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The identification and characterisation of collapsible soils: a brief review of current practice

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ABSTRACT

The existing literature contains no shortage of attempts at defining ‘collapsible soils’, all largely focussed on outlining the often unique properties and conditions required for collapse settlement to take place. First recognised and studied by Jennings and Knight (1957), collapse settlement can, in its simplest form, be considered as the radical rearrangement of soil particles upon wetting to result in a significant reduction in the total volume of the soil mass. It is this behaviour, and subsequent impact on structures founded within such materials, that make the timely identification and accurate characterisation of collapsible soils significant to the geotechnical professional.

This paper provides a brief overview of the collapse settlement phenomenon as documented in the existing literature. It includes a discussion of published findings regarding the potential for collapse settlement to occur in New Zealand soils, and some of the more widely recognised methods used to:

1. Identify the potential for collapse settlement to occur on site (i.e. recognising materials which may have a ‘collapsible’ fabric and which, under particular conditions, may undergo collapse settlement),
2. Assess the potential severity of collapse settlement (qualitative evaluation), and
3. Assess the potential magnitude of collapse settlement (quantitative evaluation).

The paper concludes with a brief discussion of collapse behaviour from an effective stress perspective and why this should be considered when assessing collapse settlement.

1 INTRODUCTION

Partially saturated soils could be dense with a high bearing capacity that will subsequently only suffer small amounts of compression under normal foundation loads. However, when wetted under load many such soils undergo a marked and sudden increase in settlement due to collapse at a microstructural level, the phenomenon which is known as ‘collapse settlement’.

Collapse settlement behaviour is associated with a wide range of naturally occurring soil deposits internationally, and is often discussed in the same breath as other ‘problematic soil behaviours’ such as shrink/swell and soft, compressible soils. Unsaturated soils such as the loess formations of China, Russia and Eastern Europe cover enormous areas of those countries and constitute perhaps the classical reference for collapsible soils, as do the Kalahari Sands of Southern Africa (Rust et al. 2010). Other examples include residual granitic soils from the lowveld of South Africa, sensitive (or ‘quick’) clays of Canada and Northern Europe (Derbyshire et al. 1994), as well as compacted soils (Booth 1977).

The geotechnical professional in New Zealand may however well be excused if ‘collapsible soils’ is not the first example of ‘problem soils’ that springs to mind, simply due to the fact that soil deposits which are prone to collapse settlement are not dealt with on a daily basis. However, with the growing pressures on land use, particularly in urban areas, brownfield sites are being considered more and more for redevelopment opportunities. In the case of back-filled and disused mining operations, rehabilitation often leads to substantial fill thicknesses, often combined with rising groundwater levels upon cessation of extraction pumping operations. The potential for collapse settlement in such scenarios is well-recognised and the understanding of which is of critical importance to ensure successful future redevelopment.

In the early years, much emphasis was placed on identifying soils with a collapse potential and also developing methods to estimate foundation settlement, should collapse occur. Very little attempt was made to develop a fundamental framework within which the mechanisms that contribute to the collapse of soils can be understood. With advances in our understanding and testing of unsaturated soil mechanic principles, it has however become apparent that the evaluation of collapse settlement in terms of total stress represents an over-simplified approach in attempting to explain such behaviour, and that collapse settlement is best considered as a function of the change in the effective stress state of a partially saturated material.

2 COLLAPSIBLE SOILS

2.1 Defining soil collapse

Collapse settlement was first recognised and studied by Jennings and Knight (1957), and the existing literature contains no shortage of attempts to define ‘collapsible soils’:

“...a soil that undergoes an appreciable amount of volume change upon wetting, load application or a combination of both” (Sultan, 1969),

“...any unsaturated soil that goes through a radical rearrangement of particles and a great loss of volume upon wetting with or without additional loading” (Dudley, 1970),

“...additional settlement...due to the wetting of a partially saturated soil, normally without any increase in applied pressure” (Jennings and Knight, 1975),

“...settlement in partially saturated soil due to an increase in the degree of consolidation” (Booth, 1977), and

“...a state of underconsolidation related to apparent cohesive strength of unsaturated soils” (Handy, 1973).

