Figure 3.9 illustrates the definition of the resistance concept.

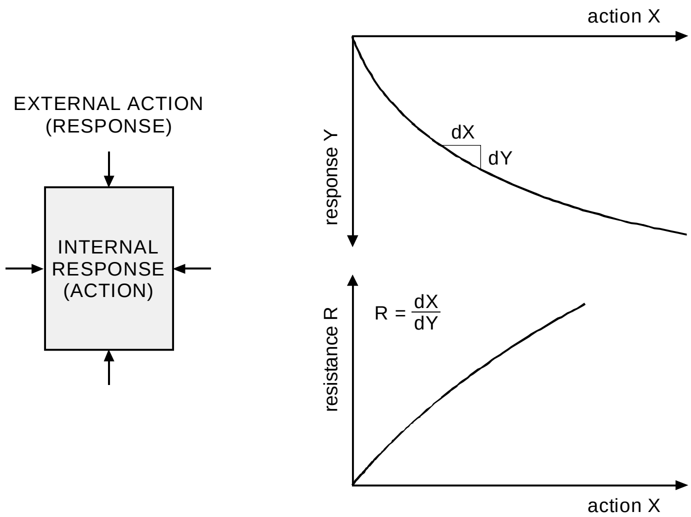

Figure 3.9 Definition of the resistance of a material (after Janbu, 1995, 1998)

3.4.1.3 Application to saturated soils

During consolidation of fully saturated clayey soils, the strain ε, which develops over an elapsed time *t*, after application of an effective stress change σ', is a function of both stress and time. The strain behaviour can best be studied by separate analysis of the soil resistance with respect to stress and time. The two resistance moduli that have been established are the Tangent modulus, M and the time resistance modulus, R respectively (Janbu, 1963).

<u>Tangent Modulus, M</u>

A typical stress-strain curve for a complete oedometer test is illustrated in fig. 3.10(a). By definition the tangent modulus is

$$\text{Tangent modulus, } M = \frac{\text{change in effective stress (given)}}{\text{change in strain (measured)}} = \frac{d}{d\varepsilon}\sigma' \qquad \text{(Equation 3.2)}$$

The tangent modulus M is plotted against increasing effective stress in fig. 3.10(b). The change in the effective stress is dramatic at the preconsolidation pressure.

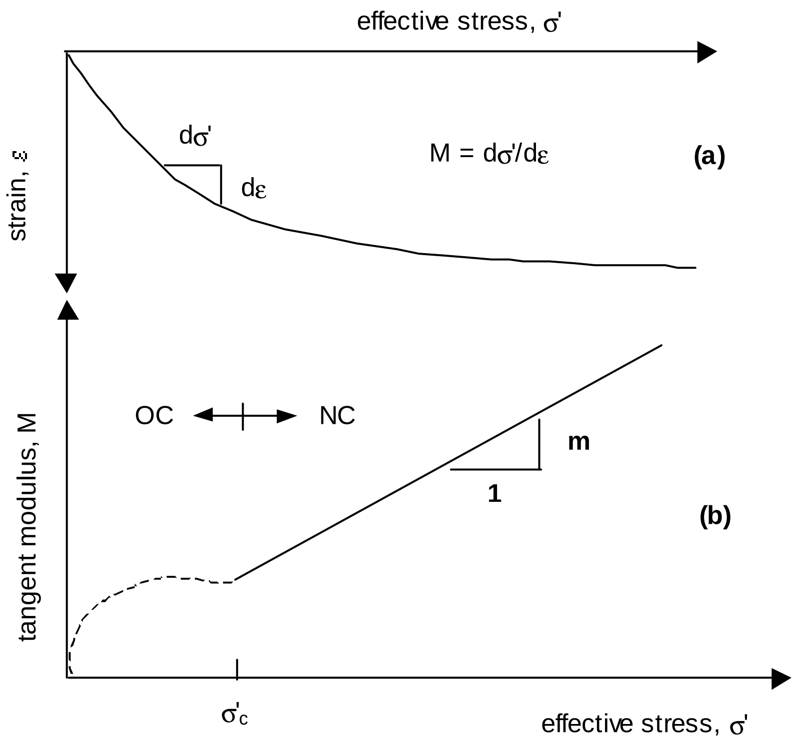

Figure 3.10 Stress-strain curve from an incremental oedometer test for clay.

For stresses above the preconsolidation pressure σ'_c, Janbu (1963) found that the virgin tangent modulus M, varies linearly with applied effective stress, such that for $\sigma' \geq \sigma'_c$,

$$\frac{dM}{d\sigma'} = m = constant \qquad \text{(Equation 3.3)}$$

where σ'is the applied stress, ε is the strain and *m* is the ***modulus number***. It depends on the initial or in situ water content. This dimensionless modulus number is the main parameter required for predicting the virgin deformation of clays and fine silts. Below the preconsolidation pressure, the tangent modulus is approximately constant. This simple plot therefore presents itself as an effective means of identifying the preconsolidation pressure of a soil.

Time resistance, R

For a specific load increment on a step wise loaded oedometer test on a fully saturated soil, the strain may be plotted versus time as in fig. 3.11(a). To study the characteristics of the curve and hence the soil behaviour, it is suggested to employ the first derivative of this curve called time resistance.

Time resistance, R is defined according to the following expression and its variation with time is plotted in fig. 3.11(b). Time is not the action that brings about the change in strain. The passage of time is considered the cause.

$$Time\ resistance,\ R = \frac{change\ in\ time\ (given)}{change\ in\ strain\ (measured)} = \frac{dt}{d\varepsilon} \qquad \text{(Equation 3.4)}$$

In this equation, *t* is the time and ε is the strain. *R* depends on both stress and time. For clayey soils, Janbu (1963) found that the long-term time resistance varies linearly with time so that for the time beyond primary consolidation,

$$\frac{dR}{dt} = r = constant \qquad \text{(Equation 3.5)}$$

This dimensionless number *r*, called the ***resistance number***, is the main parameter required for predicting creep or "secondary" deformations for clayey soils and silts. The resistance number is a function of in-situ or initial water content.

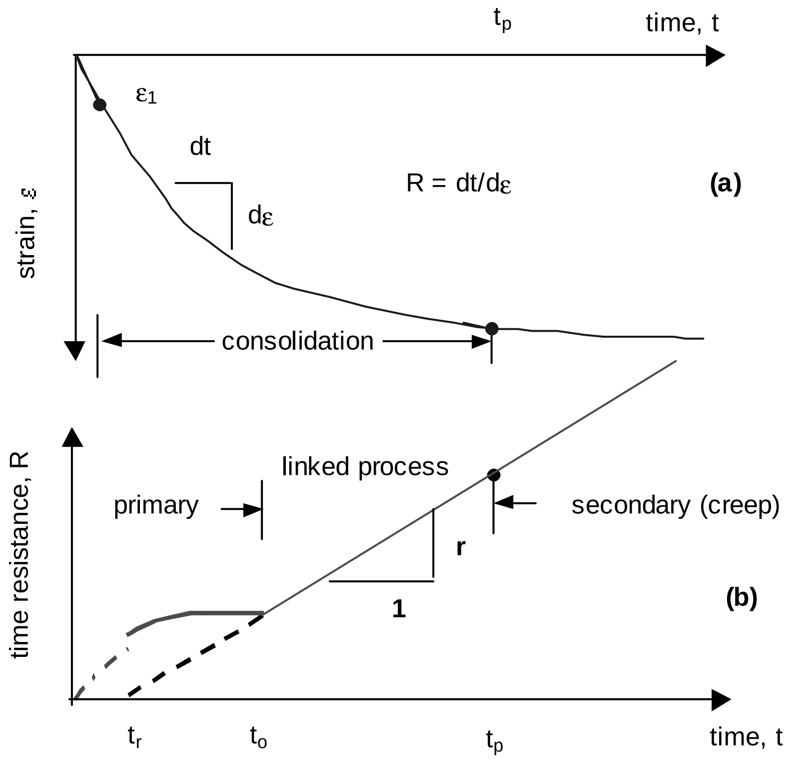

Figure 3.11 Time-strain curve of an incrementally loaded oedometer test

3.4.1.4 Merits of the resistance concept

The classical approach to analysing stress strain behaviour of saturated soils involves the void ratio and logarithm of effective stress (e-logσ' plot). The approach has registered considerable success but has the following inherent limitations that prevent its extension to unsaturated soils in general, and expansive soils in particular:

- The logarithmic scale in the e-log σ' distorts and hence hides away the details of the soil behaviour at low stresses. For instance, the log scale can not represent logarithm of one. Thus, virtually all the information at the start of a test is erased.

- The determination of the preconsolidation pressure σ'_c, using the Cassagrande (1936) and the Schmartmann (1953) corrections is highly dependent on the human factor and is therefore empirical

- The approach heavily depends on the initial void ratio of the soil. Now, the determination of the initial void ratio, e_o is both tedious and is a function of a number of variables. For instance, it requires knowledge of the specific gravity of the soil and the end of test water content. It also presumes that the water content at the end of test is the full saturation water content. The initial void ratio is very sensitive to both water content and specific gravity.

- The field corrected virgin line depends on the initial void ratio and degree of saturation, parameters that carry high degree of uncertainties in themselves.

- The concept of initial void ratio looses physical meaning in expansive soils, where the adsorbed water fill the voids for the wide range of water content.

- The approach is not readily applicable to cohesion-less soil and is therefore limited to cohesive (clayey) soils only.

On the contrary, the resistance concept offers the following advantages

- The concept is fundamentally sound and is based on principles that have been successfully applied in other fields of engineering and applied science.

- It is not sensitive to material type and hence presents in itself, a unifying framework for characterising engineering material behaviour. Janbu (1963, 1998) amply demonstrated the versatility of the concept in handling the full range of materials from soft soils to hard rock (even concrete).

- The concept simply and clearly relates the observed physical changes to the mathematical formulations. Raw data is used to obtain desired soil parameters without recourse to assumptions.

Therefore, the simplicity and rationality inherent in the resistance concept makes it the preferred method of analysis of the complex behaviour of unsaturated expansive soils.

3.4.2 The resistance concept applied to unsaturated expansive soils

When water is added to an unsaturated expansive soil, swelling pressure builds up and soil volume increases, depending on the degree of confinement. By identifying independent variables in the swelling process, it was possible to use the resistance concept in the analysis. The major departure from the analysis of saturated soils is that expansive soils have water content change (water flow) as an additional variable. It can be treated as either a dependent or an independent variable, depending on the analysis sought. The five resistance moduli associated with expansive soil behaviour are defined below and summarised in tables 3.2 and 3.3. They were conveniently defined with respect to degree of soil confinement.

3.4.2.1 Confined swelling condition

Table 3.2 Summary of resistance moduli for confined swelling

<table>
<tr><th>Resistance modulus</th><th>Action</th><th>Soil response</th><th>characteristic number*</th><th>relevance</th><th>comment</th></tr>
<tr><td>Soil resistance, S (1/kPa)</td><td>water flow</td><td>swelling pressure</td><td>soil number s</td><td>internal soil resistance to deformation by water flow</td><td>involves a physical change in the soil structure</td></tr>
<tr><td>Stress modulus, M_s (kPa)</td><td>swelling pressure</td><td>water flow</td><td>stress number m_s</td><td>soil resistance to development of swelling pressure</td><td rowspan="2">soil response without applied external load</td></tr>
<tr><td>Swelling resistance, Y (min./kPa)</td><td>time</td><td>swelling pressure</td><td>swelling number y</td><td>rate of the swelling process</td></tr>
</table>

* The characteristic numbers are normalised with atmospheric pressure to make them dimensionless

<u>Soil resistance, S</u>

Soil resistance is a measure of the soil resistance to the flow of water. Hydration leads to the swelling of clay particles and development of swelling pressure. Thus, the resistance is associated with the swelling of particles and hence the change in the structure of the soil. Consequently, soil resistance is a resistance to change in soil structure. In addition, swelling pressure can be taken as the measurable quantity that reflects the change in soil structure. It then follows that soil resistance is defined as

$$Soil\ resistance,\ S\ (per\ kPa)\ =\ \frac{change\ in\ water\ content\ (given)}{change\ in\ swelling\ pressure\ (measured)}\ =\ \frac{dw}{dP} \qquad \text{(Equation 3.6)}$$

Soil resistance can be plotted against water content to study the soil response to water flow. Therefore, soil resistance analysis focuses on the nature and effect of hydration in the soil. The slope of the curve is normalised against atmospheric pressure to make it dimensionless. It is herein called the ***soil number, s***.

Stress modulus, M_S

$$Stress\ modulus,\ M_s(kPa)\ =\ \frac{change\ in\ swelling\ pressure\ (given)}{change\ in\ water\ content\ (measured)}\ =\ \frac{dP}{dw} \qquad \text{(Equation 3.7)}$$

Stress resistance modulus is a measure of resistance to development of swelling pressure with respect to water flow. It therefore, links development of swelling pressure to hydration. When plotted against water content it shows the effect of increasing water content on the swelling pressure development. The slope of the curve is normalised against atmospheric pressure to make it a dimensionless number and is herein called the ***stress number,*** $\boldsymbol{m_s}$.

Stress modulus is reciprocal to soil resistance. Therefore, a combined analysis of the two resistances brings out the nature of the interaction between the soil, water flow and the subsequent development of swelling pressure.

Swelling resistance, Y

$$Swelling\ resistance,\ Y\ (min/kPa)\ =\ \frac{change\ in\ time\ (given)}{change\ in\ swelling\ pressure\ (measured)}\ =\ \frac{dt}{dP} \qquad \text{(Equation 3.8)}$$

The swelling resistance Y, is a resistance to the development of swelling pressure with respect to time. However, time is not the action that brings about the mobilisation of swelling pressure. The dependency of the swelling process on time is a consequence of its link to water flow. It is therefore, a resistance within the swelling process. When plotted against time, the slope of the curve is a measure of the rate of the swelling process. The slope is normalised with respect to atmospheric pressure to give a dimensionless number and is herein called the ***swelling number, y***. Mawire and Senneset (2000) analysed the mobilisation of confined swelling pressure.

3.4.2.2 One-dimensional free swelling (unconfined) condition

The condition of unconfined swelling applies to both cases of free swelling and swelling under applied external load. The appropriate moduli are summarised in table 3.3 and defined as follows.

Table 3.3 Summary of resistance moduli for unconfined swelling

Resistance modulus	***Action***	***Soil response***	***characteristic number****	***relevance***	***comment***
Tangent modulus, M (kPa)	swelling pressure	swelling strain	modulus number, m	stress-strain behaviour	soil response to the change in external (or internal) stress
Time resistance, R (min./kPa)	time	swelling strain	resistance number, r	creep/secondary swelling	with (or without external) pressure

* The characteristic numbers are normalised with atmospheric pressure to make them dimensionless

<u>Tangent modulus, M</u>

Tangent modulus is as defined for saturated soils in equation 3.2 (Janbu, 1963). The replacement of effective stress with swelling pressure in the expression is consistent with dimensional analysis. However, it is shown in chapter six that swelling pressure is another form of the internal stress. Accordingly, tangent modulus is generally defined, with the effective stress being internal or external. It is a stress resistance to deformation arising from a change in the effective stress acting on the soil.

<u>Time-resistance modulus, R</u>

Time resistance is as defined for saturated soils in equation 3.4 (Janbu, 1963). However, the strains measured can be swelling, compression or a combination of the two, depending on the magnitude of the applied external pressure relative to the swelling potential of the soil.

3.4.3 The diffusion concept

The diffusion concept has been successfully used in the field of polymer and molecular chemistry to study the movement of dyes in materials and oxygen in the biological cells (Frisch, 1962; Crank, 1975; Neogi, 1996). It has also been used in its simplest form to characterise flow of water in non-expansive soils, where no internal stresses develop during and because of the flow. This forms the basis of water flow in unsaturated, non-expansive soils (Philip, 1955). The concept is presented herein in its basic form and is used to characterise unsaturated expansive flow. The merits of the diffusion concept over mechanical flow analysis are given.

3.4.3.1 Definition

Diffusion is the movement of a penetrant (liquid or gas molecules) through a permeable material under a concentration gradient of the penetrant. The movement is commonly expressed in terms of Fick's law of diffusion

3.4.3.2 Fick's law of diffusion

The one-dimensional equation governing non-steady state diffusion is

$$\frac{\partial c}{\partial t} = \frac{\partial}{\partial x}\left(D\frac{\partial c}{\partial x}\right) \qquad \text{(Equation 3.9)}$$

where c is the concentration of the penetrant at time t, D is the coefficient of diffusivity, and x is the linear dimension in the direction of diffusion.

The coefficient of diffusivity is not necessarily a constant. It can depend on concentration, time and or the history of the diffusion process. Accordingly, in general, the partial differential equation is analytically indeterminate, having no exact solution for any boundary conditions. The simplest case of non-steady flow, is when the coefficient of diffusivity *D* is a function of the concentration only and has been popularly coined "***Fickian flow***". Otherwise the non-steady flow is anomalous and is called ***non-Fickian flow*** (Crank, 1975).

3.4.3.3 Fickian flow

By resorting to the similarity solution technique and invoking the Boltzmann transformation, the non-linear, partial differential equation for Fick's law reduces to an ordinary defferential equation. The differential equation can be numerically or graphically solved. When *D* is a constant, the coefficient of diffusion can be graphically determined from the slope of the fractional water uptake plotted against square root of time. Thus, Fickian flow can be characterised by a linear relationship between fractional uptake of the penetrant and the square root of time using equation 3.10. Fractional penetrant uptake is the ratio between the amount of penetrant (U_t) absorbed at time *t*, and the amount absorbed after infinite time or to saturation of the absorbing material U_∞. The linearity is sustained for at least 50 per cent of the fractional uptake.

$$\frac{U_t}{U_\infty} = \frac{4}{h}\left(\frac{Dt}{\pi}\right)^{\frac{1}{2}} \qquad \text{(Equation 3.10)}$$

When *D* is not constant, equation 3.10 gives some mean value for the appropriate range of concentration, and methods of successive approximations may be used to determine the variation of the diffusion coefficient with concentration.

The diffusion concept has been successfully used to characterise non-steady flow of water in unsaturated soils. No internal stresses develop during and because of the flow. Thus, water flow is Fickian. In addition, it lends itself to the simplest case of Fickian flow, where the coefficient of diffusivity is almost constant. Consequently, semi-analytical solutions are prevalent.

3.4.3.4 Non-Fickian flow

Non-Fickian flow arises when the interaction between the penetrant and the material results in the interference of continued movement of the penetrant. This can arise if for instance the interaction leads to (i) a temperature rise, (ii) chemical bonding between the penetrant and the material or (iii) using up of the penetrant during diffusion. In the case of expansive soils, the flow depends on water content, swelling pressure and/or swelling strains that develop during and because of unsaturated flow. Accordingly, the flow of water in unsaturated swelling soils is non-Fickian. Consequently, the resultant differential equation of Fick's law is not analytically determinate. The methods of successive approximation do not apply because the variation of D is not just a function of water concentration. It also depends on the swelling pressure and strains.

Uzan and Lytton (1978) assumed Fickian flow in unsaturated expansive flow. They adopted a linear relationship between the moisture diffusivity and water content. However, the application is limited because it overlooked the effect of swelling pressure and swelling of clay particles on water flow, and thus assumed Fickian flow.

3.4.3.5 Merit of the diffusion concept

The diffusion concept is valid for expansive soils because the clay-size particles of clayey or cohesive soils (<0.002mm) fall within the molecular range that is classified as colloids. The particle size for colloids ranges from 10^{-3} to 10^{-6}mm. The associated soil properties such as plasticity, adsorption of molecules and the surface forces dominate gravitational forces and are consistent with the colloidal properties of the constituent clay minerals (Yong and Warkentin (1975). This explains why

Fick's law of diffusion has been generally successful in characterising water flow in non-expansive soils. It also follows that the concept can be rationally extended to non-Fickian flow of water in expansive soils. However, it is necessary to account for the effects of the swelling clay minerals, swelling pressure and swelling strains that develop during and because of water flow.

Chapter 4

Laboratory test results

In this chapter are presented tests results obtained from the test programme described in chapter three. First is presented the routine test results, which describe the investigated soil. The swelling behaviour of the soil was investigated along three different wetting paths namely, (1) suction-gradient, (2) pressure-gradient, and (3) flooding. The wetting-paths simulate the slowest, intermediate and fastest conditions of soil wetting, respectively. The results are presented in the context of the degree of soil confinement, which also defines the mode of soil swelling (swelling paths).

4.1 Routine test results

4.1.1 Classification test results

Samples were taken from the field and tested in accordance with Standards Association of Zimbabwe procedures, SAZS No 185: Part 1:(1998). Table 4.1 summarises the soil data obtained from routine and other physical tests on the soil. The dominant clay mineral in this soil is Sodium-Montmorillonite.

Table 4.1 Summary of soil data

Description	***value***
Liquid Limit, LL	67 per cent
Plastic Limit, PL	24 per cent
Plasticity Index, PI	45 per cent
Shrinkage Limit, SL	19 per cent
Clay fraction	40 per cent
Silt fraction	28 per cent
Saturation water content	42 per cent
Grain density, G_s	2.58 kgm^{-3}
Optimum water content	18 per cent
Lower Compactive Effort (LCE)	1768 kgm^{-3}

4.1.2 Soil-Moisture Characteristic Curves

The wetting and drying Soil-Water Characteristic Curve for Avon clay were determined. The axis translation technique (Hilf, 1956) was used for up to 1500kPa suction, while the filter paper method (Chandler, 1967) was used to extend the curves in the higher suction range. The curves are presented in fig. 4.1. However, in these methods the sample is tested under unconfined conditions. Therefore, the wetting and drying curves thus obtained do not account for the possible development of swelling pressure and its effects during the processes. Therefore, the curves are considered of limited practical use in expansive soils, where swelling pressure develops and influences the soil response. However, they are presented herein for completeness and used to qualify some of the analysis, as necessary.

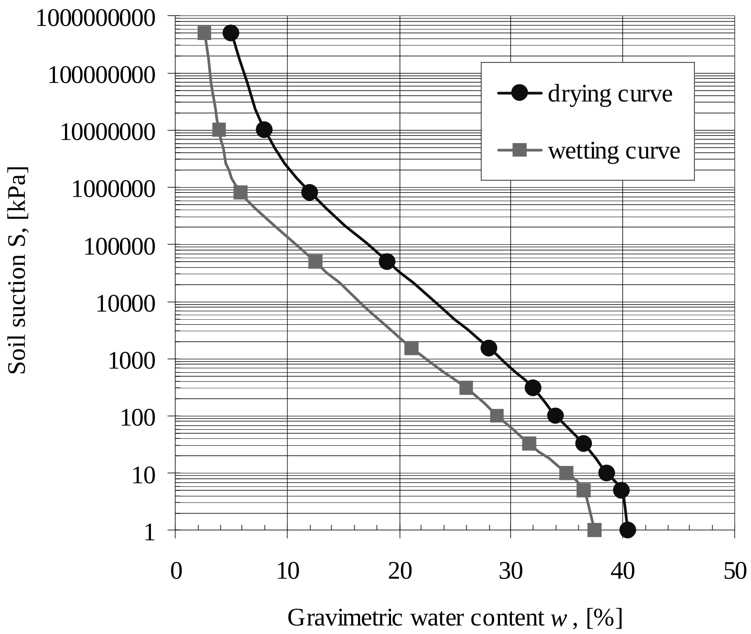

Figure 4.1 Soil-Water Characteristic Curve for Avon clay

4.2 Confined swelling test results

The test procedure and rationale for the confined swelling test are discussed in section 3.2.2 of chapter three. In the test, the soil element experiences no volume change. As such, the only variables are water flow and increase in swelling pressure. The test results bring out the effect of swelling pressure on the swelling process, independent of volume change. Twenty-seven samples were tested under confined conditions. Only representative test results are presented. Pertinent points from the test results are given alongside the results.

4.2.1 Swelling pressure - time results

The sample had initial water content of 19.8 per cent and a degree of saturation of 0.554. The mobilisation of swelling pressure with time is plotted fig. 4.2.

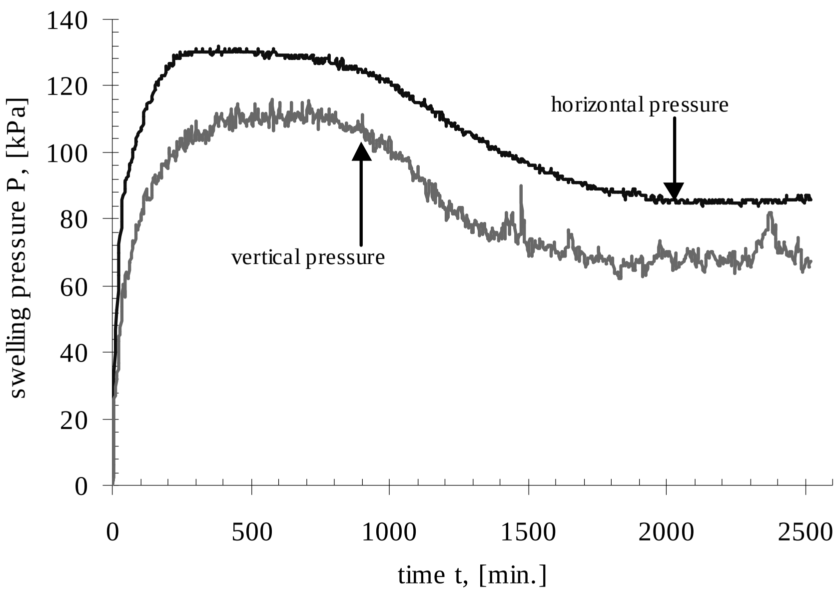

Figure 4.2 Mobilisation of swelling pressure with time (pressure-gradient)

<u>Pressure-time plot: pressure gradient wetting</u>

The graphical plot reveals several important aspects of confined swelling. The swelling pressure was mobilised in both vertical and horizontal directions, rising to a peak value and then slowly decreasing to steady state condition. The magnitude of the vertical and horizontal swelling pressures are different, with the horizontal swelling pressure being consistently higher than the vertical pressure. In this particular test result, the maximum values are 130kPa and 110kPa respectively. Lastly, the shapes of the swelling pressure curves are similar, forming a plateau at the maximum values. Accordingly, analysis can be carried out using any one of the curves without lose of generality.

These observations seem to suggest two important things about confined swelling. Firstly, both time and water content influence the swelling process. Secondly, the mobilised swelling pressure is anisotropic: the swelling pressure in the horizontal direction is higher than that in the vertical direction. The question of anisotropy is dealt with in chapter six.

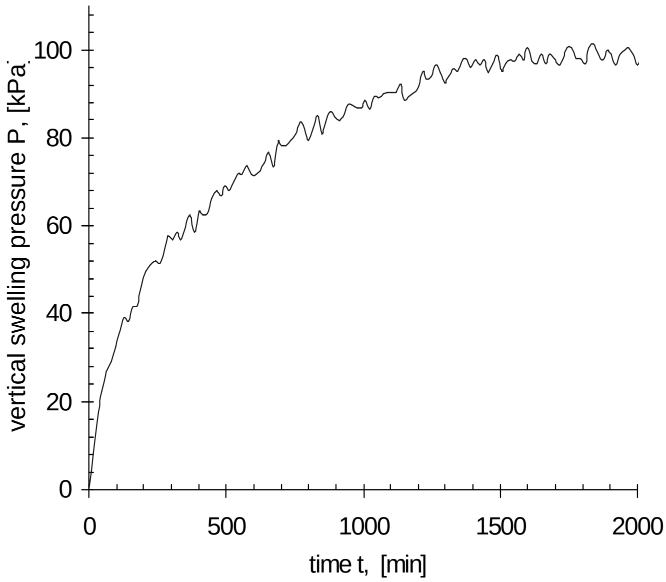

Figure 4.3 Mobilisation of swelling pressure with time (suction gradient)

Pressure-time plot: suction gradient wetting

The swelling pressure rises to a maximum value, at which the swelling process stops. The maximum swelling pressure value seems to be linked to the initial soil water demand.

