# On the Williams-van der Merwe chart for classification of expansive clay soils

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ABSTRACT: The Williams – Van der Merwe chart for the classification of the potential expansivity of soils and the prediction of heave for single-storey dwellings on the South African Highveld was first published in 1964, revised in 1976 and accepted in 1982 into TRH9 for roads. Although criticized at the time and since for the use of the so-called clay fraction passing 2 µm (the determination of which was regarded as unreliable), in addition to the plasticity index, on account of its simplicity and low cost it rapidly became the most commonly used method in southern Africa and has also been widely used elsewhere, all with apparent success. The origin and test methods used to derive the classification parameters and their applicability to local conditions and test methods are reviewed, illustrated by a case history of a road damaged by an expansive clay roadbed which classified as of low expansivity according to the chart, but of high expansivity according to the plasticity index alone. It is concluded that previous opinions that the use of the plasticity index of the whole sample alone is probably more reliable than the use of the use of the whole chart are supported, that the test methods used must be compatible with those used to derive the parameters used, and that certain adjustments to the chart could be made in the light of later experience.

## 1 INTRODUCTION

The so-called Van der Merwe (1964, 1976) chart and empirical, method for the classification of the potential expansive-ness of soils and the predictions of heave for single-storey dwellings on the South African highveld was supported by Jennings (1965), Knight (1973) and Donaldson (1973), became the method most commonly used in South Africa (Williams and Donaldson 1980, Brackley 1985), gives reasonable estimates of heave (Pidgeon, 1987), and – in spite of warnings by Van der Merwe (1964), Jennings (1965), Donaldson (1976), Williams and Donaldson 1980, Williams et al (1985) and Blight (1986) not to use it out of context and Nelson and Mills (1992) to only use it as an indicator and not for quantitative predictions – is used throughout the world, apparently with great success (Donaldson 1976).

It is the purpose of this paper only to review the origin, test methods and the soils used to derive the classification parameters and to illustrate a deficiency by means of a case history of a road severely damaged by the expansive clay roadbed which classified as of only low (usually taken as approximately zero) expansivity, and to suggest some minor improvements. Other methods using index and other tests have been reviewed for example by Williams et al (1985) and Nelson & Miller (1992).

## 2 THE CHART

Van der Merwe's (1964) original chart is that of Williams (1958) which in turn is an adaptation of Skempton's (1957) plasticity chart.

As the original chart subsequently only underwent minor modifications in 1976 the one accepted into TRH9:1982 for roads (Fig. 1) will be used as a basis for discussion.

Van der Merwe's contribution was to assume unit heaves at the ground surface of 1 inch (25 mm) per foot (300 mm), i.e. 8 % of very highly expansive soil, ½ inch (13 mm) for highly expansive soil, i.e. 4 %, ¼ inch (7 mm), i.e. 2 %, for medium, and zero for low or non-expansive soil, deriving a depth factor equation, and successfully applying the method to the estimation of the maximum total heave as measured on external pegs to five cases of known heave in the Free State and one on black clay at Onderstepoort in the North West Province.

Skempton's (1957) chart showed a plot of plasticity index (PI) against the percentage clay fraction, i.e. (passing 2 µm (P002) of a number of samples of clay soils from four sites in England, showing that each site possessed a particular activity ratio, i.e. PI/P002, which provided a convenient single-value parameter for any particular clay.

The activity concept was then applied to 27 other clay soils (mostly illitic marine, lacustrine are organic) from Britain and elsewhere with activities of 0.33–1,75 and to samples of quartz (0.0), calcite (0.18), muscovite (0.23), kaolinite (0.33 and 0.46), Ca-smectite (1.5), Na-smectite (7.7) and Mexico City (4.3) and Wyoming bentonite (6.3).

He regarded inactive clays as having an activity of <0.75, normal clays as 0.75-1.25 and active clays as >1.25.