Schwartz (1985) defines a collapsible soil as "...a soil which can withstand relatively large imposed stresses with small settlements at low *in-situ* moisture content but will exhibit a decrease in volume and associated settlement with no increase in the applied stress if wetting up occurs".

Perhaps the most widely accepted interpretation of collapse settlement in partially saturated soils is based on the work of Rogers (1995), who defines soils as being collapsible where "...the constituent parts are at a meta-stable state and have an open packing, which can collapse to form a closer packed, more stable structure of significantly reduced volume".

The interparticle forces leading to the development of the initial meta-stable state may comprise any of a combination of matric suction forces, cementation, flocculated clay particles and/or silt and clay bridges between particles (Figure 1).

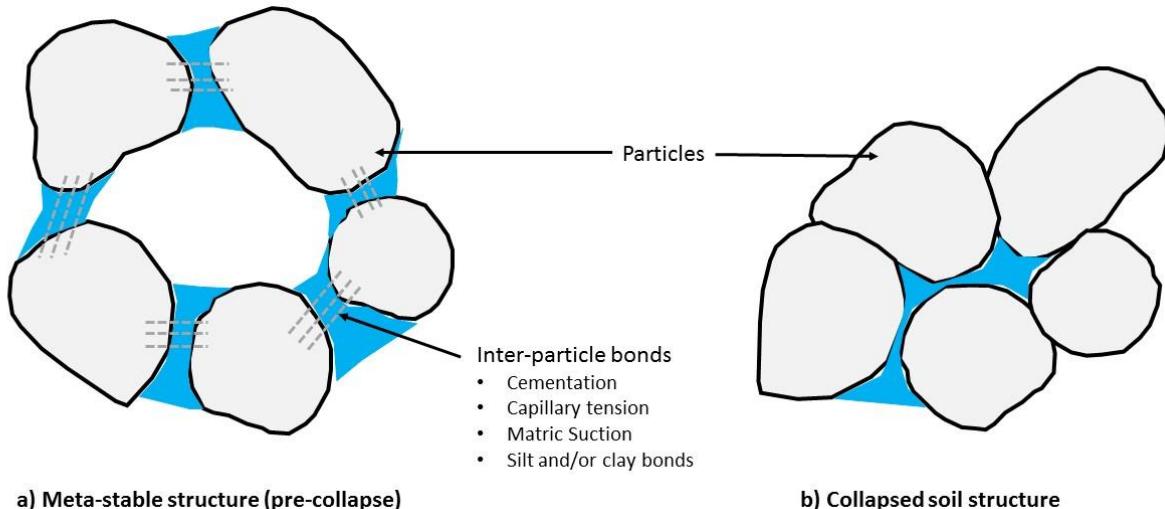


Figure 1: Schematic illustration of the mechanism of collapse settlement

2.2 Distribution

2.2.1 Naturally occurring collapsible soils in New Zealand

In the NATO ASI series Derbyshire et al. (1994) identifies the loessial and volcanic ash deposits of New Zealand as being prone to collapse settlement.

Loess of Pleistocene age is predominantly encountered over the southern portion of the North Island and east of the Alps in the South Island (Raeside 1964). The thickness of loess deposits varies geographically, with deposits > 1 m covering approximately 10% of the South Island (Figure 2). These deposits have been predominantly formed from aeolian transportation and deposition of glacially-derived rock flour, and although primarily silty, contain up to 45% clay giving rise to low plasticity clay behaviour (Yates et al. 2018).

The available literature contains a relative lack of research or reports on collapse settlement associated with New Zealand loess deposits. In their review of the geotechnical characteristics of the loess and loess-derived soils of the Canterbury region, Yates et al. (2018) however conclude that these deposits are relatively dense (with dry densities of 1.6 t/m³ and 1.8 t/m³), and hence do not display collapse settlement behaviour common to some of the other loess deposits mentioned above. This review does not include test results focussed specifically on assessing the potential collapse settlement of these deposits, and the conclusion regarding the relative absence of collapse settlement appears to be made on the basis of the reported dry densities (and perhaps experience in working with these materials).