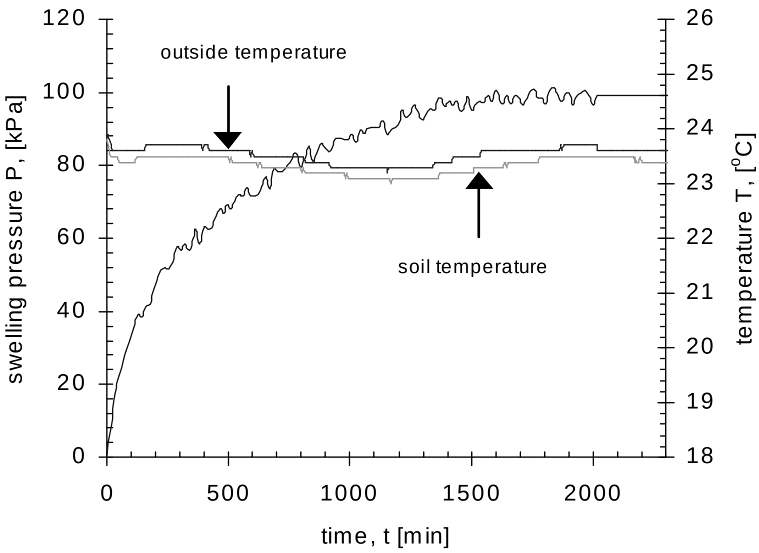

Figure 4.4 Change of soil temperature during confined soil swelling (suction gradient)

Soil temperature during soil swelling

The results presented in fig. 4.4 show that the soil temperature changed in response to the outside temperature, and not to the swelling process. In addition, the temperature changes for both the soil and the outside are within one degree Celsius. The maximum difference between the soil and outside temperatures was 0.25 degrees Celsius, but mostly fluctuating between zero degrees Celsius and 0.15 degree Celsius. The initial drop in soil temperature was the effect of placing the sensor, previously at room temperature, in the sample with a lower temperature.

The small and almost constant temperature changes experienced by the soil sample, relative to the outside temperature, indicate two things. Firstly, there was no temperature gradient between the sample and its surrounding to affect the swelling process. Secondly, the swelling process did not change the sample temperature. Similar observations were made on all confined and unconfined swelling tests carried out along the different wetting paths investigated. Therefore, it was concluded that the swelling of the investigated soil is isothermal. This conclusion simplified the subsequent analysis of test results by making temperature a constant.

4.2.2 Swelling pressure - water content results

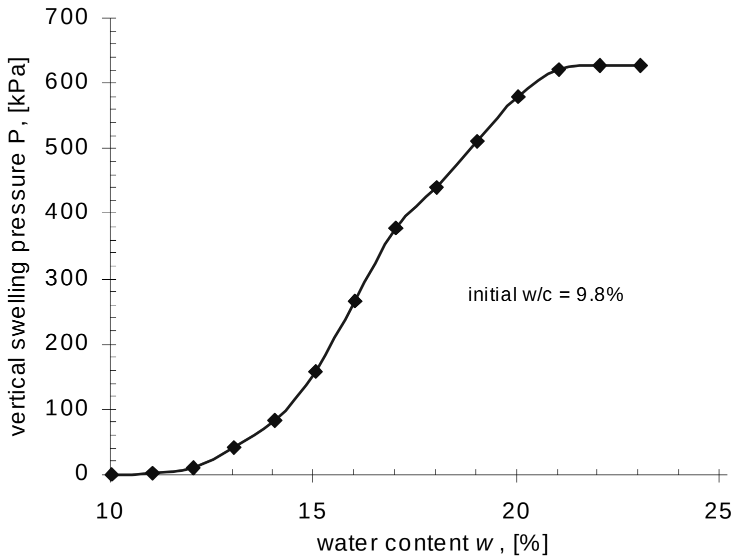

Figure 4.5 Water content - swelling pressure plot (suction gradient)

Water content-swelling pressure plot: suction gradient wetting

The swelling pressure rises to a peak value with increase in soil water content. Thereafter, the pressure appears to remain constant. It seems that the swelling process ends with the onset of maximum swelling pressure as implied by the induction concept.

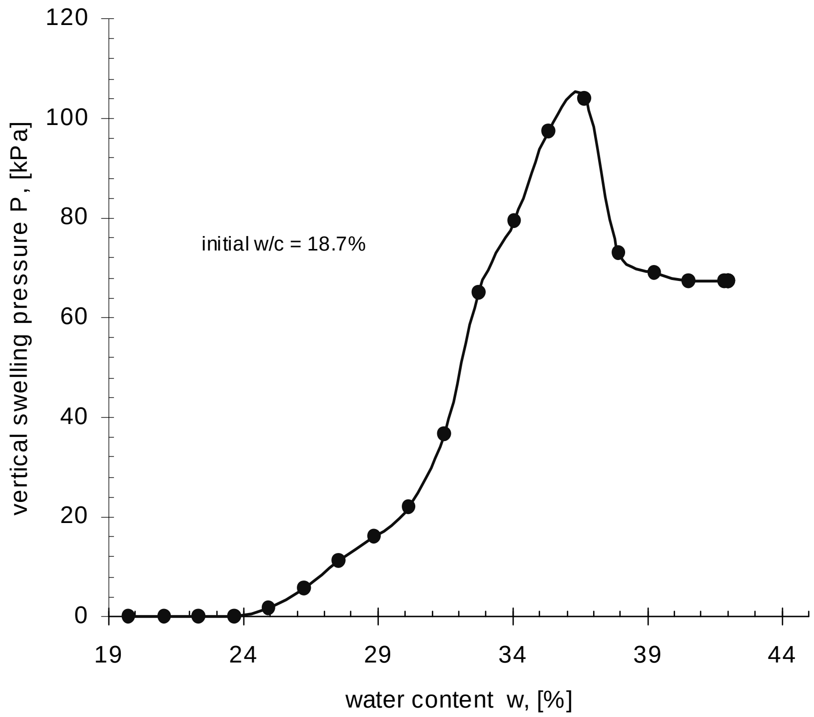

Figure 4.6 Water content - swelling pressure plot (pressure gradient)

Water content-swelling pressure plot: pressure gradient wetting

Unlike the suction wetting path in fig. 4.5, the swelling pressure drops after the peak to a residual value. This suggests that the post-peak process is not swelling, at least in the context of increase in swelling pressure. As such, it points to the completion of the swelling process at the peak stage. Apart from this, a pressure gradient is required to remove some of the swelling pressure.

4.2.3 Water flow- time results

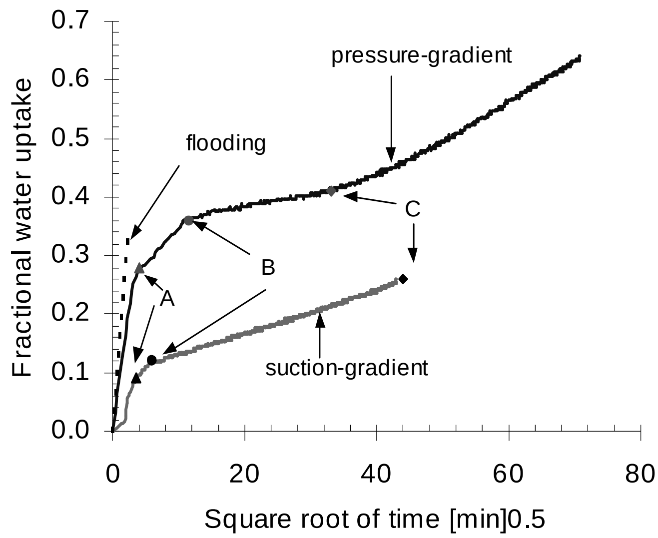

Figure 4.7 Fractional water uptake (water flow) plotted against time

<u>Confined water flow</u>

The changes in water-flow patterns for the investigated wetting paths are shown in fig .4.7. Water flow is plotted in accordance with the diffusion concept (Crank, 1975). Comparatively, the pressure-gradient path allows water to flow faster than the suction-gradient path, as expected, while flooding is fastest.

It is clear that the mobilisation of soil cohesion leads to a change in water flow mechanism. The central role of the arbitrary points A’, B and C in coupling water-flow and swelling process is also evident. The non-linear relationship up to point A’ is indicative of the mechanical nature of water flow. An advancing wetting front defines the wetting up of the soil. The transition of the wetting front into a SBS at point A’ affects the water flow pattern. The 1st-level soil destructuration starts the closure and filling of air voids. This process affects the mechanical flow of water. The effect is greater along the pressure-gradient wetting-path than the suction-gradient path because the swelling process is faster. In the former case, the increased water uptake forces a rapid transformation of the soil structure at point A'. In the latter case, the change is gentle because the driving potential arises from the soil water demand. The flow rate decreases further at point B. The voids and air-channels are effectively closed, according to the SBS physical soil model. Thereafter, water flow is by diffusion. The water flow along the two wetting paths plots almost parallel in the phase (B-C), confirming that water flow is no longer path-dependent. It is governed by the changed soil structure or Swelling Boundary Surface.

4.3 One-dimensional free swelling test results

The test procedure and rationale for the confined swelling test are discussed in section 3.2.3 of chapter three. In the test the soil element experiences simultaneous change in volume, swelling pressure and water content. As such the variables are water flow, swelling pressure and vertical swelling strain. Fifteen samples were tested in this series of tests. Only representative test results are presented. Pertinent points from the test results are given alongside the results.

4.3.1 Swelling pressure - time results

Swelling pressure - time plot: flooding

Figure 4.8 shows the mobilisation of swelling pressure with time for different initial water content. The sharper the curvature, the faster is the rate. There is a clear influence of initial water content on the shape of the curves and hence rate of swelling. The curves can be put in three different categories. The first category is for water contents less than 24 per cent. The curvature of the curves in this category is most pronounced. It sharply rises and falls. The residual pressures attain the same limiting value however, after a relatively long time. The second category is for the water contents between 24 and 26 per cent. The curves are moderately pronounced. The two curves merge during the post-peak phase, but well before the residual value is attained. In addition, the residual pressure is very slowly approached, with respect to strain.

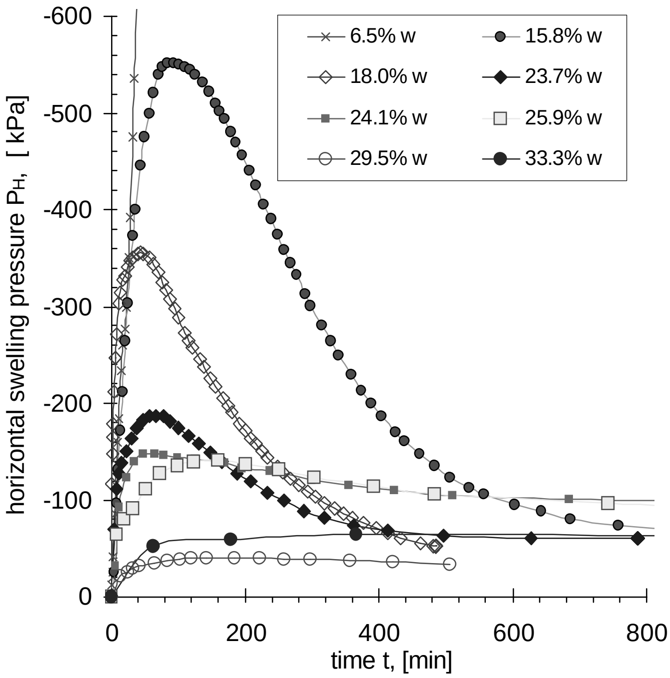

Figure 4.8 Swelling pressure - time plot (flooding path)

The last group comprises of water content above 29.4 per cent. In this category, the pressure is effectively the same through out the swelling process. In addition, the curves gently rise to a limiting value of about 50kPa. The residual pressures from the first category quickly approach that of the last, while that of the second category is delayed.

The forgoing analysis points to the central role of the changing soil in the mobilisation of swelling pressure. According to the induction concept, soil attraction is induced at point B, which corresponds to 24% (plastic limit of the soil). Thus, soil attraction restraints build up of swelling pressure for soil samples with initial water content above the plastic limit. The effect appears to increase with increasing initial water content.

4.3.2 Swelling pressure - strain results

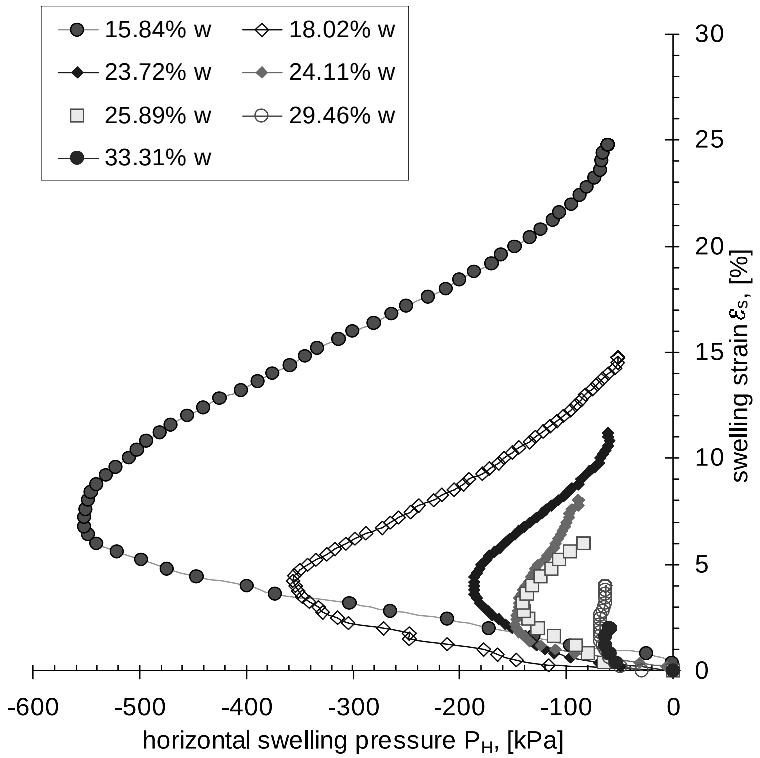

Figure 4.9 Swelling pressure - swelling strain plot (flooding)

Swelling pressure - swelling strain plot: flooding

The mobilisation of swelling pressure and swelling strain is plotted in fig. 4.9. The same categories identified in sub-section 4.3.1 above are discernible here. The curves in fig. 4.9 show a distinct change in both the shapes of the curves and mobilised pressure between two ranges of water content namely, 18.0-23.7 per cent and 25.0-29.5 per cent water content, respectively. The sharpness of the elbow decreases from 23.7 per cent and disappears at 29.5 per cent. These observations are consistent with the proposed SBS physical soil model.

4.3.3 Swelling strain - time results

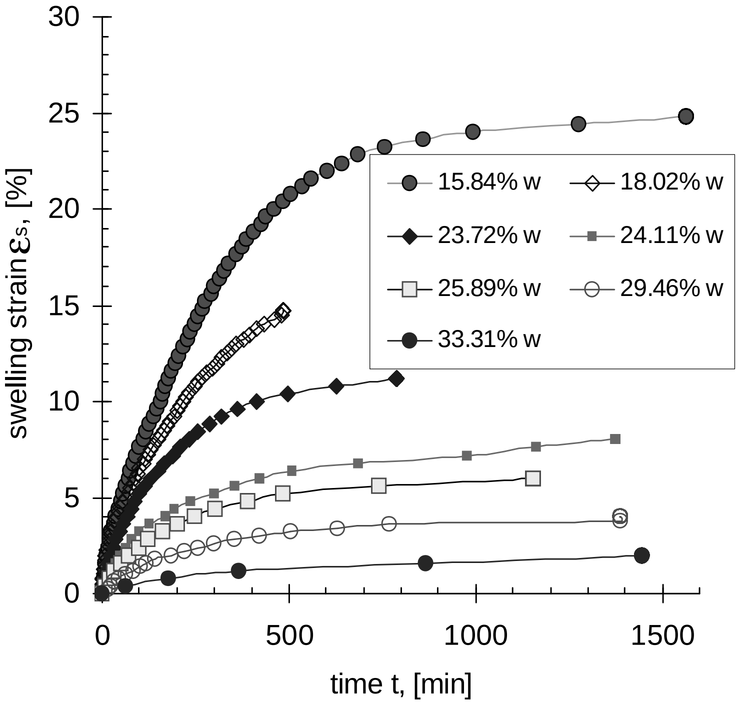

Figure 4.10 Time-swelling strain plot for a range of initial water content

<u>Swelling strain - time plot: flooding</u>

The mobilisation of swelling strain with passage of time is plotted in fig. 4.10. The same categories identified in sub-section 4.3.1 above are discernible here. The curves show a distinct change in both the shapes of the curves and mobilised swelling strain between two ranges of water content namely, 18.0-23.7 per cent and 25.0-29.5 per cent water content, respectively. The curves become progressively flatter with increase in water content. These observations are consistent with the proposed SBS physical soil model. The maximum soil cohesion that is mobilised at 24 per cent water content (point B) severely restrains developing of swelling strain.

4.4 General discussion

The laboratory test results presented in this chapter are typical of the investigated soil. They seem to support the induction concept and bring insight to the swelling behaviour of the investigated soil. However, no definitive conclusions can be made based on the raw data. However, a study of the first derivatives of the raw data (resistance concept) is one way of getting a better understanding of the soil behaviour. Accordingly, the subsequent detailed analysis is based on the resistance concept.

Chapter 5

Validation of the new concept

Test results for ‘undisturbed’ samples of an unsaturated expansive soil, tested according to the test procedures outlined in chapter three, are presented and analysed. The aim of this chapter is to validate the induction concept presented in chapter two and to determine the soil parameters. Representative test results from chapter four were analysed using the resistance and diffusion concepts, discussed in section 3.4 of chapter three. The validation was demonstrated along suction gradient wetting, which is the soil’s natural response to soil wetting. In this case, soil and stress resistances were considered. The swelling modulus analysis was carried out on results obtained along the flooding path. Flooding is the fastest wetting path and therefore facilitates the study of rapid soil response to wetting. The chapter concludes by determining the soil parameters. The work presented in this chapter is an original contribution to the analysis and characterisation of soil swelling.

5.1 Soil resistance analysis: confined swelling

In this series of tests, water was added to the soil sample at the base, with the water level in the burette kept at the same level as the bottom of the sample. Water uptake was therefore by suction gradient due to the soil water potential. Typical test results from sample no. SP-027 are plotted and analysed. The sample had initial water content of 18.7 per cent.

Soil resistance was defined in section 3.4.2 of chapter three. It is a resistance to soil deformation by water flow. The analysis was carried out with reference to the soil number. The soil number is directly related to the change in soil resistance and soil structure; the smaller the number, the faster and easier is the change.

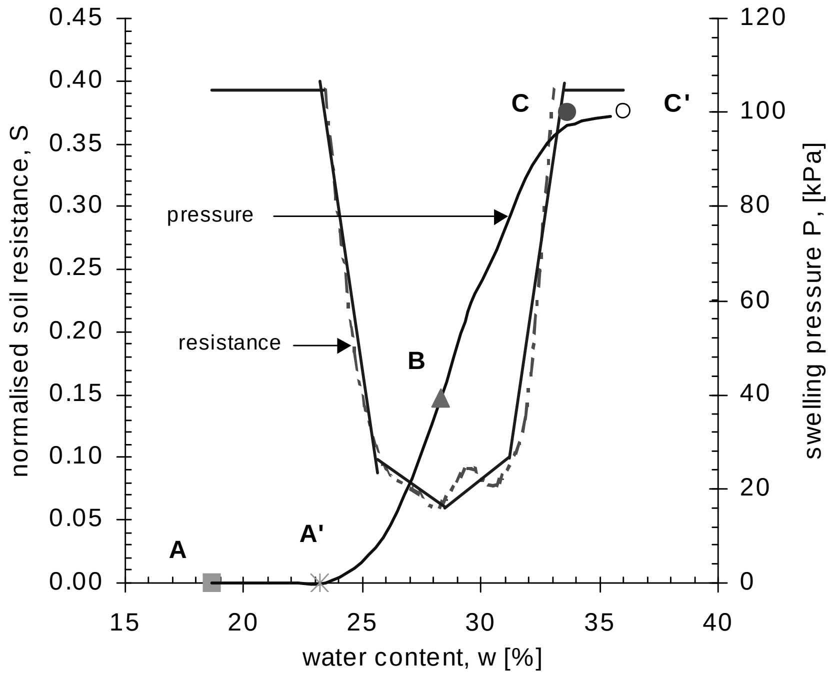

Figure 5.1 Soil resistance along suction-gradient wetting path

5.1.1 General

Reference is made to the soil resistance-time relationship in fig. 5.1 and table 5.1, which summarises the pertinent data. The corresponding increase in water content is plotted against time in fig. 5.2. The arbitrary water content points indicated are as defined in the induction concept. The positioning was based on the test results. Thus, the water content points shown in figs. 5.1 and 5.2 show that the change in water flow is linked to the change in soil structure. This confirms the establishment of dynamic equilibrium during soil swelling as proposed in the induction concept. The different values of the soil number conveniently divide the soil response into stages. Each stage is herein called a ***swelling phase***. The nature of each phase is defined during the analysis.

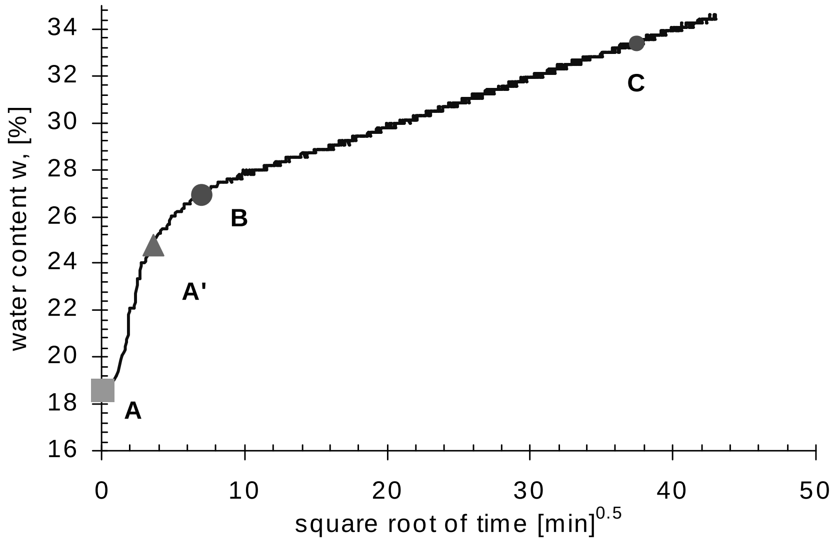

Figure 5.2 Increase of soil water content with time-suction gradient path

Table 5.1 Summary of soil resistance analysis: suction wetting-path

phase	***soil number, s***	***period (%)***	***water content (%)***	***% water increase***	***% pressure increase***
A - A'	0	0.3	18.7 - 23.2	12	0
A'- B	-0.13	1.1	23.2 - 25.6	6	8
	-0.015	8.8	25.6 - 28.3	4	14
B - C	+0.015	36.7	28.3 - 31.2	11	57
	+0.13	53.1	31.2 - 33.5	6	21
C - C'	0	(69.9)*	33.5 - 36.0	6	0

* The period is based on the swelling time from point A to point C.

From fig. 5.1 suction wetting reveals four of the six swelling phases defined in the induction concept (chapter two). This is because the soil adhesion that causes water flow is used up in the process. It is depleted at point B. Thereafter, the induced adhesion (osmotic potential) is dependent on soil attraction. Thus, beyond point C’, the remaining induced osmotic potential is not sufficient to sustain water diffusion.

There is remarkable symmetry in the change of the normalised soil resistance about point B (fig. 5.1). The two-stage loss (A-A’) and regain (A’-B) of soil resistance was explained in terms of the induction concept as follows.

5.1.2 Initial phase (A - A’)

The first phase is called the initial phase because it is the start of the swelling process. Thus, the initial phase is characterised by a constant soil resistance (s = 0) and 12 per cent increase in water content in 0.3 per cent of the swelling time. The rapid water flow in such short a period was attributed to the open pores associated with the ‘particulate’ nature of the particles, when the soil is dry. The hydrating particles press the pore water and generate pore water pressure. As such, the loss in soil resistance due to hydration is counter-balanced by a build up of pore water pressure. The sudden decrease in the soil resistance at point A’ is associated with the end of 1st-level soil destructuration and the onset of the second. The period of the initial phase (0.3 per cent) is sufficiently small to refer to initial soil response as instantaneous.

5.1.3 Soil softening (A’ - B)

Second-level destructuration characterises this phase. It involves the destruction of the primary particle bonding within the clay platelets and reduces them to hydrated clay minerals. The soil structure and hence resistance to deformation depends on the primary particle bonding. It follows then that a small weakening of the bonding by hydration leads to significant reduction in the soil resistance.

Now the loss of soil resistance is simultaneous with growth of the adsorbed water layers and hence a physical soften of the soil. Therefore, the overall soil response is herein called soil softening and the phase was appropriately called the ***softening phase***.

The softening phase is characterised by two stages however, with the first stage being very pronounced. The soil number for the first stage (s = -0.13) is four times smaller than the second stage. The soil number is rate quantity. Thus, the smaller soil number (s = -0.13) indicates that the change in soil resistance is relatively quick and easy. This is attributed to the breaking of the primary particle bonding.

The second stage of softening (s = -0.015) is about ten times slower than the first. The sudden reduction in the rate of soil destructuration is a result of reduced water flow. The depletion of suction means that water flow is reduced on the one hand. On the other hand, soil destructuration encourages mobilisation of soil cohesion. Accordingly, soil cohesion has an upper hand and provides resistance to deformation. Thus, there is simultaneous decrease in soil resistance on the one hand, and the restoration of resistance by mobilising soil cohesion on the other. However, the soil resistance due to soil attraction decreases to a minimum value at point B, where soil attraction is zero. The soil resistance at point B is due to maximum soil cohesion. This is consistent with the induction concept.

Fig. 5.3 shows the mobilisation of swelling pressure with degree of soil saturation. Two important deductions were made. Firstly, water content points A' and B do not plot on the swelling pressure line. In addition, the degree of saturation seems to increase beyond full saturation. It is asserted herewith that these anomaly points to the fact that degree of soil saturation is pressure dependent. The current expression for degree of saturation assumes atmospheric pressure all the time, hence a linear plot of the water content points. Secondly, the degree of saturation is one that is, full saturation at point B as asserted in the induction concept. This saturation is internal saturation, with respect to depletion of adhesion or suction and growth of adsorbed water layers.

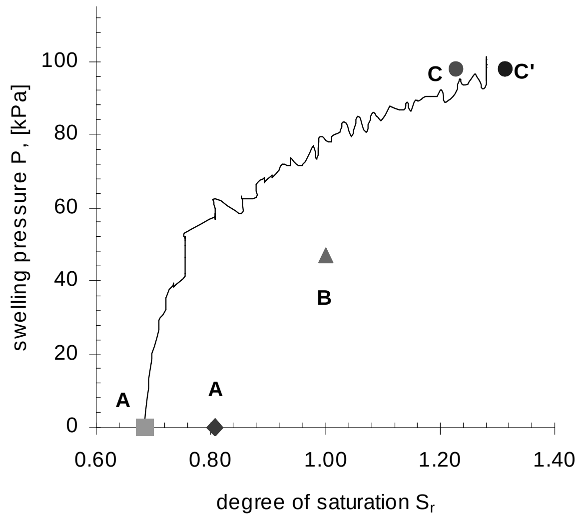

Figure 5.3 Mobilisation of swelling pressure with increase in degree of soil saturation

5.1.4 Stiffening phase (B-C)

This phase is characterised by an increase in soil resistance. In accordance with the induction concept, the increase in soil resistance is due to the induced soil attraction, which squeezes the adsorbed water layers and make them very stiff. Accordingly the phase was called ***stiffening phase***.

The increase in soil resistance is two-staged and is a mirror image of the softening phase about point B. The two stages were explained by recourse to the induction concept as follows. The maximum soil cohesion at point B accounts for the initial increase in soil stiffness (s = +0.015). That is, soil cohesion pulls the hydrated clay minerals together to within the molecular range of the solid clay minerals. In the process, the overall soil stiffness increases significantly. However, the rate is ten times slower than that due to the induction phenomenon (s = + 0.13).