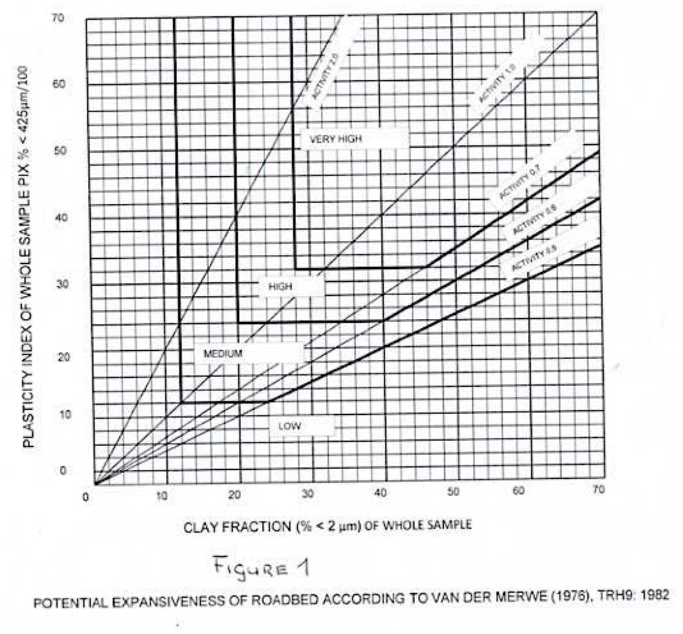


Figure 1. Potential expansiveness according to Van der Merwe (1976), TRH9:1982

Attempts by others to correlate the activity with the swelling potential of clays were found to be not very reliable (Rodriques et al. 1988).

Williams' (1958) adaptations were to provide a chart similar to that shown in Figure 1 with activity and plots of 0.5, 1.0 and 2.0 and PI and P002 boundaries for zones of low, medium, high and very high potential expansiveness mostly those of Holtz, & Gibbs (1956) but calling them the whole sample PI obtained by multiplying the PI by the fraction passing 420 µm.

Whilst the validity of this latter correction may be doubtful in some cases, most of the clay soils tested for the work reported by Kantey & Brink (1952) and Van der Merwe (1964) had at least about 90 % finer than 420 µm (A.B.A. Brink 2002, pers. comm.).

This was offered as a means of obtaining "a good idea of the possible behaviour of soil" from the use of Atterberg Limits for their recognition and already found useful by De Bruijn et al (1956).

The Kantey-Brink criteria for the identification of potentially expansive clay sufficient to damage single storey brick buildings, all three of which must be satisfied, are liquid limit (LL) > 30, plasticity index (PI) > 12, and linear shrinkage (LS) >8.

Most of these soils also lay above the A - line and to the right of the B - line (LL of 30) on the Casagrande (1947) plasticity chart. In general, Skempton's (1953) activity factor also exceeded 1,0 (De Bruijn et al. 1956).

Inspection of their plot of 250 results on samples – not all of which were considered expansive and included topsoils – but came from expansive profiles shows that their criteria of a LL > 30 and a PI > 12 only would include 222 (i.e. 89 %) of the results and exclude 28 (11 %) with LLs of 19 – 46 and PIs of 4 – 21.

Application of a PI of > 12 alone would include 231 results (92 %) and exclude 19 (8 %) with LLs of 19 – 46 and PIs of 4 – 11 suggesting that the use of the PI alone may be sufficient, supporting Williams' (1958) use of this alone as the boundary between 'low' and 'medium' on the chart of potential expansivity.

Brackley (1985) was also of the opinion of that the PI of the whole sample alone was a sufficiently good indicator of the intrinsic expansiveness of a soil – at least in the case of highly expansive soils

A large number of samples from eight areas of South Africa were tested with LLs of 25–97, PIs of 10–71, LSs of 8–21, and shrinkage limits (SLs) of 5–28.

The criteria established by Holtz & Gibbs (1956) are shown in Table 1.

Table 1. Data for making estimates of probable volume change for expansive soils from Delta–Mendota and Friant–Kern canals, California (Holtz & Gibbs 1956)

| Colloid content | PI | SL | Probable | Expansion* |
|---|---|---|---|---|
| % < 1µm | % | % | % | Degree |
| >27 | >32 | <10 | >30 | Very High |
| 18-37 | 23-45 | 6-12 | 20-30 | High |
| 12-27 | 12-34 | 8-18 | 10-20 | Medium |
| <17 | <20 | >13 | <10 | Low |

*From dry to saturated and based on a vertical loading of 1.0 psi (7 kPa) in a one–dimensional consolidometer.