Although a review of the available literature provides every indication for these deposits to generally be of low collapse potential, care should be taken to correlate collapse potential to the density of the materials alone, which is likely to be a too simplistic assumption in explaining the mechanism through which collapse occurs (discussed in Section 3.4 of this paper). Post-depositional reworking will contribute to destroy any collapsible soil fabric associated with the original aeolian deposit and, considering these deposits are comparatively thin, are likely contribute to the risk of collapse potential (or lack thereof) in New Zealand loess deposits.

The occurrence of collapse settlement in natural soil deposits of other origins (i.e. volcanic ash air fall deposits) could not be confirmed from the available published literature. This is either a general indication that these soils are either not generally susceptible to collapse settlement, or that this is not a well-recognised phenomenon.

It should be emphasised that the occurrence of surface subsidence in the form of fissures, chasms or ‘tomos’, often associated with the pumiceous soil deposits of the central North Island of New Zealand, is not related to the mechanism of collapse settlement as the topic of this paper. Such features tend to be associated with large-scale internal erosion through the washing out of fines along preferential flow-paths. Subsidence occurs when these features daylight at surface once the overlying soil cover ‘collapses’ into the underlying void, and as opposed to the particle re-arrangement at a micro-structural level associated with collapse settlement.

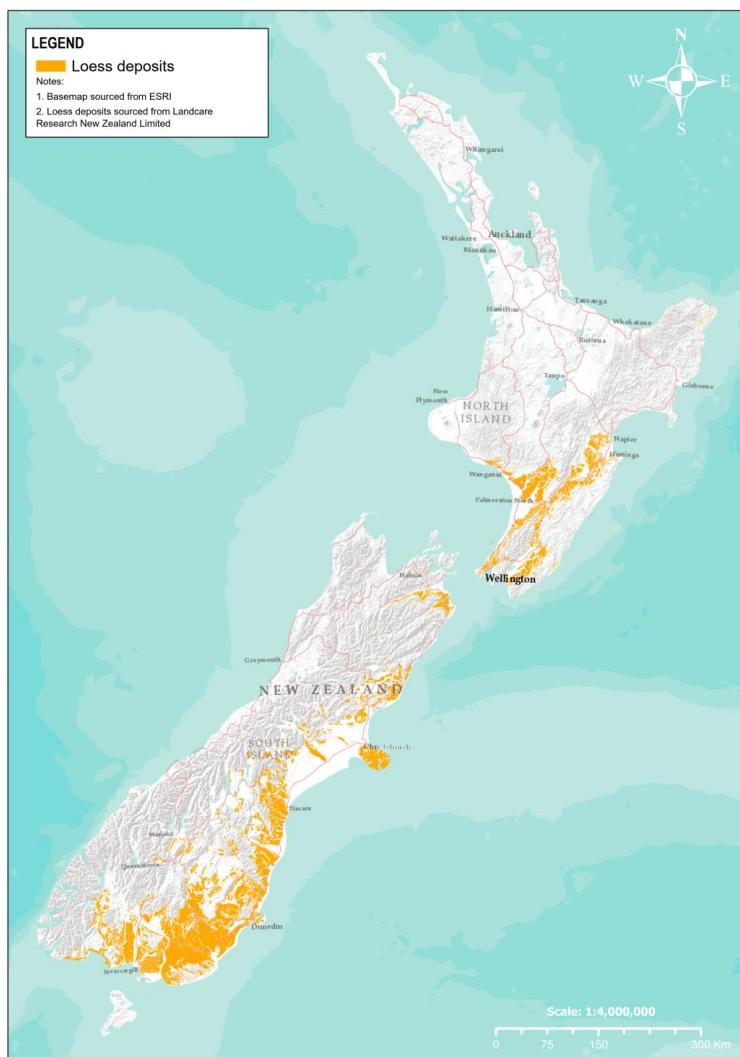


Figure 2: Distribution of loess deposits in New Zealand

2.2.2 Compacted soils

Man-made earth structures such as embankments, fills and earth dams often exhibit collapse when compacted dry of optimum (Holtz 1948). It is commonly assumed that only sandy or silty soils exhibit collapse; however it has been reported that compacted soils in general may exhibit collapse (Barden et al. 1973; Cox 1978). As settlement characteristics of engineered fills are generally not a significant concern where the fill is of limited thickness, emphasis in this paper will be placed on the potential for collapse settlement of thicker fill placement, often associated with backfill to quarries or deep shear keys.