The sudden jump in soil stiffness is due to the induction of soil attraction. The induction phenomenon depends on interparticle distances and not water content. Accordingly, it proceeds to the end. The restoration of soil resistance at point C is very significant. It confirms the induction phenomenon is an attempt by the soil to return to the initial condition, where soil structure was defined by the mobilised soil attraction as conceptualised.

5.1.5 Peak phase (C-C')

The ***peak phase*** defines the stage when swelling pressure is a maximum. The soil number is zero indicating that the soil response is not measurable. This is the case because water flow is by diffusion, under the induced osmotic potential. Apart from this, the available water is that squeezed out during induction. The diffusion of water leads to 'internal' rearrangement of the clay minerals until internal yield point is attained.

The soil swelling response suddenly stops at maximum swelling pressure. This indicates the end of the swelling process. That is, the swelling process is completed at point C', when the soil is at the limiting equilibrium (internal yield point). This should be the case because physically there is internal equilibrium, with adhesion being partly replaced by osmotic potential and partly by the adsorbed water. The adsorbed water squeezed out during induction is fully assimilated back at point C'.

<u>Validation of the induction concept</u>

The validity of the induction concept can be demonstrated considering the loss and restoration of the soil resistance. Figure 5.1 shows that the loss and regain of soil resistance between points A' and C is symmetrical about point B. The symmetry is also evident from the soil resistance data summarised in table 5.2.

Table 5.2 Interaction between softening and stiffening phenomena

phenomenon	***soil number, s***	***water content change (%)***	***Normalised soil resistance***
softening (A'- B)	- 0.13	+4	0.39
stiffness (B - C)	+ 0.13	+6	0.40

According to the induction concept, soil attraction underlies the soil resistance. Therefore, the observed symmetry points to the induction of soil attraction at point B. The soil number is a rate quantity. Now the soil numbers are reciprocal in magnitude and the pattern of soil response is symmetrical about point B. This indicates a reversal process and is indicative of the induction phenomenon. Therefore, the analysis of the test results presented in this completely validates the induction concept.

<u>Development of the SBS</u>

Validation of the induction concept implies validity of the proposed SBS physical soil model. This is so because, the SBS model is the physical complement of the induction concept. Thus, the central role of point B in the previous analysis points to the uniqueness of the soil structure at that point and hence the SBS. The long stiffening period of 97.3 per cent reflects the influence of the soil structure on soil swelling. The soil is now a thick plastic paste, whose structure takes long to change. This concurs wit the proposed SBS model.

5.2 Stress resistance analysis: confined swelling

Stress resistance was defined in section 3.4.2 of chapter three. It is a resistance to the mobilisation of stress and hence the induction phenomenon. The analysis was carried out in terms of the ***stress number*** m_s. A stress-number indicates how easy it is for stress to develop. The bigger the number, the easier is the development and hence the faster is the increase. The analysis sought to independently validate the induction phenomenon, in addition to rationalising the SBS physical soil model. The stress-resistance mobilised along the suction-gradient wetting path was analysed is presented in this section.

Reference is made to the plot of swelling pressure and stress resistance against water content in fig. 5.4. The results are from test sample number SP-027, which is typical of the series. The stress numbers for the different phases are summarised in table 5.3. The symmetry of the change in stress resistance about point B is clear in the figure. It points to the presence of an induction phenomenon, as will be shown in the following analysis.

5.2.1 Initial phase

The initial phase (A-A’) has no stress resistance (m_s = 0). According to the induction concept, stress resistance is due to either soil attraction or soil cohesion. Soil cohesion starts to mobilise at point A’, while soil attraction is being removed at this stage. Thus, the initial phase registered no stress resistance. However, the period is instantaneously small, being 0.2 per cent of the swelling time.

Table 5.3 Summary of stress resistance analysis: suction wetting path

phase	***m_s***	***period (%)***	***water content (%)***	***water increase (%)***	***pressure increase (%)***
A - A’	0	0.3	18.7 - 25.6	12	0
A’- B	+3.2	9.9	25.6 - 28.3	4	22
B - C	-3.2	89.8	28.3 - 33.5	6	21
C - C’	0	(69.9)*	33.5 - 36.0	6	0

* The period is based on the swelling time from point A to point C.

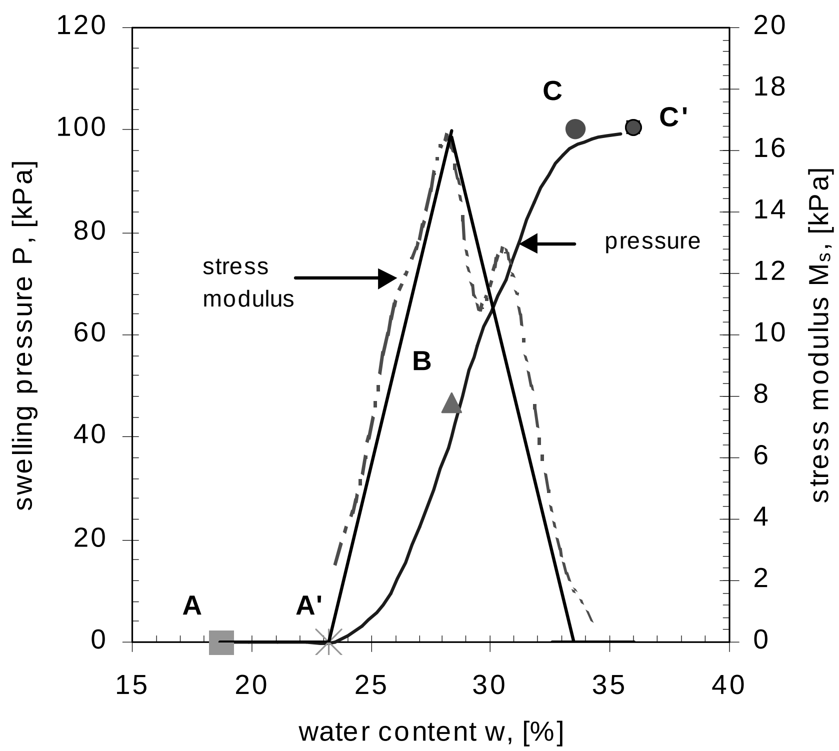

Figure 5.4 Swelling-pressure and stress resistance modulus curves

5.2.2 Softening phase

A positive stress number (m_s = +3.2) characterises the softening phase. It indicates that the development of swelling pressure is relatively easy and fast. According to the induction concept, swelling pressure (inter-particle repulsion) is a product of either soil attraction or soil cohesion. In this instance, the development of swelling pressure is reflecting the mobilisation of soil cohesion, since soil attraction is decreasing. This was explained as follows.

The relative ease of mobilisation of soil cohesion is a consequence of soil destructuration. Soil destructuration increases the number of elementary particles. This leads to rapid hydration and hence an increase in soil cohesion. In essence, the internal effect of soil attraction is considered transferred to the adsorbed water layers, hence soil cohesion. As expected the stress resistance is a maximum at point B, when soil cohesion is a maximum.

5.2.3 Stiffening phase

Point B marks a dramatic change in the stress resistance from increasing to decreasing. The negative stress number (m_s = -3.2) indicates that swelling pressure (and hence soil cohesion) development is much slower than before. This observation can be explained by postulating the onset of the induction phenomenon at point B. That is, soil attraction is induced at point B and suppresses the effect of soil cohesion. In a sense, the stress is transferred from the external (adsorbed water layers) to being internal. Since soil attraction is an ‘internal’ effect, it can not be directly measured externally, which explains the apparent decrease in stress resistance. However, in reality the soil becomes stiff as discussed in the soil resistance analysis.

5.2.4 Peak phase (C-C’)

The peak phase (C-C’) is characterised by no stress resistance (m_s = 0). This shows that the process responsible for developing stress has stopped. This points to the fact that the swelling process is completed.

Validation of the induction concept

Most vivid is the symmetry of the stress resistance mobilisation about point B, with respect to both the stress numbers and the stress resistance. It shows that the post-SBS stiffening process completely reverses the effects of the pre-SBS softening phase. Accordingly, induction phenomenon takes place at point B and it restores the soil attraction lost in the softening phase.

Suction-gradient wetting path imposes no external force on the flow of water. Therefore, water flow is in response to the soil water demand. Once the water demand is satisfied, the flow stops and equilibrium is established. This indicates that the initial water demand of the soil is equivalent to the maximum swelling pressure. Since there is no external interference on the swelling process, then the downloading of the internal resistance directly leads to the development of swelling pressure. Thus, the reversal is complete and contained. Therefore, the induction concept is completely vindicated.

5.3 Swelling resistance analysis: confined swelling

In this series of tests, water was rapidly introduced to the soil sample through the base, under a very high-pressure gradient until it flooded the top of the sample. The top end of the sample was kept covered with water. The wetting path was called flooding.

The swelling process is predominantly rheological and it is characterised by a build-up of swelling pressure. Therefore, a time related resistance was defined, which relates the building up of swelling pressure with the passage of time. It is called swelling resistance and was defined in section 3.4.2 of chapter three. The analysis was conveniently carried out in terms of the ***swelling number, y***. The bigger the swelling number the slower is the swelling process. The analysis sought to characterise the swelling process in the context of the induction concept.

The results obtained along the flooding path were analysed. A typical test result for sample no. SP-02 is plotted in fig. 5.5. The sample had initial water content of 19 per cent. Reference is made to fig. 5.6 and the summary of results in table 5.4.

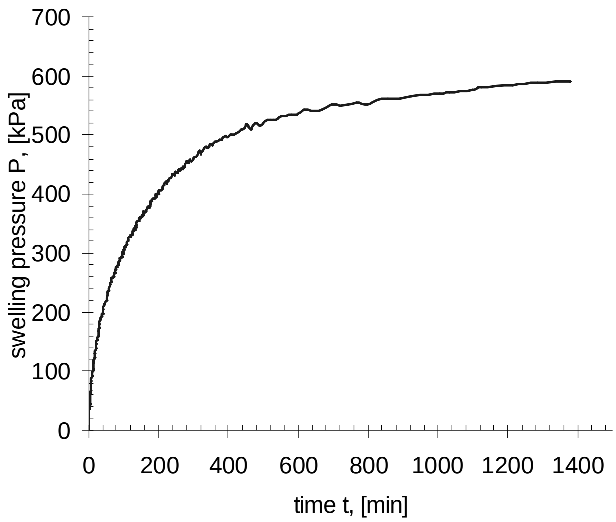

Figure 5.5 Mobilisation of swelling pressure along flooding path

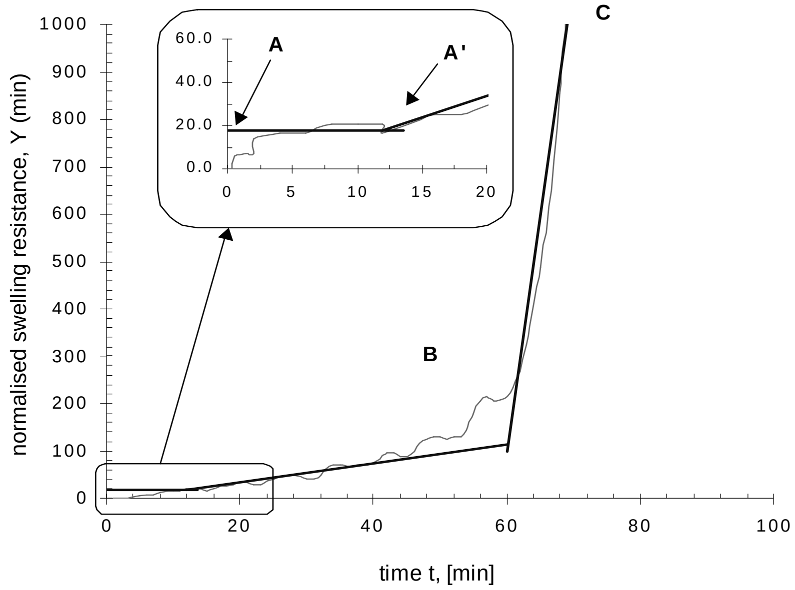

Figure 5.6 Variation of the swelling modulus with passage of time

5.3.1 General

The swelling resistance analysis shows that the soil develops an initial resistance of Y = 18 minutes. This is analogous to the initial tangent modulus in the compression of saturated soils (Janbu, 1963, 1986). The tangent modulus was discussed in section 3.4.1 of chapter three. This observation is significant to the swelling process as discussed next.

Table 5.4 Summary of swelling resistance analysis: flooding path

phase	***swelling number, y***	***period (%)***	***pressure increase (%)***
A - A’	0	2.4	16
A’- B	+2	9.6	30
B - C	+100	88	54
C - C’	+ infinite	(120)*	0

* The period is based on the swelling time from point A to point C.

5.3.2 Initial phase (A-A')

The swelling process proceeds at a constant rate (y = 0) for 2.4 per cent of the swelling period. Most significant is the presence of initial swelling modulus (Y = 18 min.). This is explained by the presence of more excess water in the pore space of the soil as follows.

Flooding makes more water available to the soil in a short period. Hydration is accelerated such that the adsorbed water layers grow relatively fast. Now, the soil particles push each other as they rearrange to take up more space. In the presence of pore water, they exert considerable force to the pore water, which manifests as excess pore water pressure. The pore water is used up to mobile soil cohesion. However, the mobilisation process involves pulling the hydrated clay minerals together, thereby exerting more pressure on the residual pore water. Thus, equilibrium is established, where the effect of reduced pore water on the excess pore water pressure is compensated by the mobilised soil cohesion. Therefore, the initial swelling resistance is due to the mobilisation of soil cohesion, which causes the mobilisation of pore water pressure. In some way, the internal effect of soil attraction is downloaded and carried as pore water pressure, before it is transferred to the adsorbed water, where it manifests as soil cohesion. This reasoning is borrowed from the analogy with primary consolidation of saturated soils. As such, the swelling resistance remains constant up to point A', where soil cohesion mobilises directly and dominates the soil response.

5.3.3 Softening phase (A'-B)

The softening phase is marked by a significant increase in the swelling modulus (y = +2). The increase is attributed to the direct mobilisation of soil cohesion, which starts at point A'. The rate of increase is steady because of the continuing dynamic equilibrium between depletion of pressed pore water and mobilisation of soil cohesion. The dynamic equilibrium stops at point B, when the pore water is depleted and the soil attains maximum plasticity. The 9.6 per cent swelling period indicate that more time is required to dissipate the excess water pressure. Conversely, it takes a while for the 'load' carried by the pore water to be transferred to the adsorbed water layers.

5.3.4 Stiffening phase (B-C)

The stiffening phase is marked by a ten-fold increase in the swelling modulus. The sudden increase of the modulus at B confirms the onset of the induction phenomenon, as already discussed. However, the very large swelling number (y = +100) shows that the induction process is very slow. The 88 per cent swelling period confirm this. The induction phenomenon is slow because it acts against its product namely, interparticle repulsion. That is, the hydrated clay minerals that are squeezed together by soil attraction have the same polarity on their surface, which generate repulsion (osmotic potential). Thus, the process takes a relatively long time.

5.3.5 Peak phase (C-C')

At point C, the swelling number shoots up towards infinite, indicating that the swelling process is proceeding extremely slowly. In reality, it means the end of the swelling process.

Validation of the induction phenomenon

The sudden increase of the swelling modulus at point B confirms the onset of a phenomenon that is independent of the hydration of water. It is the induction of soil attraction. Thus, the induction phenomenon has been independently validated for the third time.

5.4 Soil & stress resistance analysis: continuous swelling path

In this series of tests, an unsaturated soil sample was allowed to freely swell in the vertical direction. Water at a constant head (pressure gradient wetting path) was added to the unsaturated soil sample at the base using an electronic burette.

The analysis sought three things. Firstly, to validate the induction phenomenon in a more realistic situation, when all the soil variables are acting. The variables are swelling pressure, water-flow, volume-change and soil-structure. Secondly, to demonstrate the rationality of the SBS physical soil model. Lastly, to determine the soil parameters that can be used to model the soil behaviour. The analyses were carried out in terms of stress and soil resistances. This time the analyses were combined in order to understand the coupled soil response. Use of the resistance concept and specifically the dimensionless numbers (soil and stress numbers) greatly simplified the analysis. The smaller the soil number the easier is the change in soil structure, while a small stress number signifies a slow build up of swelling pressure.

The effective stress terms of the soil potentials that were proposed in the hypothesis presented in section 2.4 of chapter two, were used in the analysis in place of the soil potentials. This facilitated an understanding of the soil response in the context of soil mechanics terminology. The effective stress hypothesis will be validated in chapter six.

The two forms of effective stress are internal effective stress (soil attraction) and mobilised internal effective stress (soil cohesion). In soil mechanics terms, hydration downloads the internal effective stress (soil attraction) and transfers it to the adsorbed water layers, where it manifests as mobilised effective stress (soil cohesion). In the presence of pore water, the build up of excess pore water pressure is considered analogous to primary consolidation of saturated soils. That is, the downloaded (internal) effective stress is initially carried by the water phase as pore water pressure. It is then gradually transferred to become mobilised effective stress, with the accompanying dissipation of the excess pore water pressure. The analysis referred to effective stress and soil potentials interchangeably to suit the context of the discussion.

Results from a soil sample (sample no. FSL-021) were analysed. The soil sample had initial water content of 10.8 per cent. The soil and stress resistance results are plotted with respect to water content in figs. 5.7 and 5.8 respectively, and the pertinent information is summarised in table 5.5.

5.4.1 General

It is clear from the figures that the previously defined swelling phases do not directly correspond with the soil and stress numbers, a consequence of the coupled soil response. Nevertheless, the validity of the previous analyses is vindicated in the subsequent sections. For the sack of consistency, the previously defined phases were used in the analysis. The consistency limits were assigned to the arbitrary water content points in line with the induction concept. The rationality of this assignment is further demonstrated herein.

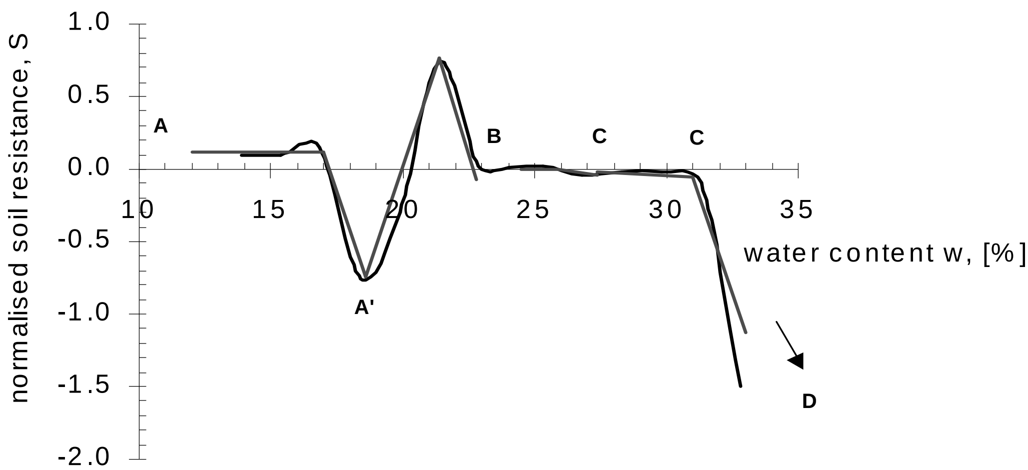

Figure 5.7 Mobilisation of soil resistance during continuous swelling

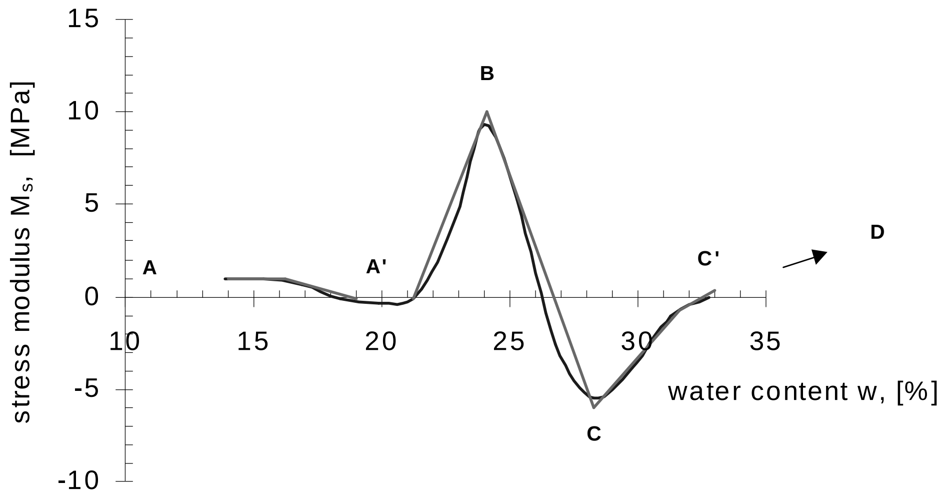

Figure 5.8 Mobilisation of stress resistance during continuous swelling

Table 5.5 Summary of soil resistance analysis data: pressure wetting-path

Phase	***soil number, s***	***stress number, m_s***	***water content (%)***
A-A’	0	0	10-14
	-0.54	-0.4	14-18.5
A’-B	+0.54	+3.5	18.5-21.5
	-0.54		21.5-23.5
B-C	-0.027	-3.8	23.5-27.5
C-C’	-0.09	+1.2	27.5-32.5
C’-D	-0.54	-	32.5--

5.4.2 Initial phase (A-A’)

Both figures 5.7 and 5.8 do not register resistance between 10 and 14 per cent water content. This is because the pore water pressure is below atmospheric value, the reference pressure. The initial constant soil resistance from 14 to 16.5 per cent (fig. 5.7) shows that the excess pore-water pressure compensates for the rapidly unloaded internal stress. The rapidly unloaded internal effective stress is transferred to the pore water and manifests as pore-water pressure. Similarly, the stress resistance (fig. 5.8) is constant up to 16.5 per cent, at which particle soil destructuration sets in. The changes in the soil and stress resistance are in accordance with the analogy with primary consolidation settlement.

At 16.5 per cent water content, the particles are sufficiently swollen to start mobilising soil cohesion, leading to 1^{st}-level soil destructuration. Soil destructuration destroys the initial soil structure and reduces soil resistance. This explains the rapid and significant decrease of the normalised soil resistance (s = -0.54). At the same time, the stress resistance decreases slowly, in apparent response to the dissipation of the excess pore-water pressure. Fig. 5.8 shows that the stress modulus becomes zero at 18 per cent water content. This confirms the earlier observation that the destructuration process effectively depletes the excess pore-water pressure. In other words, the effective stress carried by the pore water is reloaded to the soil as soil cohesion. First level soil destructuration completes at 19 per cent water content (point A'), when the soil resistance is reduced to –0.78 per kPa per atmosphere of the reference value. Now 19 per cent is the shrinkage limit of the investigated soil. Therefore, ***pint A' is the shrinkage limit as asserted in the induction concept.***

5.4.3 Softening phase (A'-B)

At point A', the stiffening effect of soil cohesion causes the soil resistance to increase. At the same time, soil cohesion causes excess pore water pressure to build up again. The equal but opposite soil number (s = +0.54) indicates that pore water pressure is building up at the same rate that it dissipated during 1^{st}-level soil destructuration. However, the stress resistance remains zero in this period. Therefore, the build-up of pore water pressure implied in the stage of (s = +0.54) is below atmospheric value. At 20 per cent water content, the stress resistance starts to increase very fast (m_s = +3.5). It means that the swelling pressure is increasing very fast. Therefore, the swelling pressure is due to build up of pore water pressure excess of atmospheric. The speedy rise must be due to soil cohesion, which mobilises rapidly in accordance with the induction concept. Accordingly, the changes in the stress and soil resistances are due to the increase of the mobilised effective stress.

Maximum soil resistance is mobilised at 21.5 per cent (S = +0.78 per kPa per atmosphere). The maximum value is equal and opposite in magnitude to the minimum soil resistance. Since minimum soil resistance is attained when the excess pore water pressure is a minimum, it is reasonable to say that maximum soil resistance corresponds to the instance of maximum excess pore-water pressure. Therefore, the subsequent decrease in soil resistance must be due to the onset of 2nd-level soil destructuration. It is simultaneous with the depletion of the excess pore water. Meanwhile, the stress modulus continues to increase. Two things emerge from these observations. Firstly, soil destructuration occurs when the excess pore water pressure has increased to a maximum value. A similar occurrence was observed before the onset of 1st-level soil destructuration. Accordingly, the zero stress modulus indicates that the pore water pressure is equal to atmospheric value. Secondly, the continuous and smooth increase in stress modulus shows that the effective stress that is carried as excess pore water pressure is smoothly transferred to the adsorbed water phase, leading to rapid mobilisation of soil cohesion. That is, the change over of swelling pressure from being excess pore water pressure to being direct interparticle repulsion is smooth. This then explains the continued increase in stress modulus as if it is coming from a continuous build up of excess pore water pressure. The soil number of (s = -0.54) during 2nd-level soil destructuration shows that the rate of soil destructuration is the same at both levels.

The soil resistance decreases to zero at 24 per cent water content (point B). It means that 2nd-level soil destructuration is completed. This is synonymous with mobilisation of maximum soil cohesion. The soil has zero soil resistance, indicating a restoration of the initial (reference) soil resistance. This confirms that the mobilisation or external storage of the internal effective stress as soil cohesion is complete. Thereafter, soil attraction causes soil compression. Therefore, the soil resistance at point B is zero and it confirms the balance between compression and swelling. That is, the soil is at atmospheric pressure. Noting that the plastic limit of the investigated soil is 24 per cent validates this point. Therefore, ***point B is the plastic limit as asserted in the induction concept***.

Soil parameter

The soil number, s = +0.54 is a measure of the overall soil response. It is considered to be a compound of pore water pressure and loss of soil attraction. Therefore, the soil parameters in terms of the two forms of effective stress need to be separated. This was done and is discussed in chapter nine.

5.4.4 Stiffening phase (B-C)

The mobilisation of maximum soil cohesion at point B induces soil attraction. According to the change in stress resistance (fig. 5.8), the mobilisation of swelling pressure is very slow (m_s = -3.8). This concurs with a similar observation made in section 5.3.4 above.

It is significant to note that the stress resistance is zero at 26.5 per cent water content, at which point the soil resistance decreases slightly. A decrease in soil resistance is associated with dissipation of excess pore water pressure. According to the induction concept, soil attraction squeezes loosely bound water out of the adsorbed water. As such, the water builds excess pore water pressure. It follows that the excess water-pressure that builds-up from 24 per cent dissipates at 26.5 per cent. This is considered as unloading of residual internal effective stress, analogous to time-dependent consolidation settlement. At that point, the stress resistance is zero and becomes negative thereafter. Therefore, the zero stress-modulus denotes the dissipation of excess pore-water pressure. In addition, the development of negative stress modulus intimates the absence and hence complete depletion of liquid water in the soil. It then follows that positive stress resistance is due to excess pore-water pressure, while negative stress resistance is due to direct interparticle repulsion-related pressure. The slight decrease in the soil resistance below the axis confirms that particle interaction is direct.

Maximum soil attraction re-mobilises at point C, corresponding to 27.5 per cent water content. At the same time, the soil resistance in the phase (C-C’) is very close to the reference value, which is the initial soil resistance. This points to the restoration of the initial soil condition, in accordance with the induction concept (fig. 2.2 of chapter 2). Being the intrinsic property of the soil skeleton, it is reasonable to say that soil attraction is the seat of the initial soil resistance. These observations concur with the previous analyses and vindicate the induction concept.

Soil parameters

The determined soil number during this phase (B-C), s = -0.027 is influenced by the induction phenomenon. Accordingly, the value is a compound value. The correct value of the soil number that reflects the mobilisation of soil attraction is determined in sub-section 9.3.2 of chapter nine.