As stated in his discussion of the paper by Jennings and Knight 91957) on the double odometer test (Williams (1958 & 2010 pers.comm.) the PI boundaries of 12, 23 and 32 are those of Holtz & Gibbs (1956) with the PI of 12 coincident with that of Kantey & Brink (1952), all cut off at an activity of 0.5, and the P002 boundaries those of the P001 limits of Holtz & Gibbs (1956): all of which are also those of Van der Merwe (1964, 1976).

The only differences between these and those on the TRH9:1982 chart as in Figure 1 are the PI of 24 for the boundary between medium and high and a

P002 of 28 between high and very high. These are probably drawing office errors and probably not significant bearing in mind the poor reproducibility of these tests. However, the TRH9 :1982 is presumably the current "legal" one.

The Holtz & Gibbs (1956) criteria were later revised by Holtz (1959), discussed by Holtz & Bara (1965), accepted by the Bureau of Reclamation (BUREC) and have remained unchanged, at least until 2004 (1974, 1998, 2004).

Table 2. Relation of soil index properties to expansive potential of high plasticity clay soils, and for estimation of probable volume change (Holtz 1959, BUREC 2004)*

| Colloid content %<1 μm | PI % | SL % | Probable expansion** % | Degree |
|---|---|---|---|---|
| >28 | >35 | <11 | >30 | Very high |
| 20-31 | 25-41 | 7-12 | 20-30 | High |
| 13-23 | 15-28 | 10-16 | 10-20 | Medium |
| <15 | <28 | >15 | <10 | Low |

* All these index tests should be considered together in estimating expansive properties.
** From air–dry to saturated and based on a vertical loading of 1.0 psi (7 kPa).

Although based initially on only samples from two projects, experience showed that the criteria in Table 1 were generally applicable to all soils encountered in the West (Holtz & Gibbs 1956). The revised criteria in Table 2 are based on actual expansion tests for 45 undisturbed and remoulded samples (Holtz 1959) and the Bureau's guidelines have received much recognition (BUREC 2004)

Other tests that were investigated (Holtz & Gibbs (1956) included the percentage smaller than 5 μm, LL, free swell, and the montmorillonite (smectite) content. However, the three tests selected were regarded as simpler and more practical, although the free swell test on the dry soil fines was regarded as often sufficient for preliminary design and Holtz (1959) that very simple staining tests also provided some identification of clay minerals. However, in South Africa the NBRI had investigated the free swell and shrinkage limit tests and discarded them in favour of the LL, PI and LS but were looking into the undisturbed SL test (Jennings 1955).

## 3 DISCUSSION

In the discussion of Holtz & Gibbs (1956) Altmeyer (1956) suggested the use of the linear shrinkage from the field moisture equivalent, for which the following boundaries had been established from a large number of soils in the greater Los Angeles area:

| LS (%) | Volume change |
|---|---|
| >8 | Critical |
| 5-8 | Marginal |
| <5 | Noncritical |

The standard error of estimate (SEE) of swell prediction based on index tests and 41 undisturbed oedometer results from the roadbed at four sites on the N1 between Pretoria and Naboomspruit in South Africa were 2.02 using the whole sample LL, 2.67 for the PI and 2.09 for the BLS (Weston 1980), suggesting the use of the BLS for heave prediction rather than the PI.

Although not part of Williams–Van der Merwe chart or the Wilson PI criteria the linear shrinkage was one of the Kantey-Brink criteria. It is more useful and the bar method (BLS) has better repeatability and reproducibility than the PI and the problem of cracking and bowing (illustrated e.g. by Stott & Theron 2015) can be reduced by using a slotted trough (Paige-Green & Ventura 1999), or by an initial period of air-drying (NIRR 1968). However, the latter should not then be used as it reduces the shrinkage measured significantly.

In the discussion of Van der Merwe's (1964) paper Wilson (1964) noted that all the soils used by Holtz & Gibbs (1956) and most of those used to verify the 1964 chart were of "normal" activity (i.e. between 0.5 and about 1.5) and that according to the work of Seed et al on compacted clays the swelling potential depends more on the PI than any other property and could be estimated to within ± 33 % from it alone.