All surface mining methods involve the extraction or stripping of near-surface materials (both waste and ore), inherently leading to the disturbance of large surface areas of land over the life cycle of extractive operations. In New Zealand, such operations are most widely represented by the aggregate quarrying industry (Figure 3).

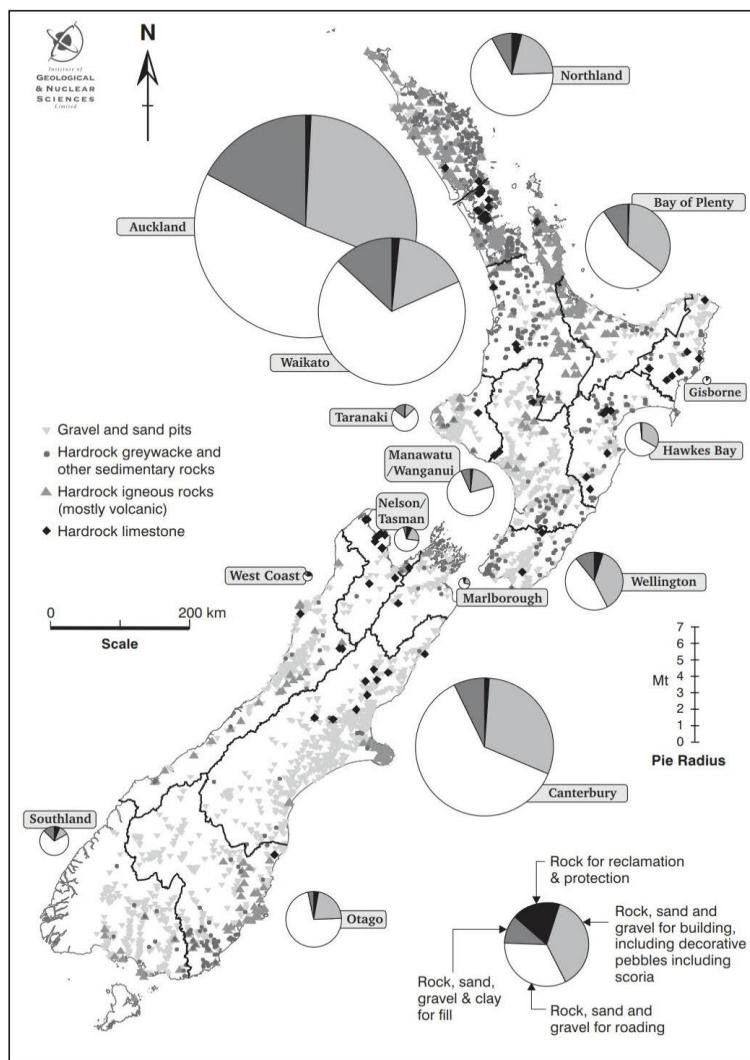


Figure 3: Location of aggregate quarries and pits with recorded production since 1967 (after the GERM database) and pie diagrams of aggregate production in 2000, by region (Christie et al. 2001)

With growing pressures on land use, such sites are being considered more and more for commercial infrastructure and housing development after mine closure (Hills 1994), and where previously the site might have been filled with uncontrolled material or left undeveloped. The redevelopment of brownfield sites is being increasingly promoted in New Zealand, particularly in the larger and fast-growing urban areas. The

Auckland Unitary Plan (UP), which became partly operative in November 2016, rezoned large swathes of Auckland for more intensive development and for redevelopment of brownfield sites.

To enable such future development, any mined-out void needs to be reinstated to some workable final surface level through backfilling. The long-term settlement characteristics of such fills, which are typically of substantial thickness, therefore becomes an important consideration.

3 ASSESSMENT OF COLLAPSE SETTLEMENT

The identification and characterisation of collapsible soils is best considered by following a staged investigation approach, with the complexity of evaluation increasing for each stage.

It should be emphasised that although this proposed guideline holds true for all types of collapsible soils, particular relationships and methods of evaluation in the published literature may be more (or less) appropriate for different types of collapsible soils, as briefly discussed in the following sections.