5.4.5 Peak phase (C-C')

The increase in stress resistance from point C is indicative of increasing water pressure. Being the reverse of the pre-peak behaviour, the response shows that soil attraction is demobilising. The stress number (m_s = +1.2) is sufficiently close to one to confirm that the change in swelling pressure and hence soil attraction, is directly proportional to the increase in water content. However, the soil resistance is practically constant up to point C'. This is because the increasing interparticle distances are still within the molecular range of the clay minerals. Significant decrease in soil attraction comes about, when the interparticle distance is equal to or greater than the molecular range. Thus, the soil resistance is constant up to point C', at which point it rapidly decreases.

The stress modulus at point C' is zero, implying zero swelling pressure. Since the water pressure is on the increase, it follows that the zero swelling pressure is with respect to the restoration of atmospheric conditions in the water phase. In other words, the removal of soil attraction has relieved the diffuse double layer of confinement sufficiently for it to partly exist as liquid water. Point C' corresponds to the water content, beyond which the free water layers between the hydrated clay minerals are sufficiently thick to start offsetting soil attraction. Since attraction is a short-range force, it is offset and it rapidly decreases. The condition registers as a yield hence a sudden decrease of the soil resistance. The soil yields in that it loses its mobilised stiffness. From the two figures, the average water content for point C' is 32.5 per cent. Therefore, ***32.5 per cent water content (point C') is the yield point*** of the soil.

Soil parameters

The soil number for phase (C-C'), s = -0.09 is also considered a compound number

5.4.6 Drift phase (C'-D)

According to the SBS physical soil model (chapter two), the change in soil structure beyond point C' is modelled as a drift of the SBS. As such this phase called the ***drift phase***.

A rapid loss of soil resistance characterises the drift phase. The rate of change of soil resistance (s = -0.54) is equal to that of soil destructuration. This is the case because in both cases, the removal of soil attraction underlies the soil response. It is a reversal of the induction phenomenon. According to the induction concept, soil attraction is completely removed at point D and soil cohesion is fully restored. Therefore, the soil resistance is expected to decrease at a rate of (s = -0.54), up to point D. The soil condition at point D is similar to that at point B, where the water pressure is atmospheric. Therefore, the soil at point D is at atmospheric pressure. Since the soil is already saturated, it follows that ***point D is the atmospheric saturation point***. This is in accordance with the dynamic soil structure model (sub-section 2.3.2 of chapter two).

Soil parameters

The soil number for this phase, s = -0.54 is the correct soil parameter because soil attraction is being removed and is hence considered passive.

5.4.7 Final phase (D-F)

According to the induction concept, stage (D-F) is the final phase, when soil cohesion is removed from the soil. The swelling pressure at point D is atmospheric and can not physically decrease below the atmospheric value. Therefore, the swelling phenomenon is subsequently devoted to removing soil cohesion from the soil, at atmospheric conditions. It is reasonable to say that the removal of soil cohesion is non-linear because it is a reversal of the effects of an electrochemical process. The determination of the appropriate soil number is discussed in section 9.3 of chapter nine.

Soil parameter

The soil parameters for phase (D-F) were not determined directly from data. This is because the laboratory tests could not be run far beyond point D. This response is discussed in chapters seven to nine. However, it is shown in chapter nine how to determine the soil parameter.

5.5 Conclusions

The significant conclusions from this chapter are:

- The results analysed in this chapter validated the induction concept and rationalised the Swelling Boundary Surface (SBS) physical soil model.
- The physical significance of the arbitrary water content points is that they are the consistency limits. Thus, the induction concept rationalised the consistency limits.
- The analysis along the continuous swelling path successfully characterised the dynamics of the swelling process. The analysis handled the simultaneous changes in swelling pressure, soil structure and water content without volume control.
- The swelling process is completed at the onset of maximum swelling pressure
- The development and motion of the SBS is central to the swelling process. Thus, the proposed SBS physical soil model correctly characterises the swelling behaviour of the investigated soil.
- Water flow in the investigated unsaturated expansive soil is predominantly by diffusion during and after the swelling process.

Chapter 6

Validation of the effective stress hypotheses

The hypotheses postulated in chapter two are validated in this chapter. The analysed test results sought to demonstrate that swelling pressure is isotropic and that soil attraction and soil cohesion are the internal and mobilised effective stresses, respectively. The chapter concludes by presenting a practical way of integrating the two forms of effective stress as a continuous function of water content. The work presented herein is original.

6.1 The nature of swelling pressure

The confined swelling test results were analysed with the view of showing that swelling pressure is isotropic.

6.1.1 Hypothesis 1

> Swelling pressure is ISOTROPIC. For undisturbed soil-samples tested in the laboratory, the two necessary and sufficient conditions that must be satisfied are (1) the measured swelling pressure is independent of the direction in which the samples are sampled and tested. (2) The swelling pressures measured in any two perpendicular directions are equal for a sample with the same dimensions in the respective directions. Conversely, the ratio of the swelling pressures in any two perpendicular directions is equal to the ratio of the surface areas in the corresponding directions.

6.1.2 Experimental test results

Two confined swelling tests were carried out to verify the hypothesis. One test was on an undisturbed soil sample that was sampled *in situ*, in the horizontal direction. The second test was on a remoulded soil sample, compacted at Low Compactive Effort (LCE) and optimum moisture content. In both tests, the swelling pressure was measured in the vertical and horizontal directions. The test results obtained are presented in figs. 6.1 and 6.2.

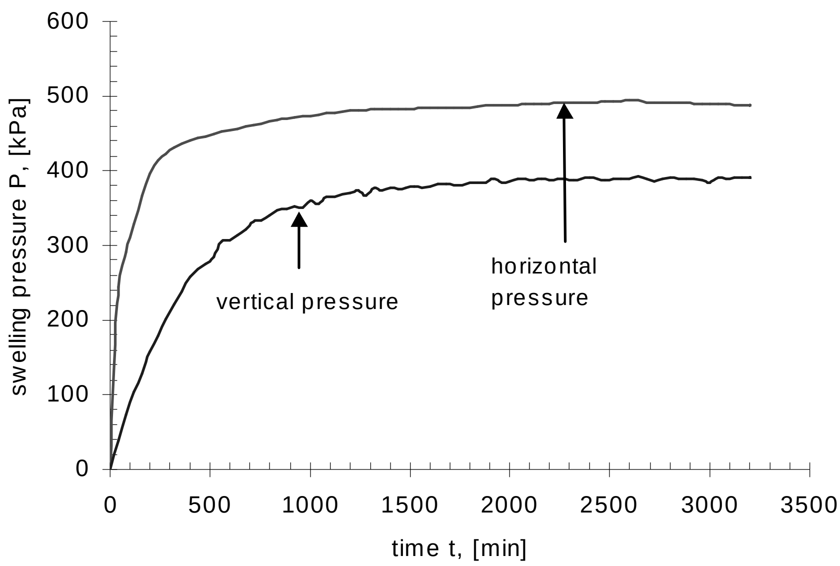

Figure 6.1 Mobilised swelling pressure for a horizontally sampled sample

<u>Condition 1:</u>

The results in figs. 6.1 and 6.2 show that the swelling pressure mobilised in the horizontal direction is higher than that in the vertical direction as previously observed in the case of a vertically sampled sample in fig. 4.2 of chapter four. Consequently, the observed anisotropy is not related to the arrangement of the soil fabric. Therefore, condition 1 of the hypothesis is satisfied by elimination.

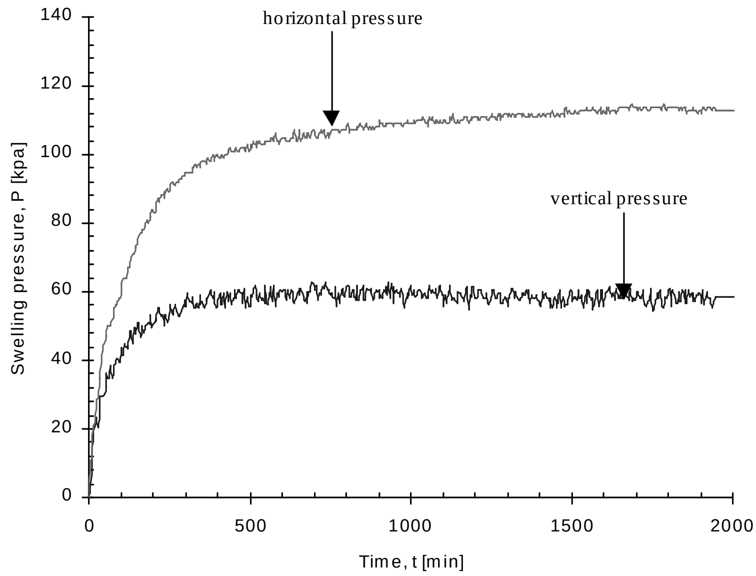

Figure 6.2 Mobilised swelling pressure for a sample compacted at LCE

Condition 2:

The test results for the horizontally sampled sample are re-plotted in fig. 6.3, in the form of variation of the swelling pressure ratio (P_h, P_v) with time. The horizontal dotted continuous line indicates the ratio of the surface areas of the sample in the directions corresponding to the measured swelling pressure, respectively.

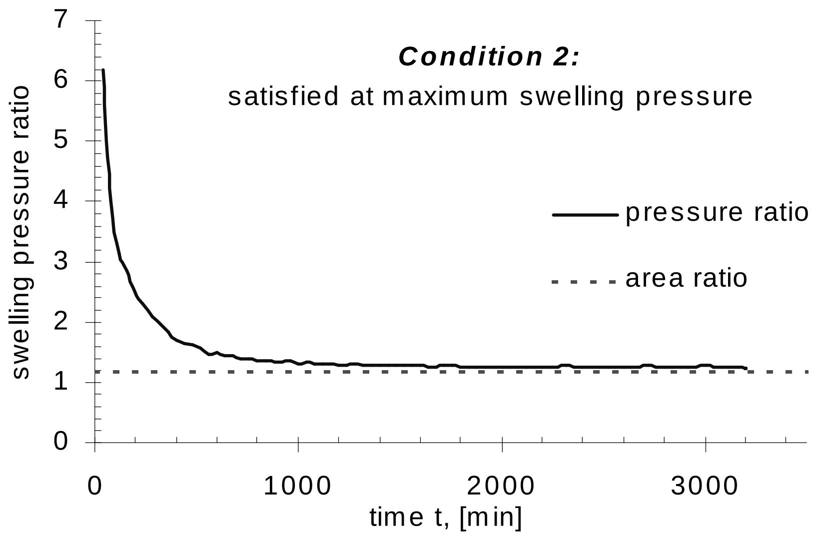

Figure 6.3 Variation of swelling pressure ratio with time

The swelling pressure ratio decreases with time, becoming equal to the ratio of the corresponding surface areas when the pressures are a maximum. This observation intimates that the swelling pressure only becomes isotropic when it attains maximum value. Therefore, condition (2) of the hypothesis is satisfied at the peak stage.

6.1.2.1 Swelling pressure is isotropic

The term ‘isotropy’ and ‘anisotropy’ are related to non-transient, steady state soil condition. Now the swelling process is transient. As established by the resistance analysis, the process is completed with the attainment of maximum swelling pressure. In other words, the peak phase is non-transient. For that reason, condition (2) of the hypothesis is satisfied at the peak stage, when steady state condition prevails. It then follows that the observed anisotropy is a reflection of the mobilisation process and not the nature of swelling pressure.

The satisfaction of condition (2) at peak pressure is significant to the swelling process. It shows that the onset of maximum swelling pressure coincides with the start of non-transient soil conditions. Therefore, the swelling process is completed when maximum swelling pressure is mobilised. In addition, it confirms the hypothesis that swelling pressure is isotropic. Therefore, ***swelling pressure is isotropic***.

6.1.3 The anisotropy in swelling pressure

The results that support hypothesis 1 indicate that the apparent anisotropy of swelling pressure is a consequence of the mobilisation process. This assertion is verified by testing a hypothesis to that effect. A hypothesis is proposed and experimentally verified as follows.

6.1.3.1 Hypothesis 2

When an expansive soil swells naturally, it mobilises swelling strain such that the resultant component of swelling pressure in any direction is the true isotropic swelling pressure. Under conditions of soil confinement, the isotropic swelling pressure decomposes such that its components act perpendicular to the direction of confinement. However, the isotropic value is the sum of the swelling pressure components. For a sample confined in the vertical and horizontal directions, the isotropic swelling pressure is computed from equation 6.2. Equation 6.2 is an expression of the mean normal stress and can be used to compute isotropic water pressure. It holds in terms of both effective and total stresses.

$$\textit{Isotropic swelling pressure, } P = \frac{1}{3}(P_v + 2P_H) \qquad \text{(Equation 6.11)}$$

In equation 6.2, P_v is the vertical swelling pressure component and P_H is the horizontal swelling pressure component. The necessary and sufficient condition that must be satisfied is (1) The swelling pressure obtained from equation 6.2 for conditions of complete soil confinement, is equal to the confined swelling pressure measured during unconfined swelling.

6.1.3.2 Experimental test and results

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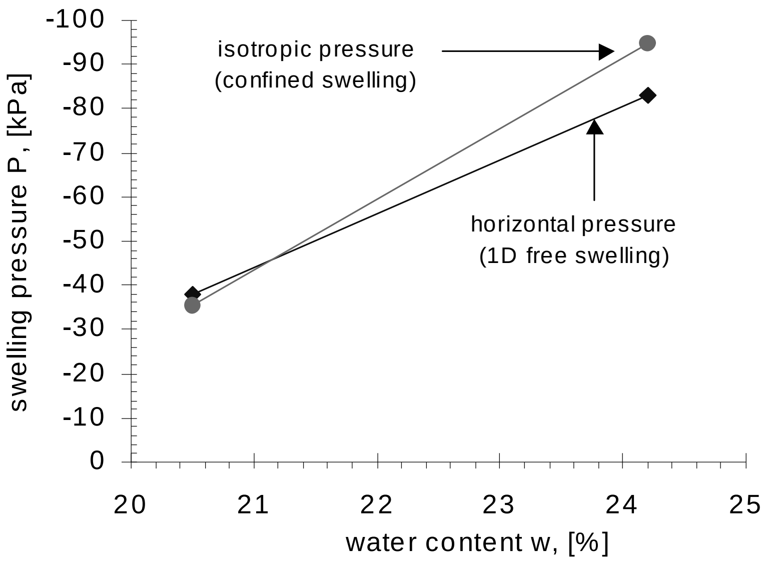

Figure 6.4 Comparison of swelling pressure measured during confined and unconfined suction-swelling

Reference is made to the swelling pressure–water content plots in fig. 6.4. The swelling pressure data is obtained from confined and laterally confined swelling tests. In both cases, the samples were swelled along suction wetting path, which is the natural swelling path of the soil. For confined swelling, the swelling pressure was measured in the vertical and horizontal directions. The sum of the two swelling pressure values was computed using equation 6.2. In the case of laterally confined swelling, the soil sample swelled in the vertical direction only, while swelling pressure was measured in the horizontal direction. Two water content points have been selected for comparison. The first point (20.3 per cent) is within the viscous soil-response phase, when the SBS evolves. It reflects the condition of interaction between depleting pore water and increasing soil cohesion (stress conversion phase). The second point (24.3 per cent) is in the post-SBS phase. The pore water is depleted, soil cohesion is fully mobilised and the induction of soil attraction is in progress.

6.1.3.3 Swelling process is isotropic

The results show remarkable agreement. The significant difference between the points at 24.4 per cent water content can be explained as follows. Soil confinement causes the swelling process to proceed in the vapour phase. The swelling pressure increases faster than the increase in water content. Consequently, the swelling pressure is not directly related to the current water content. Accordingly, the (calculated) isotropic pressure plots above the horizontal swelling pressure at 24.4 per cent. Therefore, the results confirm hypotheses 2 and vindicate hypothesis 1.

Apart from this, the results from one-dimensional free swelling show that the swelling pressure does not dissipate immediately, but is stored within the soil. Accordingly, this confirms that the SBS ‘stores’ the swelling pressure as proposed by the physical soil model.

6.1.4 Nature of swelling pressure

Adsorbed water can best be visualised as composed of water molecules, which are relatively free to move in the two directions parallel to the clay surface but are restricted in the movement perpendicular to or away from, the surface (Yong and Warkentin, 1975). The lateral movement is possible because of the transfer of molecules from one bonded position to another. Now, being a natural process, the swelling process mobilises swelling strain such that the swelling pressure measured in any direction is isotropic. However, where the soil is not free to change its structure in response to water flow, the isotropic swelling pressure is decomposed. Soil confinement cause the mobilised swelling pressure to decompose into perpendicular components, with the major component acting perpendicular to the direction of water flow. Accordingly, the observed anisotropy comes from the confining effect of the test procedure, which interferes with an otherwise natural process. Therefore, ***swelling pressure is isotropic***.

This conclusion is consistent with the origins of swelling pressure. It arises from the repulsion of the dissociated hydrated ions surrounding the hydrated surface in the diffuse double layer. Therefore, swelling pressure is fundamentally borne out of water. Accordingly, it is a water-related pressure and hence isotropic. This agrees well with the observation made that the transition of swelling from being pore water pressure to being osmotic (repulsion) pressure at point B is smooth. Accordingly, the swelling pressure measured in any direction, when the sample can freely swell, is isotropic pressure. It acts in all directions. However, unlike the hydrostatic pressure, it varies with change in water content. Therefore, the character of swelling pressure is borne out of its mobilisation.

6.1.4.1 Character of swelling pressure

The mobilisation of swelling pressure involves coupling between water flow and change in soil structure. The mode of swelling pressure development changes during the swelling process. Initially the repulsion is indirect because of the presence of pore water. Accordingly, swelling pressure manifests as pore-water pressure. However, the pore water dissipates at 24 per cent and the interparticle repulsion becomes direct. Thereafter, the resultant pressure arises from the repulsion of the dissociated hydrogen ions surrounding the hydrated clay mineral surfaces. Consequently, the manifestation of swelling pressure changes during the swelling process. Ultimately, the pressure is removed during post-swelling process. Therefore, swelling pressure is transient.

The term “swelling pressure” does not completely reflect the nature and character of the pressure for three reasons. Firstly, the soil response is swelling during the stress-unloading phase, for which the period is ‘instantaneously small (less than five per cent). For the greater part of the swelling process, the soil is ‘compressed’ under the mobilisation of soil attraction. In this phase, the swelling pressure is due to the induced soil attraction, whose mobilisation is compressive as opposed to swelling. Secondly, it is a water pressure and should be rightly referred to as such. Lastly, the clay minerals remain swollen, well after the swelling pressure has dissipated. As it stands, its name is akin to referring the excess pore water pressure generated by loading saturated soils, as “consolidation pressure”. Because it is transient and it is water-pressure, swelling pressure is a hydrotransient pressure. Therefore, the swelling pressure is herein called ***hydrotransient pressure***.

6.2 Soil attraction is the internal effective stress

In accordance with the proposed induction concept, soil attraction the internal effective stress of an expansive soil. This proposition is consistent with the fact that soil attraction is inter-clay mineral (intergranular). It also agrees with the Mitchell’s (1993) generalisation of the effective stress principle and Janbu’s (1963) definition of soil attraction as an effective stress for saturated soils. . It is therefore, necessary to validate this experimentally.

6.2.1 Hypothesis 2

Soil attraction is the internal effective stress of an unsaturated expansive soil. It is the intrinsic property of the soil and it keeps the solid particles together. Soil attraction attracts and presses the clay minerals together and is therefore responsible for the ‘inter-granular’ stress. It is characteristic of the clay mineral solids and is responsible for the shear strength of the soil. The necessary and sufficient conditions that must be satisfied are (1) the swelling process is a shearing process. (2) The mobilisation of the internal shear strength of the soil is simultaneous with the mobilisation of soil attraction and, (3) the mobilisation of the soil stiffness is mechanical and directly related to the mobilisation of soil attraction.

6.2.2 Experimental test results

Condition 1- the swelling process is a shearing process

Swelling pressure is a product of the change in the internal stress and water flow. The process is primarily rheological. Consequently, the mobilisation of swelling pressure is the appropriate measure of the change in the internal stress of the soil. The passage of time is also a qualitative measure of water content change and hence internal stress. Therefore, the swelling process is studied in the form of the mobilisation of swelling pressure with respect to time. The swelling process is expressed as a ratio of the vertical and horizontal components of swelling pressure (P_v / P_H), measured during confined swelling. The swelling pressure ratios along the two investigated wetting paths are plotted against time in fig. 6.5.

The initial high rate of swelling experienced along the flooding path (fig. 6.5), is a result of the rapid downloading of the internal stress, which manifests as excess pore water pressure. By inserting a projection on the flooding path back to zero, fig. 6.5 shows that the mobilisation of swelling pressure and hence change in the internal stress is analogous to the undrained triaxial compression of saturated soils (both sands and clays). The undrained triaxial compression of sands and clays are illustrated in fig. 6.6 and the analogy is summarised in table 6.1.

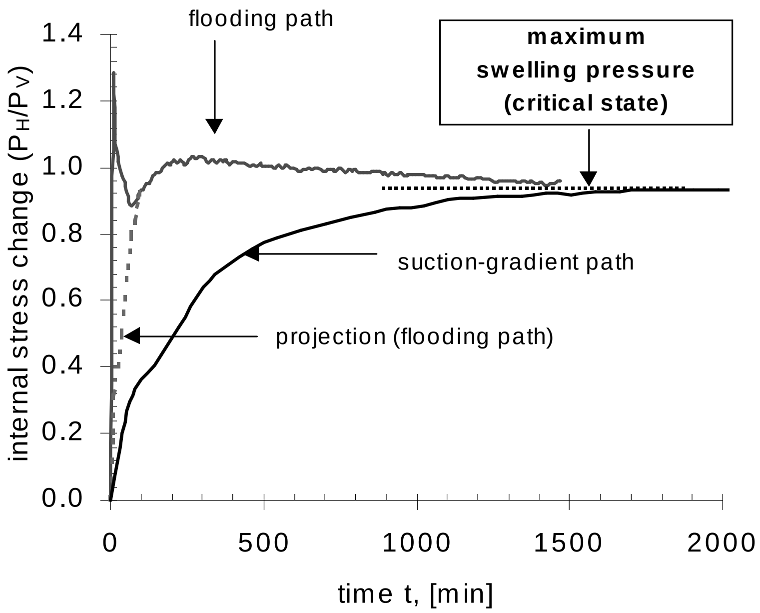

Figure 6.5 Change of the internal stress of the soil with time.

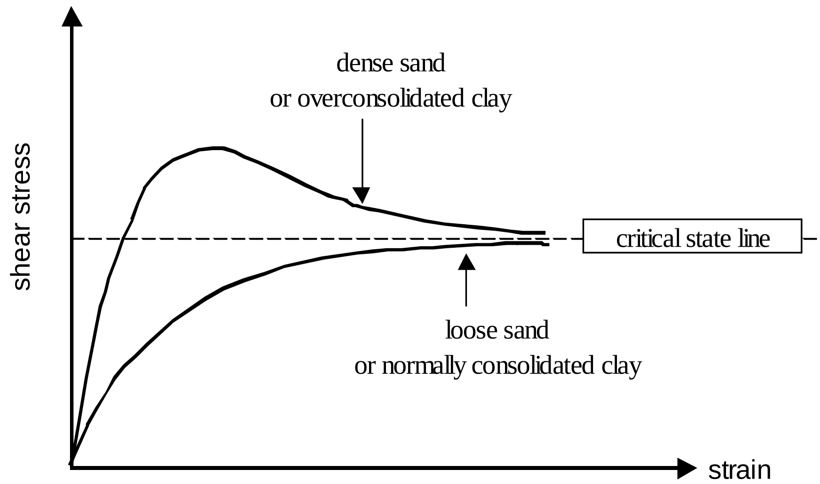

Figure 6.6 Illustration of undrained triaxial compression of saturated sands and clays.

Suction swelling is analogous to compression of normally consolidated soils, while flooding relates to the behaviour of over-consolidated soils. The different wetting-paths essentially reflect the swelling history of the soil, much the same as density and degree of consolidation reflect the stressing history of sands and clays, respectively. The analogy intimates that the swelling process is a mechanical shearing process and that the wetting paths are stress paths. In addition, the change in the internal stress is predominantly a stress mobilisation process. These observations vindicate the analyses of the experimental results in chapter four, which are with respect to the SBS physical model.

Table 6.1 Comparison between undrained compression and swelling process

phenomenon	***compression of clays***	***compression of sands***	***soil swelling***
Loading history or Stress path	Normally consolidated	Loose sand	Suction-gradient
	Over-consolidated	Dense sand	Flooding

The two different wetting paths in fig. 6.5 converge to a common internal stress value, when the swelling pressure is a maximum. Accordingly, attainment of maximum swelling pressure is not path dependent. Therefore, the analogy with triaxial compression of saturated soils can be extended further. It shows that the mobilisation of the internal stress similarly attains a critical state. The condition of maximum swelling pressure is critical to the soil in the sense that the swelling process is completed then. It then follows that maximum swelling-pressure defines a condition of critical shear strength of the soil. Meanwhile, swelling pressure does not mobilise shear strength because it is essentially water pressure and water does not have shear resistance. Therefore, the shearing is, with respect to the mobilisation of the internal effective stress, as soil cohesion in accordance with the particle interaction model.

Thus, the analogy brings out two significant aspects of the change in the internal stress of the swelling soil. Firstly, attainment of maximum swelling pressure is critical and confirms the end of the mobilisation process. Therefore, the mobilisation of swelling pressure is uniquely related to the change in the internal stress of the soil. Secondly, the change in internal effective stress is a shearing process, during which the internal stress is mobilised. In addition, it is strongly linked to the mobilisation of soil attraction, which becomes a maximum simultaneously with swelling pressure. Accordingly, the first condition of hypothesis 3 is satisfied. However, it is not sufficient to validate the hypothesis.

Conditions 2 and 3-mobilisation of soil stiffness

The swelling process is effectively time-dependent (rheological). Accordingly, analysis of time-related quantities in the swelling process capsulate the nature and character of the swelling process and hence mobilising of soil stiffness. In this regard, the relevant quantity to consider is the initial swelling modulus, Y_o. It is a resistance to the flow of water in the soil and hence to soil deformation and was obtained as shown in fig. 5.6 of chapter five. Variation of the initial swelling modulus with water content along atmospheric swelling, for the investigated wetting paths, is plotted in fig. 6.7. The values were obtained from an analysis as per section 5.3 of chapter five.

Suction-gradient path leads to a linear increase in the swelling resistance for the water-content range investigated. Pressure-gradient path gives a non-linear increase in resistance up to 24 per cent water content, after which it also becomes linear. For the flooding path, the swelling resistance is constant for water contents below 24 per cent. Above this water content, the resistance linearly increases with water content. Beyond 24 per cent water content, the rate of increase of the swelling resistance along flooding and pressure-gradient paths is the same as that for the suction gradient path.

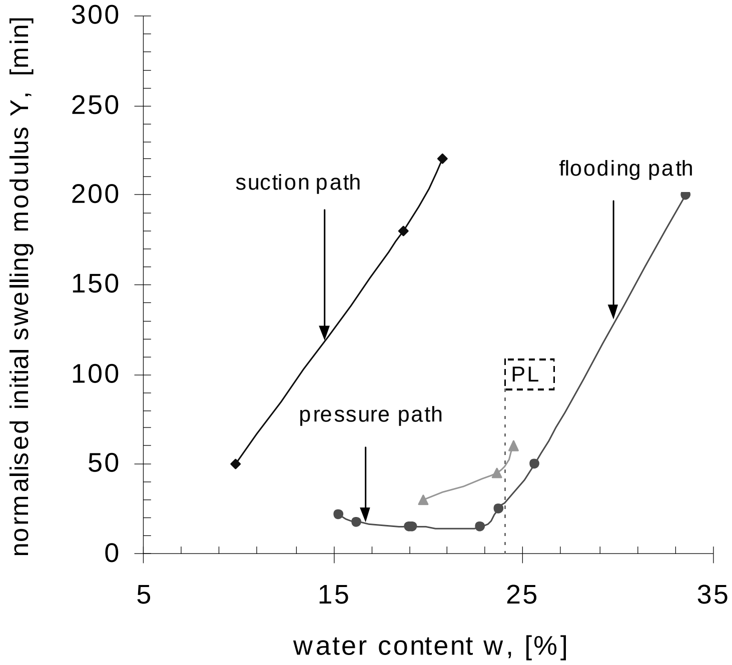

Figure 6.7 Swelling modulus - water content plot along atmospheric swelling

A close look at fig. 6.7 reveals a very surprising but striking similarity to the tangent modulus - effective stress plot for oedometer consolidation of saturated clayey soils (Janbu, 1963). This plot is illustrated in fig.6.8 and the analogous parameters are tabulated in table 6.2.