He then suggested the following classification which may be safer and would not affect the classification of any of the six numbered soils on the chart (nobody seems to have checked the others):

Table 3.

| PI of whole sample | Potential expansiveness |
|---|---|
| <12 | Low |
| 12–23 | Medium |
| 23–32 | High |
| >32 | Very High |

However, this was regarded by Van der Merwe as a backward step because soils with a high PI but abnormal activity were rare and that when a soil has an activity of just less than 0.5 and a clay content of more than 24 % it could still be expansive as the activity of 0.5 was arbitrarily chosen. He then went on to suggest that activity boundaries of 0.4, 0.5 and 0.6 should be more satisfactory.

Wilson (1973, 1976) nevertheless reprated these and gave further arguments for the use of the PI of the whole sample alone, with which Donaldson (1973) agreed, and mentioned a case where a soil with 90 % P002 but classified as of low expansiveness which cracked a building yet plotted as non-espansive, and that the boundary lines should be extended horizontally to the right

Donaldson (1973) also emphasized that the method was an empirical one for South African Highveld conditions and that it could be misleading when used elsewhere, citing the case of Mariental in a semidesert area where the predicted heave was 75 mm but there was not general heave but severe differential heave due to extraneous sources of water such as broken drains or leaking taps which cracked a building yet plotted as non-expansive and that the boundary lines should be extended horizontally to the right. In 1955 Jennings had already pointed out that all soils except for a few coarse granular soils swell and shrink with the gain or loss of moisture and (Brink 1955) that very slight heaving was known of buildings on desiccated illitic and kaolinitic clays and Donaldson (1976) again suggested that the PI lines be extended horizontally to the right to take such cases into account. In support of this view it is worth noting that the PI of pure kaolinites is usually about 25, but between about 1 and 40, the activity varies between about 0.01 and 0.41 and the bar LS between about 3 and 10 (Grim 1962).

In 1975 Van der Merwe (1976) again emphasized the empirical basis of the method pointing out the importance of the natural moisture content, climate and drainage, possibly even leading to damage due to differential settlements and calling for publication of information comparing calculated and actual heave.

However, little such information appears to have been published, Pidgeon (1987) stating that it gives reasonable estimates of heave, but Willims and Donaldson (1980) that it was not always satisfactory, particularly on residual soils and where the initial moisture contents were higher than in the type areas, and Brackley (1985) that it overpredicts in more humid areas such as Magoebaskloof, when the density is low, and underpredicts in more arid areas such as Kimberley.

Vand der Merwe (1976) then described a case of a prewetting experiment for a road with a black sandy clay with a PI of 52 and P002 of 46 [and thus of very high potential expansiveness] and a red sandy clay with a PI of 18 and a P002 of 32 [and thus of medium expansiveness] which resulted in predicted versus recorded heaves after 500 days of 96 and 62 mm and 16 and 15 mm, respectively.

He then went on to arbitrarily suggest that the boundaries on the chart be chosen at activities of 0.6 and 0.7 above PIs 23 and 32 and apart from the possible small errors already mentioned this was adopted for the version shown in Figure 1 for TRH9:1982 for roads.

The importance of careful profiling of test holes by the methods of Jennings et al. (1973) immediately after excavation not forgetting the desiccation features of shattering, fissuring and/or slickensiding with depth (see also Netterberg 2019), using more than one method of predicting heave, that oedometer tests also be used for important structures, and of engineering judgement were pointed out by many commentators in conference sessions in and/or in convenors' summaries from the first Southern African conference in 1955 starting from the seminal paper of Jennings (1955) onwards.

## 4 TEST METHODS

As far as can readily be ascertained the test methods used at the former National Building Research Institute (NBRI) to obtain the results used by Kantey & Brink (1952) and Van der Merwe (1964) used air-drying, pulverization of aggregations with a rubber–covered pestle, dry sieving, an ASTM D423 LL device, and a three-point flow curve without presoaking (A.B.A. Brink, D.H. van der Merwe, W.H. Luwes 2002 Pers. Comm.) and nearly all the clay samples had at least about 90 % finer than 420 µm (A.B.A. Brink 2002, pers. comm.).