3.1 Identifying potential for collapse settlement

In its simplest form, the potential for collapse phenomenon may be identified through the existence of a collapsible soil fabric, recognisable through the presence of certain conditions in the subsoil structures (Rogers 1995):

- an open soil structure,
- a high void ratio,
- a low dry density,
- a high porosity,
- a geologically young or recently altered deposit,
- of high sensitivity; and
- low interparticle bond strength.

In turn, Barden et.al (1973) recognised that for appreciable collapse to occur, the material should have a collapsible fabric, but also a high enough value of applied stress to exist in a metastable condition and a high enough value of soil suction to stabilise intergranular contacts. Reduction of the latter (through wetting) will lead to a change in the effective stress of the material and trigger collapse (Section 3.4).

Collapse settlement in open-cast backfill can be seen as additional settlement under constant total stress resulting from an increase in the water content of the backfill. Unlike other settlement mechanisms (e.g. consolidation or creep), collapse potential in fill does not automatically reduce with time. This is highly dependent on the fill material properties, particularly the density at which the material has been placed and the moisture content at time of placement and when collapse first occurs. Charles and Skinner (2001) state that for fill placed dry of Optimum Moisture Content (OMC), collapse potential is predominantly a function of the ratio of dry density to maximum dry density (i.e. degree of relative compaction), whereas in materials placed wet of OMC the percentage air voids predominates collapse potential. Perhaps the most critical point is the fact that collapse settlement is possible in fill material compacted either wet or dry of OMC.

Field observation and research has shown that inundation is a major trigger of fill collapse settlement in opencast backfills. An increase in moisture content of the fill may occur as either the result of downward water infiltration from the surface, through the old mine highwall, or by cessation of mine void dewatering operations, leading to the rise and re-establishment of the at rest groundwater levels within the backfill (Day 2013; Hills 1994).

It is the authors' experience that, unless a clear post-closure development plan has been agreed on, backfill material is often indiscriminately sourced and often placed with insufficient controls with regards to sequencing or proper scheduling of placement during the backfill process. Such an approach could lead to significant heterogeneity of the backfilled material due to variation in material properties (e.g. different mineralogical or chemical compositions, particle sizes, grading, etc.), as well as the degree of engineering or modification achieved during placement (e.g. compaction, saturation, etc.).

Such non-uniformity in material properties complicates the recognition of collapse potential for any particular site. It is therefore proposed that, unless it can be conclusively established that the fill does not have significant collapse potential, groundwater levels have returned to natural (pre-mining) levels or considerable collapse has already occurred; a risk of collapse settlement in the fill during or subsequent to building on the site should be assumed.

3.2 Potential severity of collapse settlement

There is no shortage of attempts in the existing literature to correlate and predict collapse as a function of material characteristics or index properties such as density, clay content, moisture content etc. It is the author's experience however, that such empirical relationships are usually very site-specific and only tend to be valid for the environment in which these were developed.

Collapse is typically evaluated in terms of total stress, as shown in Figure 4, where an unsaturated soil sample is placed in an oedometer at natural moisture content (A), loaded in stages to the desired vertical total stress (B), before the sample is inundated with water and the vertical strain is recorded (B to C). Such a collapse test with the sample inundated at 200 kPa is widely referred to as a Collapse Potential Test, and the vertical strain (B to C) during inundation is taken as the Collapse Potential. The sample is then loaded to point D and unloaded to point E under saturated or near-saturated conditions.