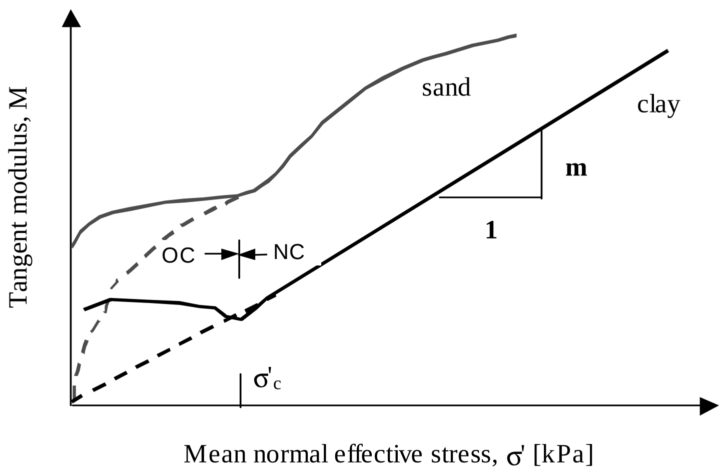

Figure 6.8 Idealised M - σ’ curves (*after Janbu, 1963, 1985, 1998*)

The analogy between water content and effective stress comes as no surprise given that the initial water content is uniquely related to the internal effective stress of the soil, and is therefore a measure of effective stress. Soil swelling along suction-gradient path is similar to oedometer compression of normally consolidated clays, while flooding corresponds to over-consolidated soils. Suction swelling is natural and hence normal-swelling to the expansive soil. On the other hand, flooding forces the swelling process. Thus, the different wetting-paths essentially reflect the swelling history of the soil. A similar parallel was drawn in section 6.2 however, with undrained triaxial compression of saturated soils. The similarity between consolidation and swelling can be extended to the physics of the processes. Both processes involve build up and dissipation of excess pore water pressure, during which the load is temporarily carried by the water phase and then transferred to the soil particles. The initial and significantly large dissipation of excess pore pressure is in both cases completed at the respective controlling points namely, preconsolidation pressure and 24 per cent water content. The respective control points divide both processes into instantaneous- and time-dependent processes.

Table 6.2 Analogy between the swelling process and oedometer consolidation test

phenomenon	***Oedometer consolidation Janbu (1963)***	***Swelling process Mawire (2001)****
Action	Effective stress, σ'	Water content, w
Resistance parameter	Tangent modulus, M (compression resistance)	Swelling modulus, Y (swelling resistance)
Loading history or Stress path	Normally Consolidated Clay (NCC)	Suction-gradient path
	Normally Consolidated Sand (NCS)	Pressure-gradient path (inverted)
	Over-consolidated Clay (OCC)	Flooding wetting path
Controlling point	Preconsolidation pressure, σ'_c	24 per cent water content

* The observations in this thesis

It is significant that for flooding, the soil resistance to swelling rapidly mobilises at 24 per cent. Janbu's (1963) tangent modulus is a stiffness modulus. Therefore, the swelling process mobilises the soil stiffness at 24 per cent. According to the induction concept, SBS physical soil model and the experimental results analysed so far, soil attraction starts to re-mobilise at 24 per cent water content. Therefore, the increase in soil stiffness is simultaneous with the re-mobilisation of soil attraction. At the same time, the only other element of the swelling process is hydrotransient pressure, which can not mobilise shear. Consequently, soil attraction is responsible for the soil stiffness. Since the consolidation process is stressing (loading) the soil, the analogy intimates that the swelling process is predominantly stressing (loading) the soil. This is consistent with the induction concept. Therefore, the swelling process is equivalent to application of effective stress or stiffness of the soil.

Similarly, the identity with the tangent modulus clearly shows that the mobilisation of swelling resistance is a mechanical process. This also agrees well with the mobilisation of soil attraction. Soil attraction is a property of the soil skeleton, whose mobilisation is predominantly a function of interparticle distance. It mobilises when the interparticle distance is within the molecular range, irrespective of the amount of water in the soil. Thus, fig. 6.7 confirms that the re-mobilisation of soil attraction at 24 per cent underlies the sudden increase in soil stiffness. Accordingly, conditions 2 and 3 of hypothesis 3 are satisfied.

6.2.3 Conclusion

Since conditions 1, 2 and 3 were satisfied, it was concluded that soil attraction is the internal effective stress of the investigated soil, as asserted in the induction concept. In view of the validation of the induction concept in chapter five and soil attraction, it follows that soil cohesion is the mobilised effective stress. Therefore, the effective stress of the investigated expansive soil is composed of an internal part (soil attraction) and a mobilised part (soil cohesion), depending on the degree of soil swelling.

6.3 Integration of the internal and mobilised forms of effective stress

6.3.1 Forms of effective stress

The pore water does not mobilise positive hydrostatic pressure during the swelling process. Therefore, the internal stress is always effective. In accordance with the induction concept, the internal effective stress manifests itself in distinct forms. The first form is called ***internal effective stress.*** The internal effective stress is the stress due to soil attraction. It is carried by the solid clay minerals. The second form is called ***converted or mobilised effective stress***. The mobilised effective stress is stored in the adsorbed water layers and manifests physically as ***soil cohesion***.

The two forms of effective stress can not be physically added together because they are acting in different media and different in nature. At the same time, it is desirable to handle the internal effective stress as a continuous function. Thus, there is need to be able to add the two forms of effective stress in a consistent manner. The integration of the two forms of internal stress is possible by combining the central role of the SBS as a storage chamber and the build up of hydrotransient (swelling) pressure. The integration was presented as a model and is outlined in the next sub-section.

6.3.2 The internal stress model

During the swelling phenomenon, hydrotransient (swelling) pressure mobilises as a continuous function of water content up its maximum value (point C). Thereafter, it decreases continuously with water content. At the same time, the conversion of the effective stress from being internal to being mobilised (external) takes place during the build up of hydrotransient pressure. Meanwhile, the soil structure is stiffest at maximum hydrotransient pressure. As such, the hydrotransient pressure is considered a measure of the internal effective stress, while the soil structure is the SBS storage chamber.

Thereafter, the mobilised effective stress is reloaded ‘internally’, with accompanying progressive destruction of the storage chamber (soil structure) as it becomes redundant. The model is summarised in table 6.3.

Table 6.3 The internal stress integration model

Swelling phenomenon	***The internal stress model***
Hydration	Unloading of the internal effective stress, σ’
Growth of adsorbed water layers & mobilisation of soil cohesion	Development of the SBS chamber
Mobilisation of swelling pressure	Loading of the mobilised effective stress
Maximum soil cohesion	Fully developed SBS chamber
Re-mobilisation of soil attraction (induction)	Strengthening of the SBS chamber to match increasing mobilised effective stress
Increase in pore water and decreasing hydrotransient (swelling) pressure	A decrease in the mobilised effective stress in accordance with the principle of effective stress
Simultaneous decrease in hydrotransient pressure and volume increase	Reloading the mobilised effective stress back to the soil skeleton (effective stress-strain response)
Increase in volume following the dissipation of swelling pressure	Removal of the defunct SBS chamber following the removal of the mobilised effective stress

The integration model was used in the effective stress - strain analysis, which is reported in chapter seven.

6.4 Conclusions

- The effective stress hypothesis was experimentally validated. Its rationality was further demonstrated by enabling the swelling phenomenon to be viewed in analogy with the well-known concept of consolidation settlement.
- The internal stress of the investigated expansive soil exists in two forms. The first in an internal form and is called internal effective stress (soil attraction). The second form is external and is called mobilised effective stress.

- Swelling pressure is isotropic, while the swelling process can be anisotropic, depending on the degree of soil confinement. In view of its nature, swelling pressure was appropriately terms hydrotransient pressure.
- The developed internal stress model integrates the two forms of internal effective stress as a continuous function of water content. This scenario is desirable in modelling the swelling soil behaviour in effective stress terms.

Chapter 7

The internal effective stress-strain behaviour

The internal stress-strain behaviour of a swelling soil requires two separate analyses with respect to the mobilisation and reloading of the effective stress. It was shown in chapter six that the measured horizontal swelling-pressure when the soil freely swells vertically is the isotropic swelling pressure. Consequently, it is equal to the mobilised effective stress of the soil. For one-dimensional case, the swelling strain in the vertical direction is a result of the change of the internal effective stress of the soil. Since the mobilised effective stress is isotropic, the term ‘effective stress’ is used herein in reference to both the ‘mean’ and internal effective stress, without further qualification.

Representative test results from a series of laterally confined swelling test were analysed and the analysis is presented this chapter. The analyses sought to characterise the mechanical response of the investigated expansive soil, to water flow. Test results obtained along the flooding swelling path were analysed in detail. The resistance concept outlined in section 3.4 of chapter three was adopted for the analysis. The analyses also highlight the significance of the arbitrary water content point B and D, in terms of internal effective stress. The analysis was carried out in the context of the internal stress model. The analysis and presentation is original.

7.1 Effective stress-strain results

Presented in this section are the effective stress-strain results obtained in accordance with the internal stress model, where swelling pressure was considered a measure of the internal effective stress.

7.1.1 Results from a range of initial water content

The internal effective stress-strain relationships of the investigated soil for a range of initial water content are plotted in fig. 7.1. An elbow characterises the general shape of the curves. It is evident that initial water content does not affect the shape of the curve, but just the starting point Accordingly, one result with sufficiently low initial was content was analysed to give the overall soil response.

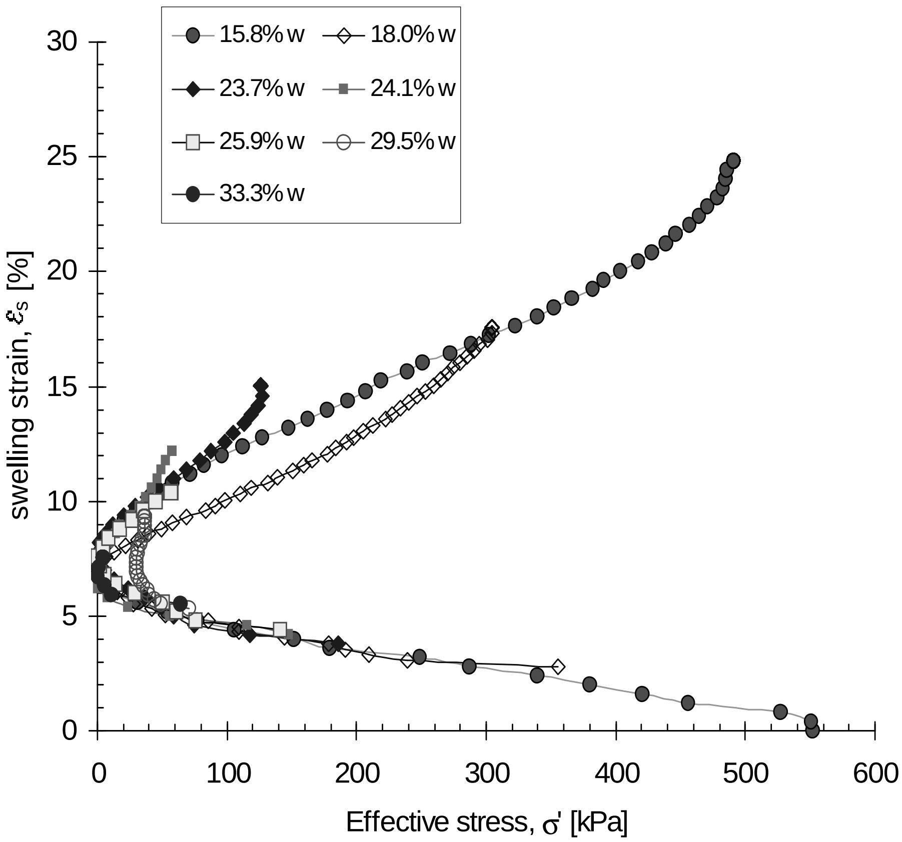

Figure 7.1 Effective stress-swelling strain plot

7.1.2 Typical stress-strain result

The internal effective stress-strain relationship of the investigated soil, with initial water content of 18.7 per cent is given in fig. 7.1. The positions of the arbitrary water content points defined in the induction concept are indicated. The points define the swelling phases as before. Points C and C' seem to be coincident with respect to swelling strain, but they are not coincident in terms of soil stiffness. This will become evident in the subsequent analysis.

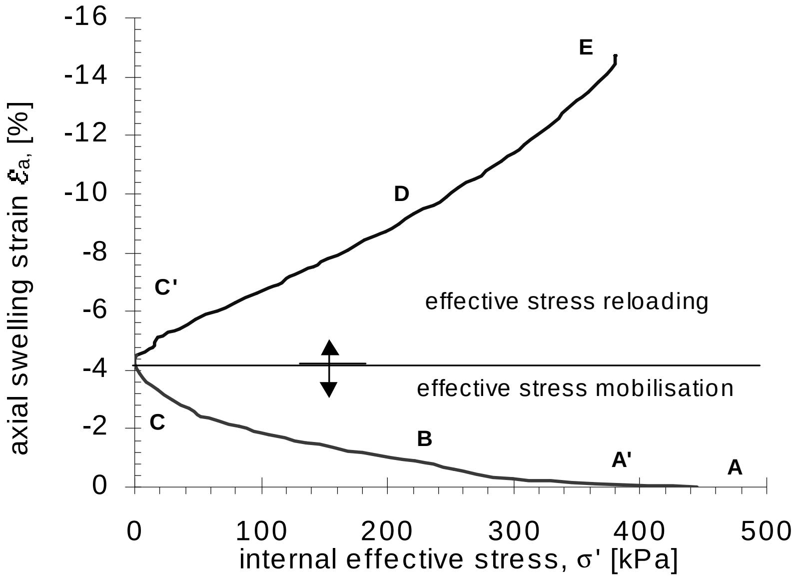

Figure 7.2 Stress–strain relationship along flooding wetting-path

The shape of the curve in fig. 7.2 has well-defined features, which have physical significance. Point C corresponds to the point of maximum hydrotransient pressure and zero internal effective stress. Since the internal effective stress now exists in the mobilised state, the elbow conveniently separates the soil behaviour in terms of effective stress. Below the elbow is stress unloading and mobilisation, while above is stress reloading. Accordingly, the two were analysed separately as present in sections 7.3 and 7.4 respectively.

7.2 Method of analysis and test results

7.2.1 Method of analysis

The tangent modulus, M as defined in subsection 2.4.3.2 of chapter 2 was used in the analyses. Specific reference was made to the shape of the tangent modulus plot and its significance in terms of increase in soil stiffness with effective stress. Fig. 7.3 is a typical illustration of the plot.

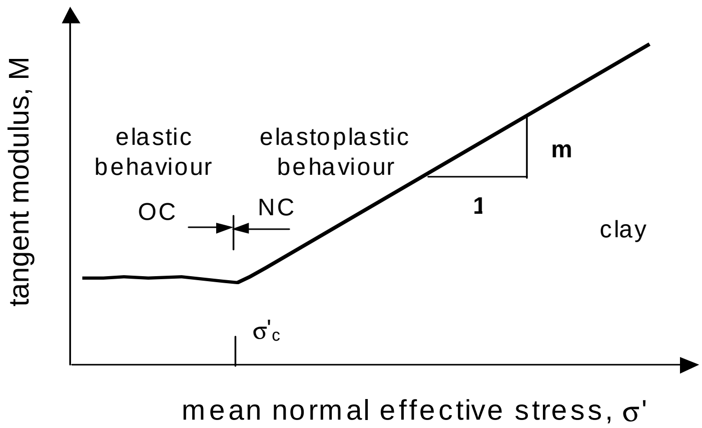

Figure 7.3 Idealised oedometer tangent modulus: over-consolidated soil (*Janbu, 1963, 1998*)

<u>Overconsolidated zone (OC)</u>

The constant tangent modulus is indicative of a reversible soil response. This is because the compressive effective stress applied is less than that which the soil ever experienced in the past. Thus, the soil response is reversible and is considered elastic. The point of change of the soil stiffness is called the ***pre-consolidation pressure***. It denotes the maximum effective stress that the soil has experienced in the past.

<u>Normally consolidated zone (NC)</u>

The effective stress applied beyond the pre-consolidation pressure takes the soil along a new or virgin compression path. The soil develops both elastic and plastic strains, hence the term elasto-plastic behaviour. This behaviour is indicative of increasing soil stiffness, as denoted by an increase in the tangent modulus.

7.2.2 Soil parameters

The soil parameters relating to internal effective stress are defined as shown in table 7.1. The terms tangent modulus, *M* and modulus number, *m* are reserved for the conditions of external loads as defined by (Janbu, 1963)

Table 7.1 Terminology for the soil parameters in the tangent modulus analysis

soil response	***parameter***	***New terminology***	***symbol***
stress (unloading) mobilisation	tangent modulus	mobilisation modulus	M_b
	modulus number	mobilisation number	m_b
stress reloading	tangent modulus	reloading modulus	M_r
	modulus number	reloading number	m_r

The coupling within the swelling process involves water content, internal effective stress and swelling strain. The influence of water content and effective stress on swelling strain is presented in this section. The arbitrary water content points are used to indicate the change in the soil structure (SBS). Pertinent features of the relationships are highlighted.

7.3 Stress mobilisation analysis

Stress unloading is synonymous with mobilisation and storage of effective stress. The internal effective stress is downloaded, converted and stored in the adsorbed water layers, that is outside the clay minerals but within the soil matrix. The mobilised form of effective stress takes up more space and accounts for the corresponding volume increase. In other words, the stored effective stress, being in a mobilised state, acts on the clay minerals, causing them to rearrange and or displace. Since the strain arises from the swelling of the hydrating clay minerals, it is appropriately called ***swelling strain***.

7.3.1 The mobilisation modulus, M_b

Reference is made to the mobilisation modulus plot in fig. 7.4. It relates to the stress-strain curve below the 'elbow' of the curve in fig. 7.2. The stages of the evolution and motion of the SBS chamber are shown in the graph. The soil was swelled along the flooding wetting-path.

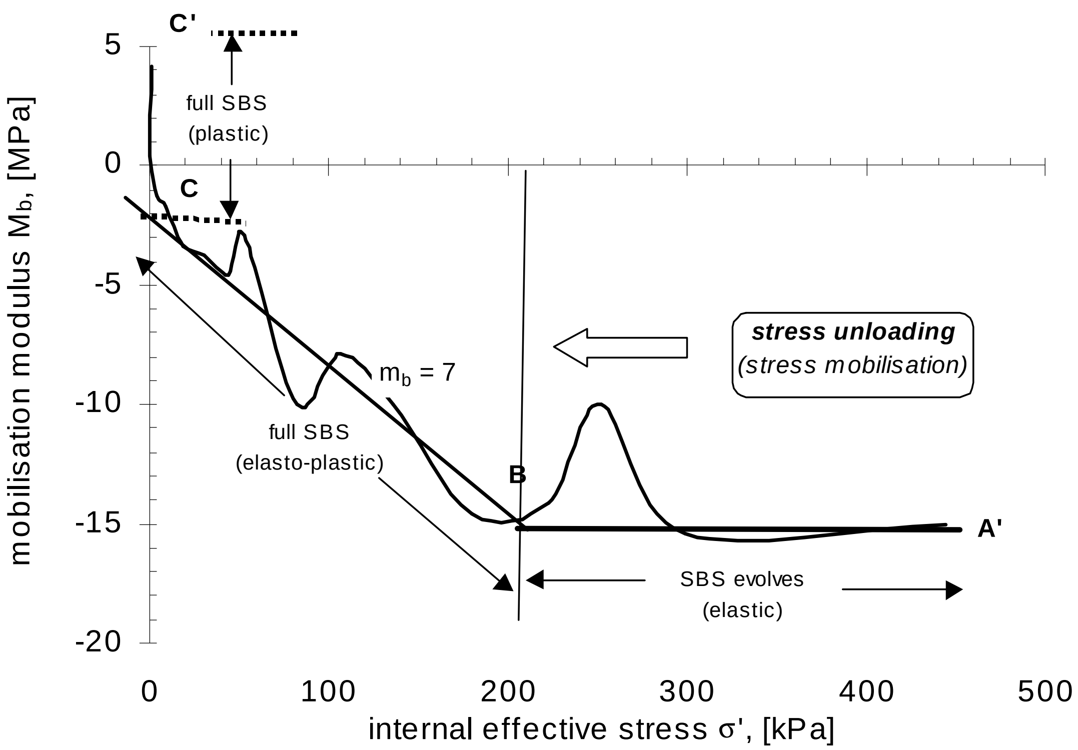

Figure 7.4 Mobilisation modulus plot

The shape of the mobilisation modulus plot bears striking resemblance with that of the tangent modulus of over-consolidated soils, which is illustrated in fig. 7.3. They are an inverted mirror image of each other about the tangent modulus axis. The inverse relationship seems to confirm that one (mobilisation modulus) relates to stress unloading, while the other (tangent modulus) is stress loading. The analogy was used to discuss the soil behaviour as follows.

7.3.1.1 Elastic swelling

The constant modulus in phase (A'-B) of fig. 7.4 indicates elastic behaviour. From the analogy above, it means that the effective stress remaining in the solid clay minerals is less than the maximum stress the clay minerals ever carried in the past without being stressed. This is expected since the mobilisation process is removing or downloading the internal effective stress from the solid particles. Thus, the soil response is elastic because internal effective stress is a function of particle distance. In accordance with the induction concept, the transfer of the internal effective stress to the adsorbed water layers is completed at point B. This assertion appears to be vindicated. This confirms that the downloading of the internal effective stress is elastic and is completed at point B. Therefore, the mobilisation of effective stress in this phase is elastic.

Conversely, soil destructuration is a consequence of the action of the mobilised effective stress (soil cohesion) on the clay platelets. As such, the constant modulus indicates that the mobilised effective stress is less than the maximum that was previously mobilised and carried by the clay minerals. This is also the case because the mobilised effective stress becomes a maximum at point B, when the clay minerals are emptied of the effective stress.

The observed elastic response can also be explained in physical terms. Initially the soil was flooded and had the pores filled with water. Phase (A'-B) is associated with hydration and 2nd-level soil destructuration. The pore water sustains hydration on the one hand, and cushions direct particle attraction on the other. That is, the soil structure disintegrates such that the linear growth of the water layers and hence swelling strain continues uninterrupted. Thus, the presence of pore water causes the soil to respond elastically to the action of mobilising effective stress.

The elasticity can also be viewed from the point of view of the internal effective stress (soil attraction). Hydration capsulate the unloading of the internal effective stress. The bonding between the clay plateletsis due to soil attraction, which is a function of inter-clay mineral distance. Therefore, the swelling is elastic because the random bonding can be re-established by reducing the clay mineral distance to within the molecular range. This is possible by inducing internal effective stress (soil drying).

The soil becomes plastic and the soil structure becomes homogenous and normal at point B. This is because it is made up of unstacked clay minerals in a plastic medium. Point B defines the ***limit of soil elasticity***.

7.3.1.2 Elastoplastic swelling

The mobilisation modulus linearly increases with decreasing internal effective stress in phase (B-C). An increase in the modulus signifies an increase in the effective stress acting on the soil, despite the observed decrease in the measured internal effective stress. This apparent anomaly is a consequence of the possible existence of internal effective stress in two forms (internal and mobilised). What is recorded in fig. 7.4 is the internal effective stress, which is stored in the solid clay minerals. The anomaly can be reconciled as follows:

The internal effective stress lost by the solid particles up to point B is related to the hydration in the phase (A-B). The effective stress carried by the adsorbed water layers between zero water content and point A is not actively mobilised and is therefore not lost. The (passively mobilised) effective-stress is activated and converted to the mobilised form in phase (B-C). Since it is associated with the initial water content and is mobilised last, it is herein called ***residual effective stress***. Likewise, the initially mobilised effective stress, which is directly associated with maximum soil cohesion at point B is herein called ***primary effective stress***.

The (primary) mobilised effective stress is stored as adsorbed water (soil cohesion) around the hydrated clay particles, while the residual effective stress is located in the water layers that are closest to the clay minerals. It is therefore 'buried' under the mobilised effective stress. Consequently, its mobilisation is considered to involve two stages. The first stage is to extract the residual effective stress and bring it to the surface, where it can be effectively mobilised. The process is effected by the induction phenomenon, during which soil attraction presses and squeezes water out of the water layers. As such, the process becomes a shearing process, in which the hydrated soil structure is altered in order to squeeze out the residual stress. The amount of water squeezed out is equivalent to the residual effective stress. Removal of part of the adsorbed water constitutes a decrease in the internal effective stress, hence the observed decrease in internal effective stress.

The second stage is mobilisation of the residual effective stress. The process takes place in the context of the induction phenomenon, where the particles are physically forced to come together, as opposed to the natural attraction due to soil cohesion. As such, the residual effective stress expresses itself in a different form namely, induced osmotic potential. According to the induction concept, osmotic potential is the equivalent of soil adhesion under saturated conditions, and is the potential linked to the development of soil cohesion (mobilised effective stress). Thus, soil stiffness increases because of the increasing mobilised effective stress.

In analogy to the tangent modulus, the linear increase of mobilisation modulus is elastoplastic. This response is a coupling between two interactions. The induction phenomenon is a function of clay mineral distance: It is considered reversible and hence elastic. However, the soil is physically plastic. Therefore, the coupled unloading and mobilisation of the residual effective stress becomes an elastoplastic shearing process.

The soil swells and hence dilates during the shearing process. Dilatancy is manifested in over-consolidated soils. Therefore, ***the investigated soil is over-consolidated***.

Meanwhile, the SBS chamber at point B is full to the (elastic) limit. Any further increase in the mobilised effective stress shears the chamber. Stressing the chamber would cause it to stretch plastically because the soil is plastic. However, the induced soil attraction reinforces the SBS chamber. Since the SBS chamber is at limiting elasticity at point B, it yields and stretches out elasto-plastically to contain the additional effective stress.

During the shearing process, soil attraction presses and stiffens the clay minerals, thereby containing the osmotic potential. Thus, the reinforced SBS chamber contains the mobilised effective stress and in a sense makes the mobilised effective stress ‘internal’. The soil stiffness at point C is equivalent to the highest stiffness of the soil under unsaturated conditions.

7.3.1.3 Plastic swelling

The soil at point C is pressure saturated and the mobilised effective stress is in the osmotic potential form, which is a water phase. This scenario is analogous to the immediate response of a water saturated soil element subjected to an external load during consolidation settlement. In the case of the water saturated soil element, the additional effective stress is immediately carried by water phase leading to a build up of excess positive pore-water pressure. The effective stress is eventually transferred to the solid particles, while the excess pore water pressure dissipates. In this case, the mobilised effective stress is similarly carried by the water phase as osmotic potential, and similarly builds up negative pore water (swelling) pressure.

Accordingly, the soil behaviour in phase (C-C’) does not involve an increase in effective stress in the soil water system, but a transfer of the residual effective stress to the clay mineral solid. This is effected by diffusion of the same amount of water that was squeezed out during the induction phenomenon. The water neutralises the osmotic potential and in that sense dissipates the negative pore water pressure. The process does not register as a measurable change in effective stress because the change involves a potential form of effective stress. However, the change is reflected in the soil structure because the form of effective stress occupies more space. This is discussed as follows.