In 1947 Casagrande had already noted that oven-drying lowers the plasticity of organic soils but that inorganic soils are affected to a much more limited extent. However, the limits may be raised or lowered and for this reason it is not advisable to dry soil samples beforehand; and "drying with any degree of vigor" tends to cause irreversible changes in clays and to be representative of natural properties limit tests must be carried our on undried samples, whilst cycles of wetting tend to increase the plasticity (Grim 1962).

It soon became well-known that oven-drying and even air-drying affects the properties of some soils. Residual soils are particularly affected and these changes cannot be reversed by rewetting (Fourie et al 2012).

As oven-drying at 105 – 110 °C for road indicators was mandatory (Department of Transport 1958, 1970; TRH1:1979) a compromise was introduced in TRH1:1986 Method A1 (b) in which oven-drying at a temperature not exceeding 50 °C was permitted.

Whilst no NBRI laboratory testing manual of the time has been obtained, the test methods were probably similar to those of the National Institute for Road Research (NIRR) (1968) which up to 1965 used the same soils laboratory at the NBRI building except that a overnight presoaking and a BS1377 LL device was used at the NIRR.

An oven-dried but presoaked method using air dispersion and tetrasodium pyrophosphate as an antiflocculant was used for hydrometer analysis. However it is possible that mechanical dispersion and a different antiflocculant was used at the NBRI similar to that of ASTM D422 for the earlier work in the 1940s and 1950s.

As far as can readily be ascertained the test methods used at the Bureau of Reclamation by Holtz and Gibbs (1956) and Holtz (1959) for the results quoted were similar to those of the American Society for Testing Materials (ASTM) of the time and in BUREC

(1974). Samples were not oven-dried and even only air-dried after it was established that drying would not affect their properties.

The usual methods of the time for testing disturbed samples of soil for building foundations both in the United States and South Africa are probably exemplified by those of the ASTM, i.e. D421-39 and 58 for dry preparation of soil for grain-size analysis and soil constants (a wet preparation method was only introduced in 1903),422 for grain-size analysis (422-39 was only for mechanical analysis), 423 for LL, 424 for plasticity and 425 for centrifuge (CME) and 426 for field (FME) moisture equivalents, and 427 for shrinkage factors (SF), i.e. SL, R, VS, and LS), all of the latter requiring that the sample be obtained and prepared according to method D421.

Both D421-39 and 58 specified air-drying at room temperature, aggregations to be broken up in a mortar with a rubber-covered pestle and the fractions passing 420 µm (P420) obtained by dry sieving.

ASTM D422-39 covered only mechanical analysis by hydrometer of the fraction passing 2,00 mm, presoaking for at least 18 hours, a mechanical stirrer, and sodium silicate as a deflocculant. In 1963 the minimum period of presoaking was reduced to 16 hours, the mechanical stirrer made optional to two types of air dispersion devices, and the deflocculant changed to sodium hexametaphosphate, adjusted to a pH of 8 or 9 with sodium carbonate if necessary. No presoaking of the P420 was specified for any of the above except the mechanical analysis.

Similar test methods involving only air-drying at room temperature, pulverization of aggregations and clay lumps with a rubber covered pestle and dry sieving out of the soil fines (P425) should be used when the Kantey–Brink or Williams–Van der Merwe chart criteria are to be applied. The use of dissimilar methods will only add to the existing reproducibility problem reported by Jacobsz & Day (2008) for example.

The closest modern methods would be TMHI: 1986 Methods AI(b) for soil preparation (with only air-drying and without the antiflocculant for the grading), A2 three-point LL, A3 for PL and PI and TMH1:1979 for LS and their presumed SANS equivalents, SANS 3001-GR 2, GR 3, GR 5, GR 12, and GR 10, respectively.

The shrinkage factors reported by both Kantey & Brink (1952), Holtz & Gibbs (1956) and Holtz (1959) were probably determined according to the ASTM D427-39 volumetric method, which was only revised in 1961.