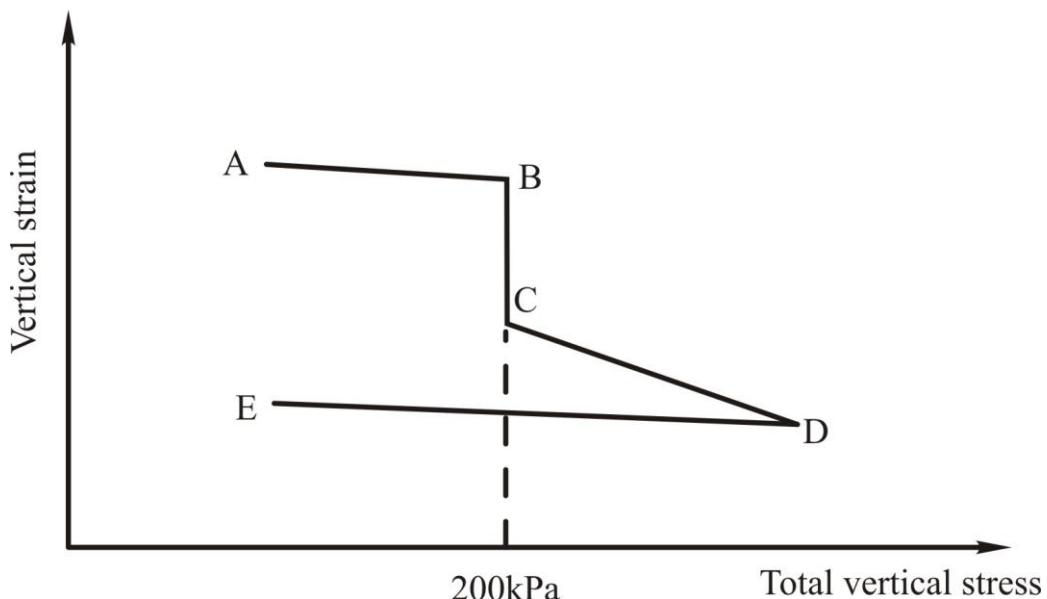


Figure 4: Collapse Potential Test

This method is superficially attractive due to its simplicity, and is widely used as an indicator test to allow judgement on the severity of possible foundation settlement due to soil collapse, as shown in Table 1 (Brink and Heymann 2014).

Table 1: Collapse Potential Test categories (after Jennings and Knight 1975)

Collapse Potential (%)	Severity of Problem
0-1	No problem
1-5	Moderate trouble
5-10	Trouble
10-20	Severe trouble
>20	Very severe trouble

It should be emphasised that Jennings and Knight (1975) intended the Collapse Potential Test to be used as an indicator of the potential severity of collapse at or near saturation of the material, and not as a method to predict the amount of collapse settlement that could occur as part of the engineering design process.

An obvious reason for this is the fact that collapse is estimated at a single applied pressure (i.e. 200 kPa), complicating the estimation of potential volume change due to collapse over a range of pressures typically expected throughout a soil profile of any notable depth. Further, the potential influence of bedding and confinement errors on any results obtained using this test method could be significant. The inaccuracy due to bedding errors becomes particularly more pronounced as the height over which the sample collapse is measured becomes smaller, such as in the case of a one-dimensional oedometer (Rust et al. 2010). As an example, the protrusion of resistant asperities of only 1 mm at either end of the prepared sample will equate to an error of approximately 10% of the oedometer sample height. Such errors could have a significant potential effect on the reported test results, potentially resulting in a gross over-prediction of the collapse potential of a material.

High-quality sampling is also required for evaluating the collapse potential of soils in the laboratory. Hight et al. (1992) showed that block sampling produces samples of the highest quality, compared with other sampling techniques (e.g. push tube sampling). However, Heymann and Clayton (1999) highlighted disturbances that may occur as a result of moisture change during storage. Even small changes in moisture content can alter the matric suction, and hence the effective stress, relatively much more so than the loading due to an engineering structure in the field (Rust et al. 2010).

Considering the potential differences in properties between fill and natural soil deposits, caution should be applied in commenting on the potential severity of collapse across the backfilled void through conventional small-scale laboratory tests alone. Such test regimes typically rely on samples collected at discrete point locations from a material of highly variable properties.

Further consideration should be given to the thickness of the collapsible horizon in the case of fill materials (typically equal to the thickness of material likely to be saturated). The severity of problem categories proposed in Table 1 could therefore be misleading when applied to collapse settlement in thick fill deposits, and where collapse potential is typically expressed as a percentage of the thickness of fill material subjected to saturation. Hills (1994) gave the following typical values for collapse, expressed as a percentage of the thickness of the saturated layer.