The soil structure (SBS storage chamber) is stiff and stretched to the limit at point C. The chamber can not accommodate the new form of effective stress (equivalent amount of water) within the soil volume. Apart from this, the reinforcing effect of soil attraction is exhausted. However, the new form of effective stress (equivalent amount of water) has a softening effect on the storage chamber. Accordingly, the chamber softens and stretches to provide the extra storage required. Following the exhaustion of soil attraction, the soil reverts to maximum plasticity. Therefore, the SBS stretches plastically to increase the storage capacity. Accordingly, the mobilisation modulus increases at constant effective stress (C-C'), confirming plastic shearing. Therefore, ***the swelling is plastic.***

The volume increase is plastic because there is no increase in effective stress. It is a rearrangement of the clay minerals. The condition of plastic flow (C-C') is consistent with the attainment of a critical state condition, as discussed in section 6.2.2 of chapter six. Soil plasticity is a condition of the clayey soil, during which the soil can deform into any shape on the application of force that exceeds the yield value, without disturbance to particle coherence or development of surface cracks.

7.3.2 Significance of point B

Point B is significant to the stress mobilisation process in terms of the change in soil stiffness, internal effective stress and soil structure. The significance points to the uniqueness of point B in the swelling process.

7.3.2.1 Internal effective stress

Maximum soil cohesion at point B is a measure of the maximum effective stress that can be downloaded and externally stored around the clay minerals. Thus, at point B the effective stress equilibrium in the clay mineral is re-established, with the stress being externally stored. Accordingly, the soil attains a condition of normal stressing and is therefore, ***normally stressed***. Therefore, point B marks the restoration of the soil's internal effective stress equilibrium. However, the normal stressing attained is with respect to the initial soil condition and not necessarily the stress history of the soil.

The amount of internal effective stress, which the soil can download elastically, is limited by two factors. It can be limited to the maximum excess stress that the soil can carry at the given initial water content or alternatively, it can be limited by the available pore space and hence the initial water-content (porosity). Point B is analogous to the preconsolidation pressure, as implied in the identity with the tangent modulus plot. In general, the effective stress at point B depends on the effective stress associated with the initial water content. However, there exists a condition of limiting effective stress at point B, which reflects the stress history of the soil, similar to preconsolidation pressure. Analogous to preconsolidation pressure, the corresponding downloaded effective stress is called the ***pre-swelling stress***. The dependency of point B on the swelling path leads to the concept of ***over-swelling ratio (OSR***), analogous to over-consolidation ratio. These deductions are consistent with the coupling that exists between the change in the internal effective stress, soil stiffness and change in soil structure.

7.4 Stress reloading analysis

The soil response above the elbow post- swelling process is characterised by an increase in the internal effective stress of the soil. Physically, the process is a dissipation of the stored hydrotransient (pore water) pressure. According to the internal stress model, the mobilised effective stress is being reloaded back to the soil skeleton (solid clay minerals). However, the effective stress is of an external form that can not physically enter the clay minerals. Therefore, it is externally reloaded and dissipates in the process. This translates into stressing (stretching) the hydrated clay and dissipates in the process. Physically, the volume increase is due to the increasing water in the soil. However, the internal stress model attributes this to the increasing effective stress. Thus, the increase in internal effective stress manifests as external, stresses the clay minerals, causing volume increase. It is reasonable to say that the volumetric strain is equal but opposite to that of physically increasing the internal effective stress (shrinking). The stressing effect constitutes the reloading process. It is considered a mechanical action of the effective stress on the clay particles. The strain arises from the stressing effect is herein called ***stressing strain.***

7.4.1 The re-loading modulus, M_b

Fig. 7.5 shows the effective stress-strain plot for the post-swelling process. The dissipation of effective stress beyond point C' has a stressing effect on the clay particles. However, the apparent increase in the internal effective stress of the soil indicates that it is stress loading of the clay particles. In order to be consistent with the changes in the effective stress in the soil, the stressing process is herein referred to as a stress reloading process.

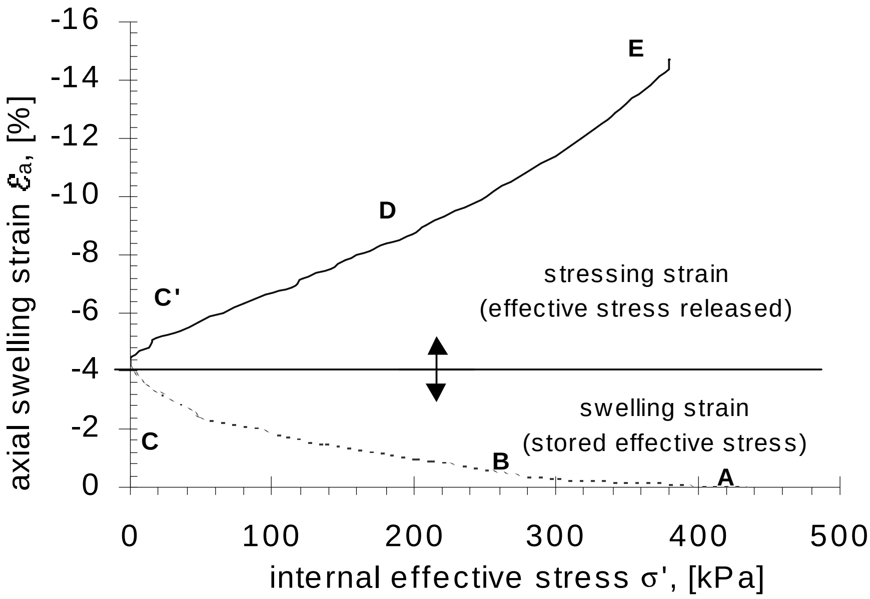

Figure 7.5 Internal effective stress-strain relationship

The observed increase in internal effective stress is confirmation that the mobilised effective stress is being reloaded to the clay minerals. However, the effective stress is stored outside the clay minerals and the reloading process is not an induction phenomenon. Accordingly, the stress is not physically stored in the clay minerals, but dissipates. However, it stresses the clay minerals externally, to the same extend that internal effective stress would compress them. Accordingly, the mobilised (stored) effective stress is the stress retained by the soil, while the reloaded effective stress dissipates. To this end, the reloading process is considered ‘internal’, but referenced to the remaining (stored) effective stress.

7.4.2 Reloading modulus, M_r

Reference is made to the reloading modulus plot in fig. 7.6. It corresponds to the reloading stress-strain curve in fig. 7.5. The reloading modulus is a mirror image of the tangent modulus of a compressing over-consolidated soil (fig. 7.4), about the effective stress axis. The relationship arises from the fact that both are stress loading and yet one involves volume increase, while the other causes volume decrease. The analogy reinforces the discussions made in respect of stress unloading in section 7.2

7.4.2.1 Elastic reloading

The reloading modulus is constant along (C’-D), indicating that the strain increases linearly with effective stress. It also indicates elastic soil behaviour, during which the soil stiffness does not change. However, this appears contradictory to the expected reduction in soil stiffness accompanying the volume increase. Generally, volume increase tends to weaken the soil and reduce its stiffness. The mechanism leading to the observed elasticity can be explained as follows. The clay particles are overstressed at point C’, with the mobilised effective stress (soil attraction) compressing the adsorbed water layers to minimum possible volume. The soil attraction at point C’ is limiting, having been stretched during plastic shearing. Therefore, the reloading of the stored effective is a stress-relief process. Naturally, the clay minerals shed off the excess stress as quickly as possible and become normally stressed. It is asserted that the quickest and easiest way to release stress is elastically. This is synonymous with removal of soil attraction. Consequently, the hydrated clay minerals relax and thus take up more volume. However, the apparent volume increase is not creation of extra void space within the soil. Rather, it is a rearrangement of adsorbed water layers, which are recovering from the squashing effect of soil attraction. The adsorbed water layers can be viewed as rebounding, a property typical of elastic materials when relieved of pressure. Accordingly, the elasticity in phase (C'-D) is a release of the effective stress excess of the previous maximum stress carried by the soil. Thus, the soil behaviour in phase (C'-D) is ***elastic***.

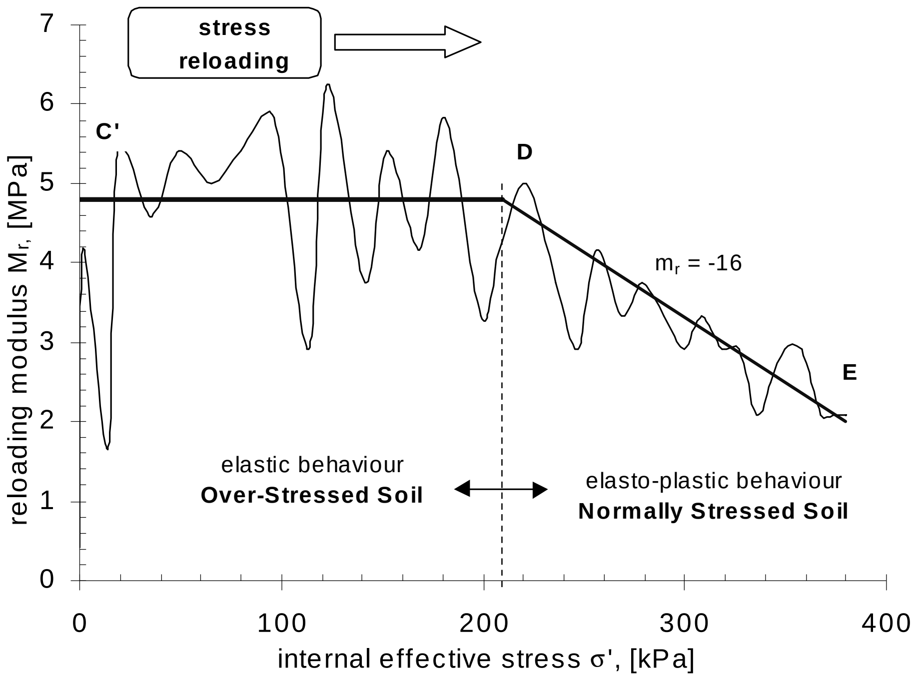

Figure 7.6 Reloading tangent modulus plot

However, the reloading modulus is highly undulating, especially after the 100kPa stress-level. This behaviour is considered to be due to the change in the soil structure. At point C', the hydrated clay minerals are clustered together under high pressure. Thereafter, the released effective stress acts on and destroys the soil structure. At some point, the link between the adsorbed water layers loosens up such that the soil stiffness can not be uniformly transmitted across the soil mass. Accordingly, it becomes localised and hence the sporadic behaviour. This behaviour seems to indicate a reversal of the soil structure.

The reloading (release) of excess effective stress completes at point D, meaning that the soil is normally stressed. The clay minerals are normally stressed, with the adsorbed water layers of adjacent particles just touching, but kept together by the maximum soil cohesion. The soil stiffness does not change because it is borne out of maximum soil cohesion, which is constant throughout the phase. It is the maximum stiffness because maximum soil cohesion is directly linked to the initial soil stiffness via the initial soil water potential. The soil attains internal equilibrium condition that is compatible with atmospheric pressure. In other words, the mobilised (stored) effective stress in the soil is equal to atmospheric pressure. Consequently, the soil at point D is normally stressed.

7.4.2.2 Elastoplastic reloading

The soil at point D is normally stressed and the SBS is water saturated and at atmospheric pressure. Subsequent dissipation of the stored effective stress beyond point D is not possible without an external action to offset the equilibrium. This is because it involves lowering the effective stress below atmospheric value. According to the SBS physical soil model, point D does not necessarily correspond to a level at the top of the soil sample. It is fixed by the steady position of the advancing wetting front during initial wetting. The soil within the SBS is swollen and water-saturated. However, the soil above the SBS is relatively ‘dry’ and has swelling potential. Consequently, point D is such that a water-content gradient exists across the top end of the SBS chamber. The water content gradient makes it possible for the water to diffuse in the SBS. Because water flow across the SBS is by diffusion, the soil above the SBS swells along suction wetting-path. It means that stress unloading and mobilisation of effective stress are coupled. In addition, the swelling process is concentrated on the upper surface of the SBS.

Meanwhile, stress mobilisation is continuously referenced to atmospheric pressure within the water-saturated SBS. Therefore, the mobilised effective stress is immediately released. Thus, stress unloading, stress mobilisation and stress reloading become a coupled process, with stress reloading being dominant. At the same time, the SBS slowly drifts into the ‘dry’ soil. Water flow in the SBS reduces the mobilised effective stress in the wet soil, in accordance with the effective stress principle. The effect is however, very slow because the water diffuses through to the top end of the SBS in response to the water content gradient. Since water does not mobilise shear, the overall impact is volume increase accompanied by a decrease in soil stiffness.

Thus, the decrease in the tangent modulus between points D and E is a result of the release of the stored effective stress, in response to the increase in free water within the wet soil. Accordingly, at point E the whole soil sample has the same water content. The water content and residual effective stress stored in the wet soil at point E depend on the external force that causes water flow. The greater the force, the more the water enters the soil. Accordingly, the smaller is the residual effective stress in the soil, as more free water stakes between particles and reduces the effective stress. That way, the closer the soil condition comes to point F, the point of zero effective stress and onset of liquid phase.

As already discussed, this phenomenon is elastoplastic shearing. Suction-gradient swelling is normal to the soil and is analogous to oedometer compression of normally consolidated clays. The analogy can be extended further. Normal reloading is similarly a result of ‘additional’ effective stress. It is ‘additional’ in the sense that it is produced outside the system originally bounded by the SBS. The additional effective stress is mobilised during the swelling process in the ‘dry’ soil above the SBS.

7.4.2.3 Demobilisation of the SBS chamber

The saturated soil at point D has maximum soil cohesion and the SBS chamber is well defined. Thereafter, the SBS chamber slowly enlarges as it takes up more water, while the ‘dry’ soil simultaneously wets up and swells. That is, it is stretches and is thus weakened by the water, which displaces the stored effective stress. The SBS drifts to the top of the soil sample (point E) and gets weaker in the process. As the stored effective stress continues to decrease under continued water flow, the SBS would stretch and move point E until it vanishes at point F. The release of the effective stress would thus, slowly demobilise and remove the chamber, as predicted by the proposed physical soil model. Therefore, the observed experimental results completely vindicate the SBS model.

7.4.3 Significance of point D

7.4.3.1 Internal effective stress

Point D defines the onset of normal stress conditions, following dissipation of excess effective stress. Suction swelling emphasises the fact that normal stress condition at point D is in reference to atmospheric pressure. In general, the internal (reloaded) effective stress at point D depends on the initial water content and swelling path. The different initial water-contents reflect the extent to which the soil is stressed. However, there is a unique initial condition such that the internal effective stress at point D reflects the stress history of the soil. It is a limiting condition and it defines the maximum internal effective stress that the clay particles have been subjected to in the past. Since the soil is being stressed or stretched, the effective stress is herein called ***pretension stress,*** analogous to preconsolidation pressure. The dependency of point D on the swelling path leads to the concept of ***over-stressing ratio (OSR***), which is analogous to over-consolidation ratio.

7.5 Conclusions

The pertinent conclusions that can be obtained from this chapter are as follows.

- The tangent modulus analyses validated the applicability of the proposed internal stress conceptual model, in analysing the stress-strain behaviour of the expansive soil investigated.

- The soil condition corresponding to points B and D are significant in terms of both soil structure and effective stress of the soil. The soil has normal structure and is normally stressed at both points. The 'normality' observed at these points is emphasised by the results along suction swelling, where the two points have approximately the same internal effective stress.

- Point B is associated with limiting elasticity. Point D is the atmospheric saturation point and is associated with limiting plasticity.

- The initial water content of the soil does not change the mechanisms of the swelling process, as proposed in the model. However, the soil response from given initial water content takes up the response consistent with the swelling phase, within which the initial water content falls.

Chapter 8

Internal effective stress & the intrinsic soil property

The analyses in chapter seven reveal that the change in the internal effective stress during soil swelling is such that the soil condition at points B and D attains limiting conditions. The corresponding effective stresses are pre-swelling and pretension stress respectively. Both quantities relate to the stress history of the soil. Accordingly, this chapter sets to experimentally establish the unique connection between points B and D. The uniqueness of point D was established by determining the effective stress at point D that is analogous to the preconsolidation pressure.

For the investigated unsaturated expansive soil, the internal effective stress is coupled with the soil water content. Therefore, the uniqueness of the water content points was investigated from the water-content point of view. That is, to determine the limiting water content, when the effective stresses at points B and D are equal. Thereafter, the intrinsic soil property was defined in terms of the relationship between internal effective stress and water content. This is also presented here for the first time. The chapter concludes by outlining the determination of the intrinsic soil property from routine laboratory test results. The work presented in this chapter is an original contribution by the writer.

8.1 The internal effective stress and limiting water content

The pre-swelling and pretension stresses characterise the mobilisation (unloading) and reloading processes of the internal effective stress respectively. Accordingly, it is sufficient to study the two stresses as a means to characterise the connection between the mobilisation and reloading of the effective stress during soil swelling.

8.1.1 Internal effective stress

Analogous to oedometer compression of saturated soils, it is possible to determine the equivalent of preconsolidation pressure during soil swelling. Of the two points, B and D, point D directly relates to the preconsolidation pressure because it is similarly along a stress loading path. In addition, it focuses on the effective stress that is directly acting on the solid clay minerals.

8.1.1.1 Pretension stress, σ'_T

The maximum effective stress that the soil can carry without being over-stressed is herein called ***pretension stress, σ'_T***. It is equal to the limiting effective stress at point D. In general, the effective stress at point D depends on the initial water content. Therefore, a plot of the variation of tangent modulus at point D with initial water content reveals the water content that relates to the pretension stress.

Reference is made to the reloading modulus plot in fig. 8.1, in which test results from a wide range of initial water content are plotted. The internal effective stress is expressed in its measurable form of hydrotransient pressure. The influence of initial water content on the mobilised effective stress and hence maximum possible internal effective stress is evident. The condition of dramatic change in the reloading modulus exists for water contents between 18 and 23 per cent. The mobilised effective stress corresponding to the change point is 210kPa. It separates the elastic from the elasto-plastic soil response. Accordingly, ***the pretension stress is 210kPa***.

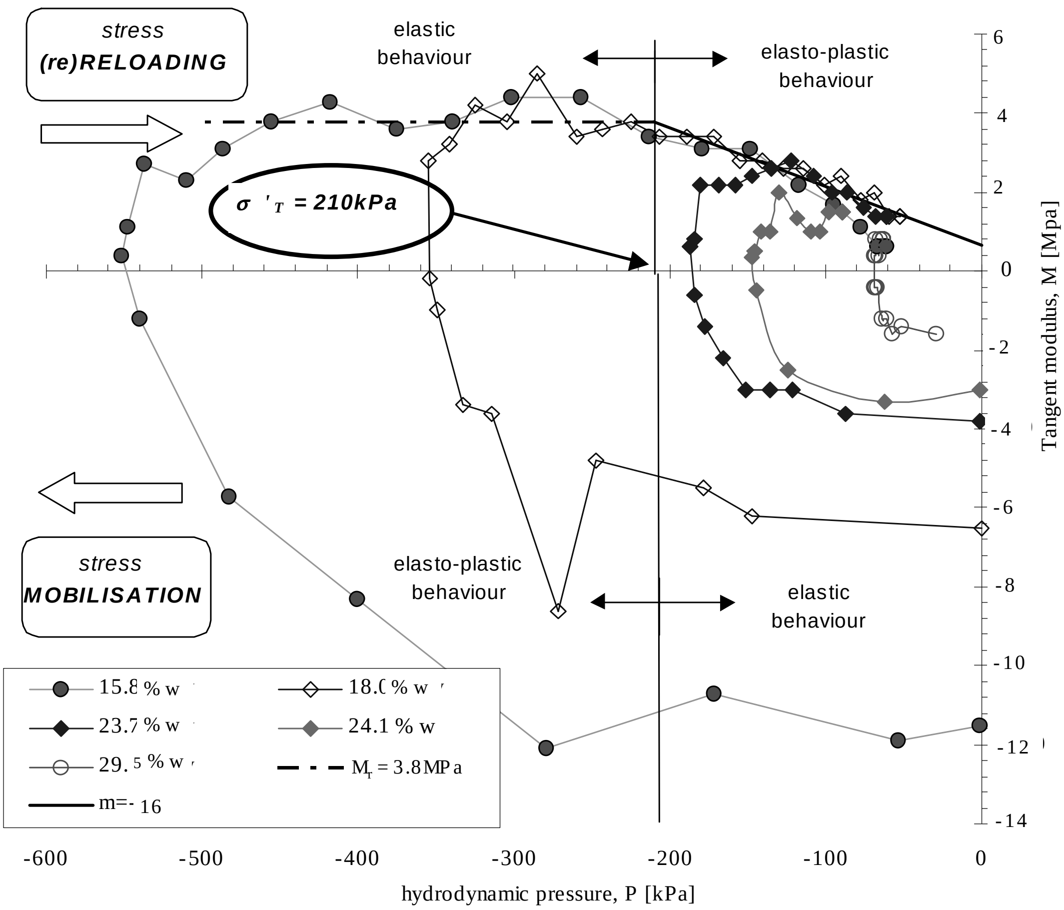

Figure 8.1 Determination of the pretension stress

8.1.2 The limiting water content

The uniqueness of points B and D and their link to the pre-tension stress were determined by recourse to the initial water content and is discussed in the next sub-section. Reference is made to the stiffness modulus plot in fig. 8.2. The sample had initial water content of 18.7 per cent and was swelled along flooding path. The decrease in the reloading modulus from point D can be extrapolated beyond point E to point F. Point F has zero internal effective stress. The rationality of the extrapolation becomes evident in the ensuing discussion.

Two very important deductions were made from the figure. Firstly, the effective stresses at point D and B are equal to the pretension stress. That is, the figure shows a limiting case, when the effective stress carried by the water phase at point B is equal to the internal effective stress of the soil at point D. The pretension stress refers to the effective-stress that is reloaded (restored) to the soil, while pre-swelling stress relates to the effective stress that is lost (mobilised) by the soil. However, it is fundamental to define the stress history of a soil in terms of the effective stress remaining in and acting on the soil rather than that removed from the soil. Accordingly, the pre-swelling stress is herein called ***post-tension stress***, in complement to ***pretension stress***.

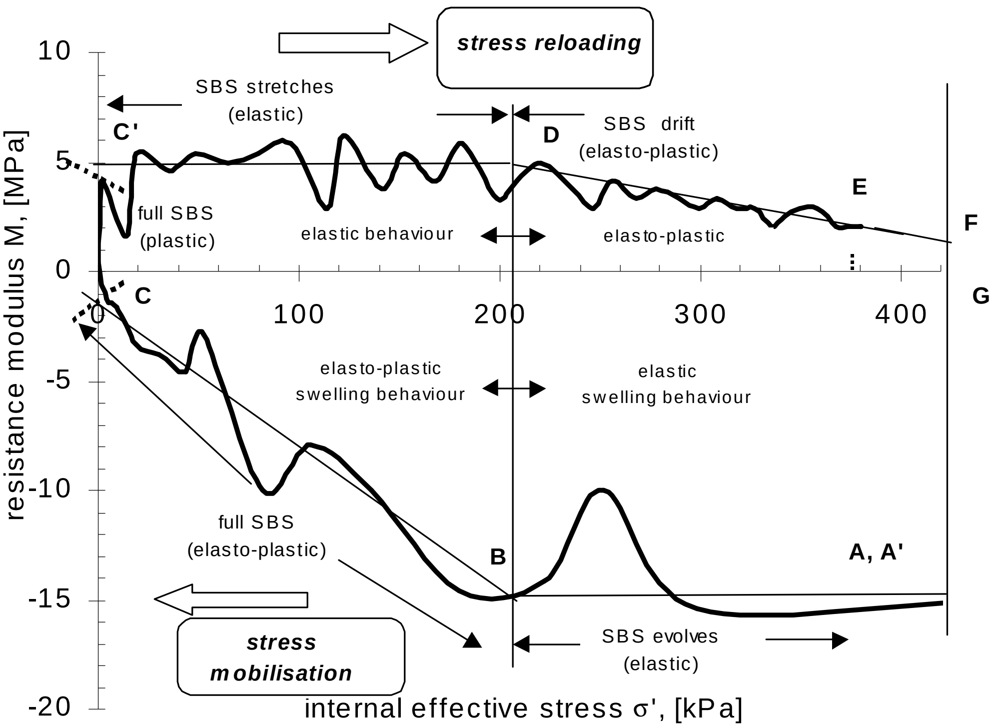

Figure 8.2 Internal effective stress-modulus plot at shrinkage limit

Secondly, the initial water content of 18.7 per cent is very close to 19 per cent, which is ***the linear shrinkage limit of the investigated soil.*** This observation confirms the rationality of the shrinkage limit. Earlier, Williams and Sibley (1992) showed the uniqueness of the linear shrinkage limit in a different way. They experimentally showed that the volumetric air-content, heat of wetting, tensile strength, total suction and thermal resistivity dramatically change at the linear shrinkage limit. Accordingly, the rationality of the shrinkage limit is deemed established. Therefore, the internal effective stress at the shrinkage limit is of fundamental importance and gives the shrinkage limit significance. This points to the rationality of the shrinkage limit and hence the other consistency limits.

In view of this, it is instructive to study the stress-stain behaviour of the soil, when the initial water content equals the shrinkage limit.

8.1.3 Pretension ratio

The maximum internal effective stress of the soil, measured at point C, can be expressed as a ratio of the pretension stress. The ratio is herein called ***pretension ratio***. In general, the pretension ratio depends on the initial soil water content. The case when the shrinkage limit is the initial soil water content is special. Then the pretension ratio is two. It is special because according to the induction phenomenon, the internal effective stress can be measured during the unloading and reloading processes. Therefore, such a scenario mirrors the reversibility of the change in internal effective stress about the pretension stress point. (A similar behaviour was observed in chapter five, but was expressed in terms of the soil structure (SBS)). Such is the case for the soil condition presented in fig. 8.1. It sets a water content limit, below which the soil can mobilise effective stress more than twice the pretension stress.

8.1.4 The effective stress at point D

The pretension stress at point D is not physically present in the soil because it is released or dissipated. This then seems to suggest that the mobilised effective stress at point D is equal to atmospheric value. At the same time, the soil water content at the end of the test (Point E in fig. 8.2) was 40 per cent. A similar test run for 10 days, under a pressure gradient of 250kPa per metre, showed that the soil did not take up any water beyond 40 per cent. It seemed that the removal of the mobilised effective stress beyond point D requires excessive force, which may not be provided by mere water flow. Accordingly, the mobilised effective stress at point D should be higher than the atmospheric value. A quick reconciliation of the internal effective stress at the initial condition provides an explanation.

At any initial water content, the soil has layers of water adsorbed around the clay minerals. That is, the initial soil condition is associated with mobilised effective stress. This is the effective stress, which is mobilised by taking the soil from zero water content (dry state) to the initial water content, which in this case is 19 per cent. The effective stress at initial water content was referred in chapter seven as ***residual effective stress,*** σ'_r.

According to the characterisation of the internal effective stress, the induction of soil attraction in phase (B-C) activates mobilisation of the residual effective stress. Therefore, the residual effective stress, which is subsequently mobilised accounts for the high soil stiffness at point D. That is, the effective stress carried by the water phase at point A' is still stored in the water at point D however, in a mobilised condition. Therefore, the effective stress condition at point D is the same as that at point A' however, with all the effective stress being in the mobilised state that is, being carried by the adsorbed water phase. This then explains and confirms the observed stable equilibrium associated with the atmospheric saturation point (point D).

8.2 The intrinsic soil property

The arbitrary water content points (consistency limits) of the investigated soil measured under atmospheric pressure conditions are point A' = 19 per cent, point B = 24 per cent, point C = 28.5 per cent, point C' = 33.3 per cent and point D = 38 per cent. Points F and G were not experimentally determined. As noted in section 8.1.4, the residual effective stress at point D is not removed by water flow. Nevertheless, the significance and water content values of points F and G can be determined with the help of the available data. This requires characterisation of the coupling between soil water content and the internal effective stress.

8.2.1 Coupling between soil water content and internal effective stress

The relationship between incremental water content and incremental effective stress during the swelling process is given in table 8.1. The corresponding stages of the stress changes are indicated in the fourth column.