Table 2: Mean values and standard deviation for collapse settlement (Hills 1994)

Placement Method	Collapse Settlement (%)	
	Mean	Standard Deviation
Controlled	Performance	0.25
	Method	0.40
	Thick Layer	0.90
Uncontrolled	1.20	0.41

A more suitable approach to evaluate collapse of variable fill deposits is therefore perhaps through a combined large-scale field test and targeted laboratory test regime, of which the former are likely to provide a better indication of the behaviour of the in-situ material on site. Examples include long-term flooding experiments, which have been successfully carried out at a backfilled site in South Africa (Day 2013), or modified plate load tests at various backfill levels.

3.3 Potential magnitude of collapse settlement

The double oedometer test described by Jennings and Knight (1957) is perhaps the best known approach to quantify the potential of collapse settlement. The test procedure comprises testing of two undisturbed samples in the oedometer; one sample at its sampled moisture content and the other saturated prior to testing. Allowance is made for different initial void ratios of the two undisturbed samples, as well as the consolidation state of the material. Perhaps the biggest advantage of this test procedure is the ability to quantify collapse for a range of applied stresses. This method is however also plagued by the same uncertainties surrounding bedding errors as discussed for the Collapse Potential Test in Section 3.1, and considerable emphasis needs to be placed on ensuring the test results are correctly interpreted. The reader is referred to the detailed description of this test procedure in Jennings and Knight (1957), the discussion of which is outside the scope of this paper.

Heymann (2000) and Rust et al (2005) described a test for measuring the collapse settlement of soil in a triaxial apparatus. Conducting a collapse test in a triaxial apparatus means a larger specimen is used compared to in the oedometer, and the use of local strain instrumentation allows for any errors due to bedding to be entirely excluded. In their study, Rust et al. (2005) compared the collapse settlement test results from standard oedometer test procedures to the results from triaxial testing for a site in Mozambique, with the results from the latter concluded to be considerably more reliable and consistent due to the absence of bedding and confinement errors.

3.4 Collapse in terms of effective stress

Schwartz (1985) pointed out that the degree of saturation was a vital parameter in assessing collapse potential and suggested that it should be used as a determining factor for collapse estimates (i.e. a critical degree of saturation exists for collapsible soils above which collapse would not occur). However, it is well known that soil behaviour depends on the effective stress, and a fundamental interpretation of the collapse phenomenon should be made in terms of effective stresses. This however, requires accurate real-time measurement of soil suction to correctly model a collapsible soil in terms of effective stress, which is not realistic for the practising engineer or a commercial soils laboratory to adopt (Rust et al 2005).

A number of theoretical frameworks have been developed to express the effective stress for unsaturated soil. Bishop (1959) formulated the following equation to quantify effective normal stress (σ') for unsaturated soil in terms of total stress (σ), pore air pressure (u_a) and pore water pressure (u_w):

$$\sigma' = (\sigma - u_a) + \chi(u_a - u_w) \quad (1)$$

$(\sigma - u_a)$ is termed the net normal stress and $(u_a - u_w)$ is the matric suction. χ is a parameter that quantifies the contribution of the matrix suction to the effective normal stress. χ varies between 0 for a completely dry soil and 1 for a completely saturated soil. For these two conditions, Equation 1 reduces to the classic Terzaghi equation ($\sigma' = \sigma - u$) for effective normal stress of a soil where the pores are completely filled either with air or water.

Bishop (1959) suggested that χ is a function of the degree of saturation (S_r), soil structure and whether the soil is in a wetting or drying phase. Due to a lack of experimental data, early workers often suggested that $\chi = S_r$. However, Alonso et al. (2010) pointed out that this may a reasonable assumption for coarse soils, but for clayey material at low degrees of saturation, it leads to the second term in Equation 1 being unrealistically high. They reported data to show that $\chi = S_r^N$ where N typically varies between 1 and 2 for silts and between 4 and 6 for clays.

Rust et al. (2005) proposed a conceptual yield model to explain soil collapse using effective stress paths as shown in Figure 5 and briefly explained below.

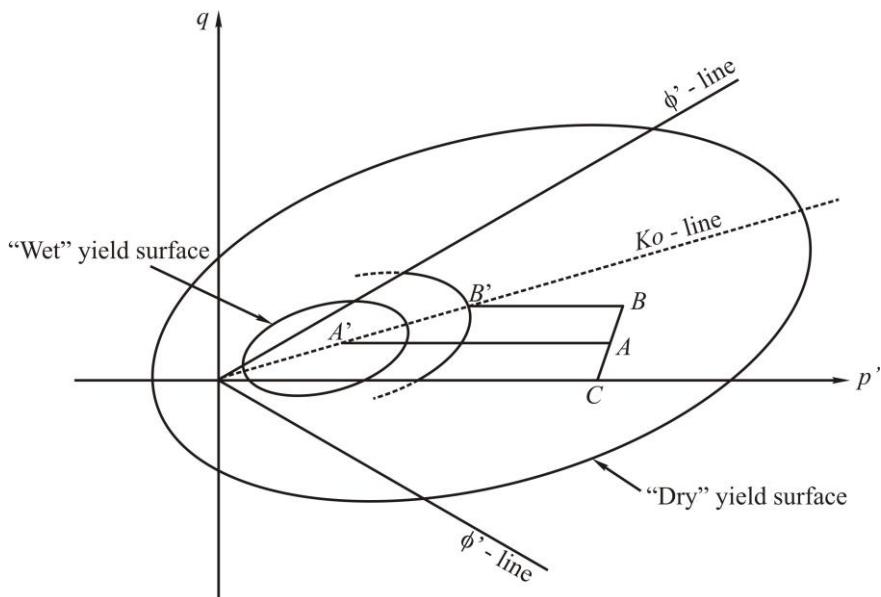


Figure 5: Conceptual yield model for collapsible soil (from Rust et al. 2005)

Consider an *in-situ* collapsible soil with stress condition A in Figure 4 above. The stress condition is inside the "dry" yield surface. If wetted, the path will move to point A' and the *in-situ* stress state will be inside the now smaller "wet" yield surface for the *in-situ* soil, which is now weaker than before it was wetted. But since the stress state stayed inside the shrinking yield surface during wetting, no collapse of the *in-situ* soil profile occurred. If the *in-situ* soil at point A is loaded, the stress state may move to point B. Alternatively, if the soil is first sampled, it will move to point C where it does not experience any shear stresses ($q = 0$), before it is loaded in the oedometer to point B. At point B the soil is still at its *in-situ* moisture content and hence still inside the "dry" yield surface. If the soil is now wetted, the yield surface will shrink as the soil weakens and the stress state will move toward B' where the yield surface and stress state coincides with the one dimensional strain (K_o - line) (Brink and Heymann 2014).

Even though this proposed framework has not been verified experimentally, primarily due to experimental difficulties, it is useful since it attempts to describe the collapse phenomenon in terms of effective stresses. It also shows that from a fundamental effective stress perspective, the behaviour is much more complex than the behaviour shown in terms of total stresses in Figure 3 (Brink and Heymann 2014).

4 CONCLUSIONS

A review of the existing literature confirms the apparent scarcity of natural soil deposits in New Zealand with collapse settlement potential. Opportunity for further research exists in this regard, focussed specifically on characterising New Zealand soils in terms of their potential for collapse settlement. Collapse settlement is however also associated with compacted soils (i.e. fills), and which are more widely distributed. This poses a particular risk where infrastructure developments are underlain by fill of substantial thicknesses combined with rising groundwater levels, as is often the case for backfilled mined-out voids.

To evaluate collapse settlement, an approach focussed on increasingly detailed levels and complexity of investigation is proposed. There is no One-Stop-Shop approach to this, and a suitable investigation program should be developed on a case-by-case basis for the individual site where the potential for collapse settlement may exist. This should take into account the limitations to existing methods to evaluate collapse settlement.

The interpretation of soil collapse in terms of total stress leads to an oversimplification of the behaviour of the soil. This masks the contribution of the change in effective stress. Effective stress due to matric suction may be higher than the stress applied to the soil as a result of operations related to construction of infrastructure. It follows that changes in moisture content will have a greater effect on the effective stress, and hence the strength and stiffness of the material, than stress changes due to engineering activity.

Geotechnical professionals therefore need to consider not only the effect of loading the soil due to construction, but also the consequence of moisture change on the effective stress experienced by the soil.

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