Table 8.1 Coupling between effective stress and ware content

w/c points	***Δ water content (%)***	***Δ internal stress (kPa)***	***stress condition***
O - A’	19	σ'_i	Internally mobilised effective stress
A’- B	5.0	210	(unloading) pre-swelling stress
B- C	4.5	210	(induction) of soil attraction
C - C’	4.7	210	(neutralising) of soil attraction
C’- D	4.8	210	(reloading) pretension stress
D - F- G	19 + 5	$\sigma'_i + (\sigma'_T)$	residual effective stress (point D)

The water content was physically measured, while the changes of the internal effective stress were deduced from the special condition pertaining to the shrinkage limit as the initial water content (fig. 8.1) as follows. The internal effective stress at the shrinkage limit is 210kPa. Therefore, the pre-swelling stress, pretension stress and the internal effective stress are all equal to 210kPa.

The residual effective stress was mobilised and is retained in the soil at point D. At the same time, the reloaded effective stress at point D is equal to the pretension stress σ'_T. Therefore, the effective stress to be ‘removed’ in phase (D-F-G) is the sum of the initial effective stress and pretension stress. However, the soil does not physically have the pretension stress at point D, as implied in table 8.1. This is because internal reloading of effective-stress (C’-D) is a stress-dissipation process, which involves the removal of soil attraction. Its effect is manifest in the volumetric strain. The physical significance of the pretension stress at point D is in terms of water content change. The water content change corresponding to the pretension stress is five per cent. Therefore, the physical change in the soil beyond point D (38 per cent) is the removal of residual initial effective stress $\sigma'_{r,}$ while the associated water content change is 24 per cent (19+5 per cent).

8.2.2 Definition of the intrinsic soil property

Table 8.1 shows that the internal effective stress is uniquely related to water content The effective stress change is 210kPa and the corresponding water content change is five per cent on average. That is, the rate of change of internal effective stress with respect to water content is a constant and is the same during mobilisation and demobilisation of the internal effective stress. The rate is considered intrinsic to the soil and is herein called the ***pretension rate, K*** (kPa per percentage water content). The pretension rate can be conveniently normalised against atmospheric pressure and expressed as a pure number, herein called the ***pretension number, k***.

For the investigated soil, the pretension rate, K = 42kPa per percentage change in water-content, while the pretension number, k = 0.42. Since the rate is intrinsic, it can be similarly expressed with respect to the degree of soil saturation.

8.2.3 Rationality of the pretension rate, K

The rationality of the pretension number can be demonstrated by consideration of the degree of saturation during pressure gradient wetting. The change in the degree of soil saturation during confined swelling was analysed. The swelling path is most restrictive and hence extreme. The thermodynamic nature of soil swelling is such that the consistency limits are a function of the mobilised hydrotransient (swelling) pressure. Therefore, confined swelling path presence a good test for the uniqueness of the pretension rate and hence the pre-tension number.

Table 8.2 summarises the relationship between the degree of saturation of the soil and the ratio of water content to the saturation water content. The values of the degree of saturation were obtained from the pressure gradient results presented in fig. 4.6 of chapter four. The soil sample had an initial water content of 19.7 per cent, which is reasonably close to the shrinkage limit, for use in comparison. For the purposes of this analysis, the initial water content is considered equal to the shrinkage limit (19 per cent). The corresponding degree of saturation plot is given in fig. 8.3. The atmospheric swelling path describes the pressure-free path along which the consistency limits are determined. The water content at the arbitrary points (consistency limits) was normalised against the water content at point C, the end of the swelling process.

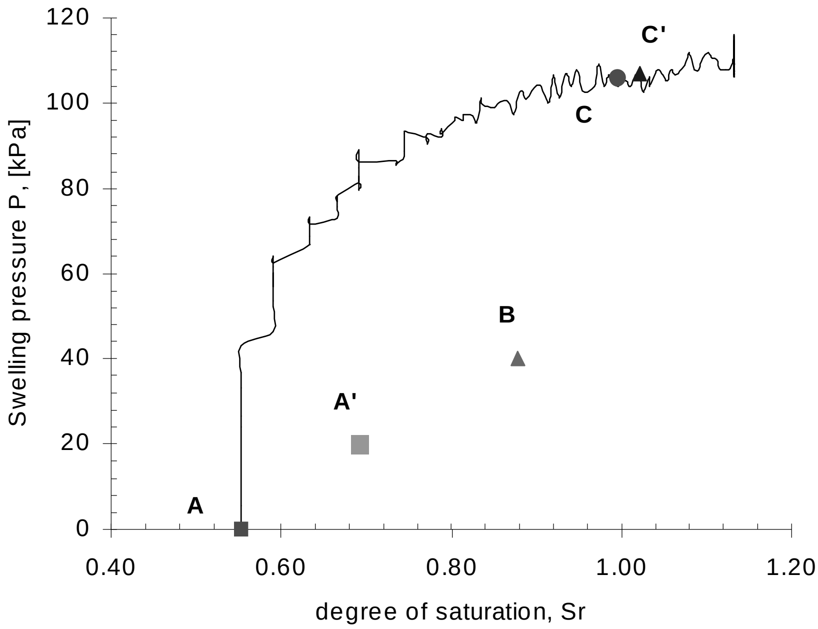

Figure 8.3 Degree of saturation - swelling pressure plot: pressure gradient wetting during confined swelling

Table 8.2 Comparison of degree of saturation and water content ratio

confined swelling (pressure gradient)			***Atmospheric swelling***		
w/c point	*w/c (%)*	*degree of saturation*	*w/c point*	*w/c (%)*	*normalised w/c*
A	**19.7**	**0.558**	A’	19.0	0.667
B	24.0	0.85	B	24.0	0.85
C	28.5	1.00	C	28.5	1.00

According to the coupling implied in the pretension rate, the normalised water content at the consistency limits is equal to the degree of saturation. Comparison between the degree of saturation and normalised water content shows that the two are equal at points B and C. However, this seems not to be the case at point A’ (shrinkage limit), yet it is a point of fundamental significance. It is therefore, asserted that the apparent discrepancy arises from the different reference points in terms of water content but same pressure. The degree of saturation at point A' is referenced to atmospheric pressure at that point (19 per cent water content), whereas the normalised water content at point A' is referenced to atmospheric pressure at zero per cent water content. This is because the saturation water content (28.5 per cent) is tied to the effective stress, which is mobilised from zero per cent water content. Carrying out the following simple mathematical analysis, using the pretension rate proved this assertion. In the process, the rationality of the pretension rate was demonstrated.

8.2.3.1 Degree of saturation at point A’

From phase (B-C) in table 8.2, the degree of soil saturation changes by three per cent for every percentage change in water content. Therefore, the degree of saturation at 19 per cent water content, calculated from zero per cent is 19 x 3 = 57 per cent or 0.570, which shows reasonable agrees with the measured value of 0.558 (table 8.2). Thus, the pretension rate was capable of estimating the initial degree of soil saturation of the soil sample.

8.2.3.2 Normalised water content at point A’

From table 8.2, the change in degree of saturation corresponding to the change in the pretension stress is 15 per cent. Therefore, the corresponding pretension rate is (210/15) = 14kPa per degree of saturation.

Now, the difference between the degree of saturation at point A' and the normalised water content (0.667-0.558) as a percentage is 10.9 per cent. Accordingly, the stress or pressure difference between the two is (10.9 x 14) = 152kPa. If we allow for the build up of pore water pressure to atmospheric value, the pressure difference is 52kPa. This pressure difference is what is recorded at point B in fig. 8.3 (40kPa) as swelling pressure. It is not recorded directly at point A' because though the mobilisation of effective stress (soil cohesion) starts at point A', it is cushioned by the excess pore water. The mobilised effective stress associated with the water uptake at point A' (initial condition) becomes active at point B, when the pore water is depleted. Thus, the 40kPa swelling pressure at point B is really a measure of the pressure associated with the initial condition at point A'. The apparent difference of 12kPa is largely because the current expression of determining degree of saturation assumes constant (atmospheric) pressure. This also explains why the water content points are not coincident with the swelling pressure results.

8.2.3.3 Comment

It is remarkable that the discrepancy between the measured values of the degree of saturation at the shrinkage limit (point A') and the normalised water content is reasonably accounted for by using the pretension rate. Thus, the pretension rate was able to reconcile swelling pressure, degree of saturation and soil water content in a consistent manner. The analysis vindicates the uniqueness of the pretension rate. Therefore, ***the pretension rate is the intrinsic soil property***. Accordingly, it forms a rational basis for formulating internal effective stress models of expansive soils.

8.3 Laboratory determination of the intrinsic soil property

The pretension rate is a ratio of the pretension stress to the corresponding change in soil water content. It is desirable to determine both the pretension stress and the associated water content points from routine laboratory tests. This greatly simplifies the characterisation of the expansive soil. One way to determine the pretension stress in a routine manner is to capitalise on its analogous role with preconsolidation pressure. Accordingly, the preconsolidation pressure of the investigated soil was determined and compared with the pretension stress. For water content change, advantage was taken of the rationality of the consistency limits and specifically those at points A' and B.

8.3.1 Determination of the preconsolidation pressure

The preconsolidation pressure of the investigated soil was determined from the continuous loading (CL) consolidation test results. It is one of the standard test procedures at NTNU. Details of the test equipment, laboratory test-procedure and theory for interpretation of test results are given by Janbu *et al*. (1981). The soil sample was fully saturated before the test. The strain rate used for the test, after several trials was one per cent per hour.

8.3.2 Test results

The continuous loading (CL) consolidation test results are presented in fig. 8.4, in the form of the tangent modulus plot.

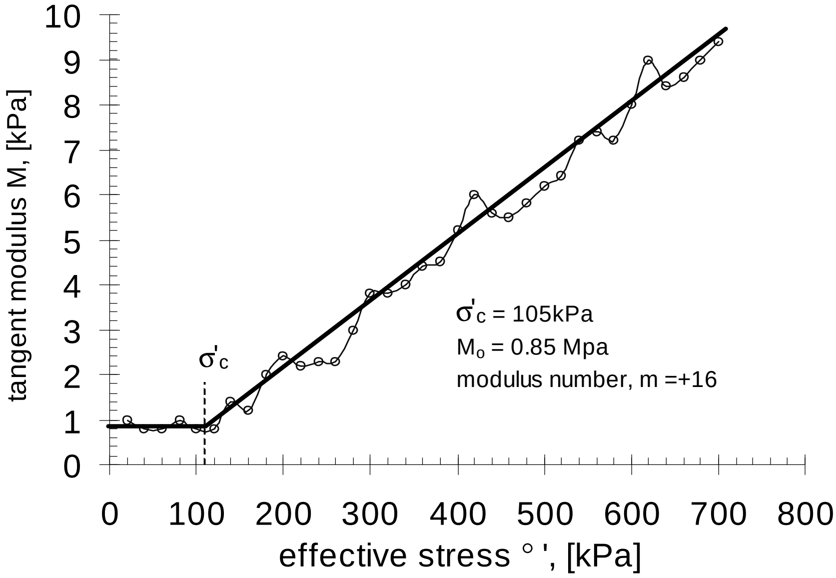

Figure 8.4 Consolidation test results on the investigated expansive soil.

8.3.3 Analysis

Fig. 8.4 reveals a well-defined change in soil response at 105kPa stress-level. Accordingly, ***the preconsolidation pressure is 105kPa.*** The initial ***soil stiffness is 0.85MPa***, while the ***modulus number, m=+16***. The significance of these parameters is discussed below in the context of soil swelling, with particular reference to figure 8.5. Figure 8.5 is a reproduction of the reloading modulus in fig. 8.1.

8.3.3.1 Initial soil stiffness

The initial value of the initial soil stiffness (fig. 8.4) is 0.85MPa, while the extrapolated residual soil stiffness, measured at point F during the reloading part of soil swelling (fig. 8.5) is 1.0MPa. Point F represents the liquid limit. Therefore, the soil was fully saturated when it was compressed.

8.3.3.2 Modulus number, m

The modulus number from fig. 8.4, ***m = +16***. The reloading modulus from fig. 8.5, $\boldsymbol{m_r = -16}$**.** These observations clearly show that oedometer compression of the saturated soil 'reverses' soil swelling from point F towards point D. This assertion is further reinforced by noting the values of the soil stiffness discussed above.

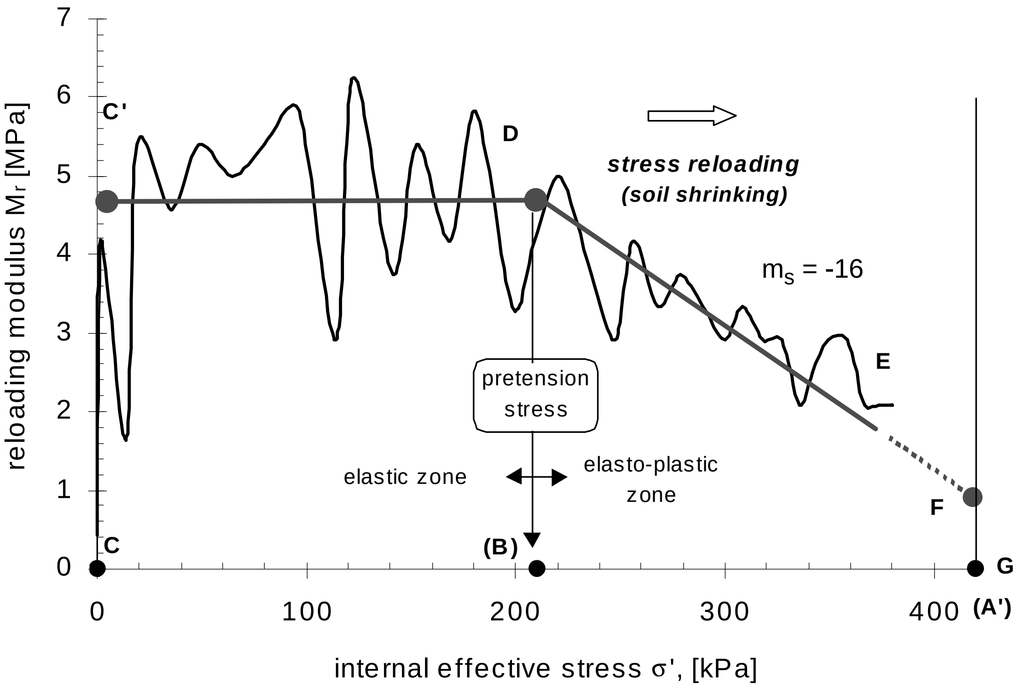

Figure 8.5 Reloading modulus plot. Initial water content = shrinkage limit

8.3.3.3 *Preconsolidation pressure*

The significance of the preconsolidation pressure should be viewed in the light of the following: The preconsolidation pressure, as determined above, is effective in that the pore water pressure is deducted from the applied stress. However, in expansive soils the adsorbed water offer resistance in the form of water pressure, yet it is not dissipated. It does not dissipate because the adsorbed water is electrochemically attached to the clay mineral and can not be removed mechanically. Therefore, the preconsolidation stress as determined, does not account for the mobilised effective stress (effective stress in the adsorbed water layers). It is related to the internal effective stress (solid clay mineral contact).

<u>Mobilised preconsolidation pressure is equal to the pretension stress</u>

The pretension stress is a composite effective stress from composite soil particles (hydrated clay minerals). It is the sum of the ‘internal’ stress (interaction between the solid clay minerals) and the mobilised stress (interaction between the hydrated clay minerals). A tension ratio of two (fig. 8.1), indicates that the effective stress at the shrinkage limit is equally distributed between the water and solid parts of the soil particles. This agrees well with the observation that the pre-swelling pressure is equal to the pretension stress.

Therefore, the preconsolidation pressure (105kPa) is half the pretension stress (210kPa). The other half is not reflected in the preconsolidation pressure because the adsorbed water carries it and was considered excess pore-water pressure. It seems to suggest that if the residual water pressure in the adsorbed water is included in the determination of preconsolidation. Then, the preconsolidation stress becomes equal to the pretension stress. Using the incremental loading oedometer compression test could validate this.

Consequently, the preconsolidation pressure as determined in sub-section 8.3.1, is referred to as ***effective preconsolidation pressure***. The term sounds like a double emphasis, but it is considered necessary here, in order to distinguish it from that which includes the mobilised effective stress. The preconsolidation stress, which includes the mobilised effective stress, is similarly called ***mobilised preconsolidation pressure*** or simply preconsolidation pressure. Accordingly, ***the mobilised preconsolidation pressure is equal to the pretension stress***. Therefore, pretension stress can be determined from the consolidation test results.

8.3.4 Expression for the intrinsic soil parameter, k

The pretension number, *k* is the characteristic soil parameter and hence most important in characterising the swelling soil behaviour. It can be obtained from routine test data as follows. The pretension stress is equal to the preconsolidation pressure. The change in the pretension stress occurs between water content points A’ and B, which are respectively, the shrinkage (SL) and plastic limits (PL) of the soil. Therefore, the expression for the determination of the pretension rate, K is

$$Pretension\ number, k = \frac{2\sigma'_c}{(PL - SL)} = \frac{\sigma'_{cm}}{(PL - SL)} \qquad \text{(Equation 8.1)}$$

In equation 8.1, σ'_c is the effective preconsolidation pressure and σ'_{cm} is the mobilised preconsolidation pressure. The pretension rate obtained from equation 8.1 was used to generate the different forms of the effective stress principle for the investigated soil, which are presented in chapter nine.

8.4 Conclusions

- The link between the water content points is a consequence of the change in the internal effective stress of the soil. Accordingly, the change in soil consistency during soil wetting is governed by the change in internal effective stress.
- The consistency limits have fundamental significance in the swelling behaviour of the investigated soil. They relate the effective stress in the soil to the physical changes in the soil.
- The linear shrinkage limit is the most rational limit and is reproducible. Accordingly, the shrinkage limit of the investigated soil can be used as a fundamental parameter that links the soil consistency to internal effective stress changes.
- The pretension stress of the investigated soil is 210kPa.
- The intrinsic soil property of the soil was defined and determined. It is a rate quantity and was called ***pretension rate, K***. It was conveniently normalised and expressed as a dimensionless number called the ***pretension number, k***.
- The preconsolidation stress of the investigated soil is uniquely related to the pretension stress. Thus, the intrinsic soil property can be easily determined from routine laboratory test results.

Chapter 9

Simulation of internal effective stress changes during soil wetting

In this chapter is presented, for the first time ever, the application effective stress principle to an expansive swelling soil. The effective stress principle is presented in its basic form and its applicability to expansive soils qualified. With the help of the intrinsic soil-property defined in chapter eight and the resistance concept, the expressions for the effective stress principle for a expansive soil are presented. Thereafter, the input parameters are given. This culminates in the simulation of the internal effective stress changes (interaction mechanism) during soil swelling. The chapter concludes by rationalising the liquid limit in terms of effective stress change.

9.1 The principle of effective stress

Terzaghi's (1936) principle of effective stress successfully describes the stress-strain behaviour of fully saturated soils. His original 1936 statement states in part, that

> …. all measurable effects of a change of stress, such as compression, distortion and changes of shearing resistance are exclusively due to changes in the effective stresses σ_1', σ_2', and σ_3'.

The following important deductions can be made from this statement, and form a basis for extending the effective stress principle to expansive soils.

- The effective stresses in a soil element remain the same during any loading or unloading process on the soil element, where there is no volume change and strain development.
- If the effective stress is increased, the soil will compress. Conversely, if the effective stress is reduced, then the soil will swell, i.e. increase in volume.

9.1.1 Application to expansive soils

From the above deductions, the swelling behaviour of the investigated soil is consistent with the principle of effective stress. As was experimentally shown, the unloading (mobilisation) and reloading (dissipation) of the internal effective stress is responsible for soil swelling. In the absence of external static loads, as is the case with internal soil swelling, the pore water does not mobilise positive hydrostatic pressure. Rather, it leads to hydrotransient pressure development, which is a measure of the mobilised effective stress. Accordingly, the internal effective stress has two forms, the ***internal effective stress*** and the ***mobilised effective stress***. Consequently, the principle of effective stress can be stated in two forms namely, the internal and mobilised forms. The internal form applies to isotropic stiffness, while the mobilised form relates to shear deformation. The principle was consistently applied to take account of the dynamic nature of the internal effective stress.

9.2 Formulations of the effective stress principle

9.2.1 The internal form of the effective stress model

The ***pretension number, k*** capsulate the change in the internal effective stress. Therefore, the internal effective stress model is

$$\Delta\sigma'_i = k\Delta w \qquad \text{(Equation 9.1)}$$

In equation 9.1, k is the pretension number and Δw is the fractional change in soil water content, in a given swelling phase. Equation 9.1 is valid for the water content range between the shrinkage limit and atmospheric saturation water content. Thus, the change in the internal effective stress is step wise, changing at the consistency limits.

9.2.2 The mobilised form of the effective stress model

Mobilised effective stress (soil cohesion) is responsible for the soil resistance to shear deformation by water flow. Accordingly, the normalised soil resistance is the appropriate resistance modulus. The mobilised effective stress equation is obtained as follows.

By definition, soil resistance,

$$100S = \frac{dw}{dP}, \quad while \quad 100\frac{dS}{dw} = s \text{ (soil number)} \qquad \Rightarrow S = s\frac{w}{100}$$

On substituting for S in the definition and integrating over appropriate limits, we obtain equation 9.2.

$$\Delta\sigma_w{}' = s\frac{w}{100} = \Delta P = \frac{100}{s}In\left(\frac{w}{w_o}\right) \qquad \text{(Equation 9.2)}$$

In equation 9.2, w_0 = reference water content, w is the current water content and s is the soil parameter. The constant value of 100 is the atmospheric pressure value used to normalise the soil resistance. The arbitrary water content points lend themselves as natural reference water content points, given that they define the change points for the soil numbers.

Equation 9.2 holds for the mobilisation and demobilisation of the effective stress except for the phase immediately after yield point (C’-D). In this phase, the mobilised effective stress is elastically reloaded and manifests at point D as pretension stress. Accordingly, the increase in internal effective stress dominates the phase and overshadows the removal of mobilised effective stress. Consequently, the demobilisation of effective stress in this phase is in accordance with equation 9.1 however, with an appropriate constant in place of the pretension number, k.

9.2.3 Hydrotransient pressure, U_p

The composition of hydrotransient pressure changes during the swelling process. The unloading and reloading of the internal effective stress underlies the change. Accordingly, two different expressions are used.

9.2.3.1 Pore water pressure stage

The build up and dissipation of excess pore water pressure characterises the initial mobilisation of hydrotransient pressure. The appropriate modulus is the stress modulus, M_s and the appropriate equation is obtained from the definition of the stress modulus as follows.

By definition the stress modulus, $M_s = \frac{dP}{dw} \quad \Rightarrow dP = M_s dw$

Upon integration over the appropriate limits, we obtain equation 9.3.

$$\Delta P = \Delta U_p = M_s(w_1 - w_0) \qquad \text{(Equation 9.3)}$$

In the equation, expression, M_s is the initial stress modulus and w_o and w_1 are the initial and final water content, respectively. The initial stress modulus corresponds to the initial soil stiffness during oedometer compression (see sub-section 8.3.3 of chapter eight).

9.2.3.2 Induction stage

The presence of soil attraction induces the continued development of hydrotransient pressure. The development of hydrotransient pressure becomes non-linear with respect to water content. Therefore, the appropriate equation is obtained by using the derivative of the stress modulus as follows:

By definition, $M_s = \frac{dP}{dw} \quad \textit{and} \quad \frac{dM_s}{dw} = m_s \quad \Rightarrow M_s = m_s w$

We substitute M_s in the first expression of the definition above to obtain an expression in terms of P, w and m_s. Upon integrating the resulting expression we obtain

$$\Delta P = \Delta U_p = \frac{m_h}{2}(w_1 - w_0)^2 \qquad \text{(Equation 9.4)}$$

In equation 9.4, w_0 = reference water content, w_1 is the current water content and m_h is the stress number corresponding to hydrotransient pressure. The arbitrary water content points lend themselves as natural reference water-content points.

9.3 The input soil parameters

9.3.1 Internal effective stress (soil attraction)

From the analysis in section 8.4.2 the soil parameter for internal effective stress is k = 0.42. However, k = 0 in phase (C-C'), where the internal effective stress is a constant.

9.3.2 Mobilised effective stress (soil cohesion)

The soil parameters during soil swelling are the dimensionless numbers and the initial moduli. The measured parameters obtained along the continuous swelling path are the appropriate values, from which we can determine the soil parameters. The results obtained along pressure gradient wetting are used to determine the soil parameters. Accordingly, table 5.5 of chapter five is reproduced in table 9.1.

Table 9.1 Summary of soil parameters (from table 5.5 in chapter 5)

Phase	***soil number, s***	***stress number, m_s***	***water content (%)***
A-A'	0	0	10-14
	-0.54	-0.4	14-18.5
A'-B	+0.54	+4.0	18.5-21.5
	-0.54		21.5-23.5
B-C	-0.27	-3.8	23.5-27.5
C-C'	-0.09	+1.2	27.5-32.5

Table 9.1 Summary of soil parameters (from table 5.5 in chapter 5)

Phase	***soil number, s***	***stress number, m_s***	***water content (%)***
C'-D	-0.54	-	32.5-

The soil number as determined, relates to the soil resistance and hence stiffness of the soil. Since the mobilised effective stress is responsible for soil stiffness, it follows that the directly measured soil number relates to mobilised effective stress. However, the soil numbers in some phases relates to the combined effect of the induction phenomenon or build up of pore water pressure. Therefore, it was necessary to decompose the number according to the different forms of the effective stress. The decomposition of the soil parameters is explained in the following paragraphs, while the final values are summarised in table 9.2.

Phase (A-B). The phase is associated with the build up of excess pore water pressure, as intimated by the reversal of the soil number in the phase. Accordingly, the switching of the soil number means that the appropriate soil parameter is the difference between the two. Since the numbers are equal and opposite, the removal of soil attraction (reversal of the induction phenomenon) has a doubling effect. Therefore, s = +1.08. The parameter appears as a positive value because the pore water pressure is measured as a positive quantity.

Phase (B-C). Phase (B-C) is associated with the induction of the internal effective stress (soil attraction). From table 9.1, the stress number relating to soil attraction m_s = -0.4, which is the intrinsic (pretension) number (k = 0.42). Now hydration and induction phenomena are of a different nature. As such, their effect on the soil is not coupled. Accordingly, the appropriate soil number is the sum of the compound soil number and the pretension number. That is, s = {-0.027+0.4)} = +0.373.

Phase (C-C'). In this phase, internal effective stress is a maximum and the pretension number is therefore, zero. The stress number (m_s = +1.2) is close to one and thus shows that the phase is associated with change of effective stress from one form to another within the water phase. Accordingly, the pretension number does not affect the mobilised effective stress. Therefore, the appropriate soil parameter for mobilised effective stress is likewise, s = (+0.09 - 0) = +0.09. The relatively very high soil number agrees with the slow change in soil structure involving the reloading of the residual effective stress. This intimates that the soil is very stiff and hence does not significantly change in structure.

Phase (C'-D). In this phase, the internal effective stress is being removed and is therefore passive. Meanwhile, the reintroduction of liquid water in the soil (sub-section 5.4.4 of chapter five) amounts to a building up of pore water pressure. It increases to atmospheric value at point D.

The phenomenon of removing soil attraction and simultaneous building up of pore water pressure is analogous to that in phase (A-B). The analogy is further highlighted by the identity in the soil numbers. Therefore, there is a doubling effect on the soil number (rate of change of soil structure) as in phase (A-B). However, in this instance the soil is softening. Accordingly, an increase in rate of soil softening means a small soil number. In other words, the effect is to double the rate, which means the compound soil number is halved. Therefore, the correct value of the soil number is s = +0.54/2 = +0.27.

Phase (D-F). Phase (D-F) involves the removal of the mobilised effective stress equal to the initial effective stress. The removal takes place at atmospheric pressure, a condition similar to that during internal (initial) mobilisation. It is reasonable to say that the rate of demobilisation is the same as during internal mobilisation. This is because there is no build up of excess water to influence the rate. The internal mobilisation rate is governed by and hence is equal and opposite to the removal of soil attraction. Therefore, s = +4.1.

9.3.3 Hydrotransient pressure

Hydrotransient pressure is water pressure and is therefore important in the dynamics of the internal effective stress. The characteristic parameter relating to hydrotransient pressure is herein called ***pressure number, m_h***. It is of the same family as the stress number. However, the stress number is decomposed as necessary, to remove the effects of effective stress.

Phase (A-B). Up to point B, hydrotransient pressure manifests as excess pore water pressure. Accordingly, the appropriate parameter is the initial stress modulus, M_s corresponding to maximum mobilised effective stress (soil cohesion). Thus, the maximum pore water pressure occurs just before point B, when soil cohesion becomes a maximum.

Meanwhile, the soil stiffness at the preconsolidation pressure is a measure of the stress modulus at water saturation, which is equal to that associated with maximum soil cohesion or initial stress modulus. From section 8.3.2 of chapter eight, the stiffness is 0.85MPa. However, the stiffness due to the adsorbed water phase is half that is, M_s = 0.42MPa

Phase (B-C). During this phase, the adsorbed water layers play a double role in terms of carrying the effective stress. Firstly, the effective stress is carried as actively mobilised effective stress (soil cohesion). Secondly, the residual effective stress is carried by the water phase as osmotic potential (interparticle repulsion). That is, the water layers are the seat of the net charge that gives rise to the repulsion. Since, hydrotransient pressure is a measure of and thus reflects the internal effective stress, it is reasonable to say that the parameter relating to hydrotransient pressure is double that of the internal effective stress (k = 0.42). Therefore, m_h = +0.82.

Phase (C-C'). According to the particle interaction model, phase (C-C') is associated with the diffusion of squeezed out water. As such, the diffusion of the squeezed out water characterises the hydrotransient pressure. The parameter relating to water is s = -0.54 (phase (A-A')). However, the diffusion of the squeezed out water is a repeat process with respect to the movement of the squeezed out water. Therefore, there is a cancelling effect. Consequently, $m_h = (-0.54 + 0.54) = 0$.

Phase (C'-D). The increase of hydrotransient pressure is dependent on the change in internal effective stress (soil attraction). The coupling between the two is such that the latter is reflected in the former. In this phase, the internal effective stress is being removed and is therefore passive. Therefore, the rate of build up of hydrotransient pressure is equal to that expressed in the stress number in the phase. That is, $m_h = +0.52$.

Table 9.2 Summary of soil parameters for the effective stress models

phase	***water content (%)***	***'k' for the internal effective stress model***		***'s' for the mobilised effective stress model***		***'m_h' for hydrotransient pressure***	
		data	***refined****	***data***	***refined****	***data***	***refined****
A'-B	19.0-24.0	-0.4	-0.42	+0.104	+0.101	0.42MPa	4.2MPa
B-C	24.0-28.5	+0.4	+0.42	+0.373	+0.36	-0.82	-0.85
C-C'	28.5-33.5	0	0	-0.09	-0.083	0	0
C'-D	33.5-38.0	-0.4	-0.42	-0.27	-0.27	+0.52	+0.54
D-F	38.0-57.0	0	0	-4.1	-4.1	+8.0	+8.2

* the value used to generate the interaction mechanism.

Phase (D-F). Removal of hydrotransient pressure is related to the removal of the initial effective stress. The residual effective stress was mobilised (activated) during phase (B-C). Its mobilisation led to build up of pressure excess of atmospheric value. Now the pressure is demobilised at atmospheric pressure. For the water pressure to remain atmospheric, the effect of removing the excess effective stress on the water pressure should be constant. In other words, the relationship between hydrotransient pressure and atmospheric pressure must be constant. Therefore, the parameter is the normalised value of the mobilised effective stress. That is, $m_h = +(798/100) = +8.0$

9.4 Simulation of the internal effective stress changes during soil wetting

The internal effective-stress (soil attraction) and mobilised effective stress (soil cohesion) make up the effective stress of the soil. Apart from this, the action of water on the soil leads to a build up of hydrotransient pressure. Accordingly, the interaction mechanism presented herein shows the development of the three with the changing soil water content. The appropriate soil parameters from table 9.2 were used in the effective stress equations developed in section 9.2 to generate the interaction model in fig. 9.1. The model presented in this figure is that of the soil wetted by flooding path and it is plotted to scale. The interaction mechanism was developed as follows.

9.4.1 Internal effective stress

The internal effective stress at 19 per cent water content is the pretension stress and is equal to 210kPa. The subsequent increase in adsorbed water layers increases the interparticle distances and thus causes downloading of the internal effective stress. It is completely downloaded at 24 per cent water content, when the soil water potential is exhausted. However, it is immediately induced and starts to re-mobilise. Thus, the internal effective stress re-mobilises to a maximum value at the pressure saturation point (28.5 per cent).

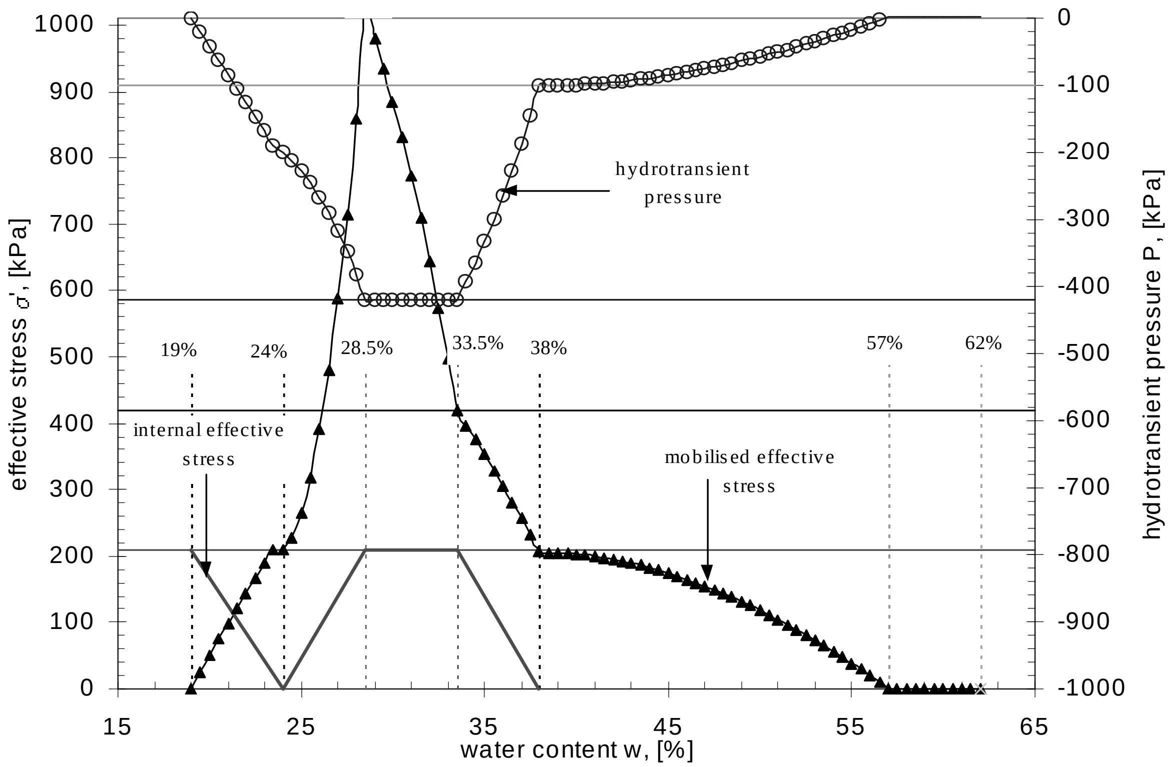

Figure 9.1 Coupling between the different forms of effective stress and water content

At 28.5 per cent, the soil has a very high ionic concentration gradient. The concentration gradient leads to water flow by diffusion. Thus, water comes between the clay minerals and offsets the internal effective stress. This is achieved by the stacking of water molecules between the clay minerals, until the interparticle distance equal the molecular range of the clay minerals. The process is equivalent to the removal of the internal shear strength of the soil, arising from the internally mobilised effective stress. However, until the interparticle distance is equal to or greater than the molecular range, soil attraction and hence internal effective stress remains a maximum. Accordingly, the internal effective stress remains constant. The shearing becomes limiting, when the interparticle distance is equal to the molecular range of the clay minerals. At that point (33.5 per cent), the soil yields and the internal effective stress starts to decrease. Thus, the soil is capable of deforming into any shape under a stress excess of the yield value, without disturbance to the soil structure. This condition is called plasticity and the shearing phenomenon is called plastic flow. Therefore, the internal effective stress is constant during plastic shear (28.5-33.5 per cent). The decrease in soil attraction beyond the yield point is governed by the amount of water creating and occupying the interparticle space. As such, the internal effective stress linearly decreases in response to increase in water content. It is completely removed at 38 per cent. However, the soil does not disintegrate because of the mobilised stress of the soil.

9.4.2 Hydrotransient pressure

In the presence of excess pore water, the downloaded effective stress at 19 per cent causes a build up of negative pore water pressure in the soil. Accordingly, hydrotransient pressure initially manifests as excess pore water pressure. Being water pressure and being dependent on the linear growth of the adsorbed water layers, it linearly increases (negatively) with water content up to 24 per cent. At 24 per cent, the pore water depletes. Depletion of pore water leads to direct particle interaction and hence continued mobilisation of hydrotransient pressure. This is a consequence of its dependency on soil attraction (initial effective stress), which mobilises from 24 per cent water content. Accordingly, the pressure slowly mobilises as indicated by a change in the slope of the curve from 24 per cent. It becomes a maximum at the pressure saturation point. The magnitude of the maximum value of hydrotransient pressure, recorded between 28.5 and 33.5 per cent, is twice the pretension stress value (210kPa), as is reflected by the results plotted in fig. 8.2 of chapter eight.

Thereafter, hydrotransient pressure decreases (negatively) at the same rate that water content offsets internal effective stress. This is because the increasing water content has the same effective on both soil attraction and hydrotransient pressure. At 38 per cent, the pressure is atmospheric as expected. Beyond 38 per cent, its change is in response to the removal of the mobilised effective stress. Accordingly, it is non-linear.

9.4.3 Mobilised effective stress

The effective stress carried by the water phase constitutes the mobilised effective stress. It is made up of the initial effective stress at 19 per cent water content and the effective stress mobilised between 19 and 24 per cent. The former is initially internal and hence inert. Consequently, the reference value of the mobilised effective stress at 19 per cent water content is zero. However, it is activated by the induction of soil attraction at 24 per cent. Accordingly, it is ‘mobilised’ between 24 and 28.5 per cent. Since the mobilisation process involves removal and conversion of the internal effective stress, it non-linearly increases with water content from 19 per cent. It attains a value equal to the internal effective stress at 24 per cent because the internal effective stress is completely downloaded and mobilised. Thereafter, it rapidly increases after 24 per cent water content, despite the exhaustion of the soil water potential. This is due to the activation of initial effective stress by soil attraction. The re-mobilisation of the initial effective stress is completed on attainment of pressure saturation at 28.5 per cent. The mobilised effective stress increases beyond the pretension stress value by an amount equal to the residual effective stress. The residual effective stress is much higher than the pretension stress because it corresponds to 19 per cent change in water content (initial water content), compared to the 5 per cent change in the case of pretension stress.

The decrease in the ionic concentration gradient from 28.5 pre cent has an immediate and hence measurable effect on the mobilised effective stress. Accordingly, the mobilised effective stress decreases. The rate of decrease is fast because it is tied to the removal of soil attraction, which depends on the rapid decrease in the ionic concentration (water content) gradient. At the yield point, the internal effective stress starts to demobilise, while the mobilised effective stress slowly becomes active. In addition, the water content gradient is significantly reduced. Therefore, the removal of the mobilised effective stress is much slower than before. However, its removal linearly relates to the increase in water content up to 38 per cent, a consequence of the influence of the internal effective stress.

The mobilised effective-stress at 38 per cent is equal to the initial effective stress at 19 per cent however, in a mobilised form. Accordingly, it is equal to that at 24 per cent water content. It then non-linearly reduces to zero at 57 per cent. The observation in fig. 8.3 (chapter eight) shows that it takes a water content increase of 19 per cent from 38 per cent, to remove the effective stress from the soil. This agrees with the assertion that the decrease in internal stress beyond 38 per cent is a removal of the initial effective stress, which is mobilised at 19 per cent. The decrease is much slower and non-linear as indicated by the slope of the curve. This is not surprising because it is removal of the effective stress, which is stored as soil cohesion. In addition, the effective stress and the soil structure are in equilibrium with atmospheric pressure and therefore form a very stable soil structure. Accordingly, it takes a significant amount of water to separate the clay minerals and hence to destroy the stable, electrochemically bonded soil structure.

9.4.4 Zero internal effective stress

The mobilised effective stress reduces to zero at 57 per cent. The presence of soil stiffness (F-G) in fig.8.3 shows that the disintegration of soil structure during soil wetting is not simultaneous with the removal of effective stress. This is because the basic soil particles (hydrated clay mineral) is a composite particle. Accordingly, disintegration of the soil is in two stages.

9.4.4.1 Liquid limit

The first stage is the cessation of interparticle contact. Interparticle contact is associated with soil cohesion and hence the mobilised effective-stress. Accordingly, this is achieved at 57 per cent water content.

The liquid limit. It is herein defined as the water content at which the soil losses its resistance to flow or shear deformation. Shear resistance to flow is attributed to effective stress. Therefore, the condition of zero internal effective stress in the soil is sufficient to define the liquid limit. Consequently, the liquid limit (point F) of the investigated soil is 57 per cent.

9.4.4.2 Particle disintegration

The second stage involves the separation of the water phase (adsorbed water layers) from the individual solid clay minerals. The water layers do not “fall off” from the solid particle at zero effective stress because of the induction effect of soil attraction during phase (B-C). Soil attraction presses the water layers against the soil skeleton. The stiffened water layers are ‘glued’ to the soil skeleton. However, the effect is purely physical and can be physically reversed. As noted earlier, the compression squeezes out five per cent water from the water layers. Accordingly, the soil requires an additional five per cent increase in water-content to fully recover from the impact of the induction phenomenon. The process is herein called ***particle disintegration***. Therefore, the particles start to disintegrate as 62 per cent. This is a 24 per cent-water content increase from the atmospheric saturation point. In retrospect, the 24 per cent water content change agrees with the fact that the same amount of water used to unload the effective stress from the soil (0-24 per cent), is required to remove its effect from the soil. It is measured from the atmospheric pressure point, following the complete removal of the effect of soil attraction.

According to the SBS physical soil model, the mobilisation of soil cohesion and soil attraction is modelled as the evolution of the SBS chamber. Now, the evolution of the SBS under forced water flow (flooding) is an unnatural and traumatic event for the soil. As such, it takes a complete revolution (100 per cent water change) for the soil to erase the impact (at 24 per cent water content) from its memory. Accordingly, particle disintegration is considered a removal of the tattered SBS chamber, now that the mobilised effective stress is completely reloaded. Thus, the additional five per cent water content is meant to disintegrate and detach the mesh of adsorbed water layers from the soil skeleton. Being essentially water, the mesh does not provide shear resistance. However, it is capable of offering stiffness by virtue of its volume and density. Accordingly, the soil retains a stiffness modulus of 8MPa at the liquid limit (point F). This is the bulk modulus of the adsorbed water layers. Its subsequent removal is a plastic process because it does not involve change in effective stress.

Disintegration point. The disintegration point is herein defined as the water content, where the adsorbed water and the individual clay minerals start to separate. It is higher than the liquid limit by an amount equal to the water content range over which the impact of soil attraction is registered (phase B-C). Therefore, the disintegration point (point G) is 62 per cent. Complete particle disintegration similarly requires five per cent water content change. Accordingly, particle disintegration starts at 62 per cent and completes at 67 per cent water content.

9.4.5 Rationality of the liquid limit and disintegration point

The water content of 57 per cent (point F) is three times the shrinkage limit. That is, the soil requires the same amount of water to (i), internally download the internal effective stress (0-19 per cent), (ii) re-mobilise and demobilise the internal effective stress (19-38 per cent) and (iii), to remove the downloaded effective stress (38-57 per cent). Thus, its link with the shrinkage limit is a feature that completely vindicates it as characteristic limit in the soil response. Apart from this, the water content is twice the pressure saturation point. This is reasonable because the same amount of water used to mobilise maximum effective stress (saturation point) is required to completely remove the stress.

Now, the soil mobilises maximum effective stress at the saturation point and yield at the yield point. Similarly, it attains minimum effective stress at the liquid limit, before it disintegrates at the disintegration point. In view of this analogy, it reasonable to say that the plastic soil response (F-G) in fig. 8.3 is a yielding of the soil structure analogous to the yielding of the soil strength (C-C’). This makes point *G* a yield point in terms of soil structure. The yielding phenomenon is consistent with the complete change in the physical state of the soil thereafter. Since the liquid limit is twice the saturation point, it is reasonable to say that the disintegration point (structure) is twice the yield point (strength). This lead to water content value of 66 per cent. The value is reasonably close to the upper limit of the disintegration process (67 per cent). Clearly, the values of the liquid limit and the disintegration point are rational. In addition, the existence of the disintegration point is vindicated.

Therefore, the internal effective stress-strain behaviour of the investigated expansive soil was characterised thus.

9.5 Conclusions

- The link between the water content points is a consequence of the change in the internal effective stress of the soil. Accordingly, the change in soil consistency during soil wetting is governed by the change in internal effective stress.
- The consistency limits have fundamental significance in the swelling behaviour of the investigated expansive soil. They relate the effective stress in the soil to the physical changes in the soil.
- The linear shrinkage limit is the most rational and is reproducible. Accordingly, the shrinkage limit of the investigated soil can be used as a fundamental parameter that links the soil consistency to internal effective stress changes. However, the limits require redefinition.
- The attainment of the liquid state of the soil is not simultaneous with the disintegration of the soil structure. This is because the elementary soil particles are composite particles, made up of the clay mineral and the adsorbed water. Thus, the hydrated clay minerals disintegrate only after the disintegration of the soil structure.
- The disintegration of the soil particles is a process, which occurs after the liquid limit. It requires twice the amount of water squeezed out of them during the induction phenomenon.
- The preconsolidation stress of the investigated soil is uniquely related to the pretension stress.

- The attainment of the liquid state of the soil is not simultaneous with the disintegration of the soil structure. This is because the elementary soil particles are composite particles, made up of the clay mineral and the adsorbed water. Thus, the hydrated clay minerals disintegrate only after the disintegration of the soil structure.
- The disintegration of the soil particles is a process, which occurs after the liquid limit. It requires twice the amount of water squeezed out of them during the induction phenomenon.

Chapter 10

Summary, conclusions and recommendations for further research

10.1 Summary

The research work presented in this thesis used the resistance concept to validate the induction concept, a new concept of visualising the swelling phenomenon. The resistance analysis successfully rationalised the new concept as formulated. This led to an understanding of the fundamental swelling soil behaviour from a soil resistance perspective. The pertinent aspects of the research can be summarised as follows.

- A split ring oedometer, capable of measuring horizontal stress, was adapted with minimum modifications, to suite the laboratory testing of an unsaturated expansive soil.
- A comprehensive laboratory test programme was designed to characterise the mechanical behaviour of an unsaturated expansive soil. The results were analysed using the resistance concept. New definitions of the resistances appropriate to swelling soils were defined and rationalised. The resistance concept was used for the first time to analyse expansive soil behaviour
- Following a critique of the soil water potential, a new concept for visualising the particle-level interaction phenomenon was formulated. For the first time, the adsorptive forces (soil potentials) were decomposed and characterised from a mechanistic point of view. Soil attraction and soil adhesion are the primary potentials. The two complement each other. Soil cohesion and repulsion were considered secondary potentials.

- A dynamic equilibrium between the soil potentials, soil water content and soil structure was postulated. The interactions constituted an induction phenomenon, hence the name ***induction concept***. The concept was validated using the test results.
- The formulations of a soil particle model and a dynamic soil structure model were based on the induction concept. The models are compatible with conventional understanding of soil behaviour.
- The induction concept was used as a base to postulate an effective stress hypothesis. The hypothesis postulated that (a) soil attraction is the internal effective stress, (b) soil adhesion is potential internal effective stress, (c) soil cohesion is mobilised effective stress and (d), swelling pressure due to particle repulsion is isotropic pore-water pressure. The hypothesis was validated using the test results.
- The change in particle-structure is reflected in the change in soil consistency (structure) and was quantified by means of a physical soil model.
- The swelling soil behaviour under three unique swelling paths was investigated. The paths are confined (no volume change), atmospheric swelling (pressure increases to atmospheric value) and continuous swelling (pressure, volume and water flow simultaneously changed). The analysis rationalised the induction concept, the physical soil model and the effective stress hypothesis.
- A new conceptual model for handling the internal effective stress of the soil, as a continuous function of water content was developed. It was subsequently used in characterising the effective stress- strain behaviour of the investigated soil.
- The effective stress-strain analysis revealed a unique relationship between internal effective stress, water content and soil structure (soil consistency). The consistency limits were thus rationalised. It also led to a new definition of the intrinsic soil property.
- The intrinsic soil property of the investigated soil was rationalised. It can be easily determined from test results obtained from routine laboratory equipment.
- Appropriate formulation of the effective stress principle for the investigated expansive soil was developed for the first time. It accounts for the dynamic relationship between water flow and change in effective stress.
- The effective stress model was used to simulate the changes in the internal effective stress of the investigated soil during soil wetting.

10.2 Conclusions

- The proposed induction concept for characterising particle-level behaviour of expansive soils was successfully rationalised using the laboratory tests results. Thus, the induction concept is a viable alternative concept for visualising the particle-level phenomenon of the investigated expansive soil.
- The effective stress hypothesis was successfully validated. Therefore, the effective stress principle can be applied to the investigated soil as follows:(a) soil attraction is the internal effective stress, (b) soil adhesion is potential internal effective stress, (c) soil cohesion is mobilised effective stress and (d) swelling pressure due to particle repulsion is isotropic pore-water pressure. In addition, swelling pressure is a direct measure of the internal effective stress of the soil.
- The swelling process is a re-arrangement of the internal effective stress to effectively resist water flow. The mobilised form of effective stress (soil cohesion) is capable of resisting water flow by reducing it to a diffusion process
- The internal stress model rationally integrates the two forms of internal effective stress as a continuous function of water content.
- The effective stress principle can be used to characterise the mechanical behaviour of the investigated expansive soil
- A physical soil model quantified the change in soil structure during the swelling process. It is called the Swelling Boundary Surface (SBS) model.
- The analysis successfully rationalised the consistency limits. The consistency (index) limits reflect the change in internal stress of the investigated soil. They define rational point in terms of effective stress
- The intrinsic soil property is linked to the effective stress at the shrinkage limit. It is called the pretension stress or number
-
- The linear shrinkage limit is a rational water content limit. It is linked to the pre-tension stress of the soil. The pre-tension stress of the soil characterises the pretension number and is equal to the mobilised pre-consolidation pressure. The linear shrinkage limit links the other consist-

ency limits, as redefined. The link is underlined by the changes in the internal effective stress. Accordingly, the consistency limits are rational limits, which can be used to characterise the stress-strain behaviour of the investigated soil.

- The intrinsic soil property was defined and rationalised. It is herein called the ***pretension rate, K*** (kPa per water content change). It relates the change in the pretension stress with soil water content or degree of soil saturation. The pretension rate can be normalised against atmospheric pressure to give a dimensionless number, called the ***pretension number, k***.
- The pretension stress is equal to the pre-consolidation pressure of the investigated soil.
- Swelling pressure is essentially excess (negative) pore-water pressure, which is transient. The term 'swelling pressure' is not reflective of its nature and character. Accordingly, it is herein called ***hydrotransient pressure***.
- Water flow in the investigated expansive soil is predominantly by diffusion. Accordingly, the different wetting paths do not affect the period of the swelling process.
- The swelling response of an expansive soil to water flow is a consequence of the change in the internal effective stress. The pretension stress uniquely characterises the internal effective stress changes. It is coupled to the shrinkage limit. Consequently, the shrinkage limit is a fundamental a parameter. In addition, it rationally links the other consistency limits. Accordingly, the revised consistency limits can be used to correlate the fundamental soil behaviour. In addition, they can be used in developing models for expansive soils.
- The induction concept enables the treatment of soil behaviour in terms of the principle of effective stress. However, because of the composition of the internal stress, it is in two forms. The first form is the internal effective stress form, while the other is the mobilised form.
- The effective stress principle has been successfully applied to the changes in internal effective stress of the investigated unsaturated expansive soil. However, the composition of the internal effective stress is such that the effective stress principle is decomposed to the internal form and the mobilised form.
- The consistency limits were redefined in line with the development of soil plasticity. The redefinitions are as follows. The linear shrinkage-limit (no change). The elastic limit was previously called the plastic limit. The saturation point is the pressure saturation water content. The plastic limit is the atmospheric saturation point. Zero effective stress defines the liquid limit.

However, this is not coincident with disintegration of the soil structure. Accordingly, an additional limit, the disintegration point, was defined. It is the water content, at which the adsorbed-water, part of the hydrated clay minerals, detaches from the solid part.

- The swelling process can be visualised in three different but related perspectives. (1) It is a reversal process, where soil attraction is unloaded and reloaded. (2) It is a conversion process, where the internal effective stress (soil attraction) is unloaded and converted to mobilised effective stress (soil cohesion). (3) Analogous to consolidation settlement, it is a stress unloading process. The stress is unloaded and similarly consolidated in a mobilised form in the adsorbed water. The adoption of the SBS physical model elegantly ties the three together.

- The proposed SBS physical soil model adequately characterises the physical response of the investigated soil during swelling and shrinking. It is an appropriate complement to the measurable quantities of the swelling phenomenon. Accordingly, it forms a sound basis of developing of constitutive models for unsaturated expansive soils.

- The current method of determining the liquid limit leads to the upper limit of the particle disintegration point. However, the revised definitions are consistent with the changes in the internal effective stress. In addition, the liquid limit has been rationalised in terms of its relative position and its determination.

- Soil plasticity is correctly defined during soil wetting. The development of plastic flow is identical to that of a heated metal bar. The identity comes out of the link between heat energy and the internal effective stress of the soil. Heat energy is converted to effective stress during soil drying.

- Soil plasticity develops during the wetting cycle and is analogous to the plastic flow of a heated metal rod. The unloading and mobilisation of the internal effective stress is analogous to the increase in the kinetic energy of the molecules in the metal rod.

- The induction concept adequately explains the conversion of the effective stress from heat energy during soil drying. It stems from the equivalence between heat energy and effective stress, in terms of performing mechanical work (Joule, 1894). Therefore, cyclic swelling is essentially an addition and removal of heat energy from the soil.

- Soil suction does not cover the full range of the swelling phenomenon. The range over which it diminishes is comparatively instantaneous. Accordingly, it is not a suitable stress variable to use for the investigated soil. It would be appropriate in soils, which do not generate swelling pressure. In such cases, the elastic limit and the pressure saturation points are coincident.

- The swelling process is analogous to triaxial and oedometer compression of saturated soils.

10.3 Recommendations for further research

The research findings recorded herein are of a fundamental nature. As such, the implications are far reaching in terms of characterisation and modelling of unsaturated soils in general and expansive soils in particular. The subject of expansive soils can now be similarly explored to the same extend as saturated soils. It is significant that the new concept developed herein offers an opportunity to integrate unsaturated and saturated soil behaviour, in terms of the effective stress principle. Accordingly, the recommendations for further research are in the following areas.

- Validate the INDUCTION CONCEPT using results from soils covering a wide range of plasticity
- Develop charts or monograms for the soil properties for different soils (for design purposes)
- Time rate effects & water flow in swelling soils
- Develop soil models (soil shrinking, cyclic swelling & soil-structure interaction) & validate them with field data
- Extend the induction concept to unsaturated soils in general
- Re-define the consistency limits and classification of expansive soils in terms of the soil parameters and the consistency limits. Development of simplified guidelines for the practising engineer.
- Development of the theory of swelling soil behaviour and water flow.
- Development of swelling soil-structure interaction models
- Full-scale field investigations to validate the models
- Develop practical design guidelines for engineering on expansive soils, e.g. shallow foundations, earth-retaining structures, etc.

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