This probably includes the linear shrinkages reported by Kantey & Brink (1952). The linear shrinkage from the FME determined by this method may differ from that determined from the LL by the bar method usually used in South Africa, and a comparison of the two is needed.

Soil samples must not be oven-dried because it can greatly reduce the Atterberg limits and de-aired water must be used in the liquid limit test (Sparks 2011).

Road indicator tests such as those of TMHI : 1986 Method AI(a) involving drying at 105–110$^{o}$ C should not be used on potentially expansive soils for building foundations (e.g. Stott & Theron 2015), fill or roadbeds.

The importance of using the correct test methods does not seem to be generally appreciated and should improve the reproducibility and accuracy of empirical methods of estimating heave using the soil constants and the hydrometer test.

For example, it is probably not widely realized that the only other popular South African index method (Bryne et al. 2019) i.e. that of (Weston 1979) using only the LL of the whole sample and the initial moisture content, was developed using the NITRR methods of the time (NIRR 1968), i.e. wet sample preparation, oven-drying at 55–60 $^{0}$C, overnight presoaking and a BS 1377 LL device, and not those of TMH1: 1979 or the Department of Transport (1970) or its 1948 and its 1958 predecessors , all of which would yield significantly different results. Moreover, all other factors being equal, LLs determined with a BS1377 cup or cone device yield LLs (and therefore also PIs) on average 4.0 units higher than the ASTM/TMH cup throughout the range of LL of 15 – 80 for the 89 soils tested. (Sampson & Netterberg 1984).

## 5 POSSIBLE CHANGES TO THE CHART

### 5.1 *Case histories*

An investigation into the severe longitudinal cracking of two roads on a black clay roadbed in the North West Province (Netterberg & Van Coppenhagen 2022) showed that it was due to the highly expansive clay roadbed despite it classifying as ‘low’ on the TRH9:1982 chart for potential expansivity (Fig. 2).

However, the six samples taken classified as expansive according to the Kantey & Brink (1952) criteria and medium to very high according to the PI alone (the Wilson 1964, 1972, 1976 criteria).

The reasons for this are believed to be the incorrect use of road indicators including oven-drying at 105 – 110 $^{o}$C instead of air-drying as for foundation indicators and the presence of gypsum in the clay which was not noticed or recorded in the soil profiling.

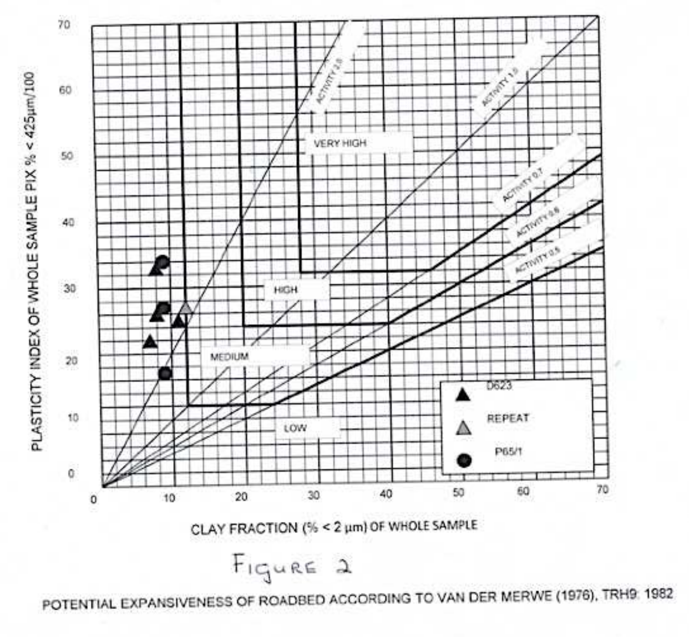


Figure 2. Potential expansiveness of roadbed samples according to TRH9:1982

Three percent of bassanite (plaster of Paris, $CaSO_4 \bullet H_2O$) and 30 % smectite were subsequently found in an XRD analysis. As bassanite is not known from such soils but gypsum ($CaSO_4 \bullet 2H_2O$) is, it must have been formed by the oven-drying and would have aggregated and partially cemented the clay, probably leading to erroneously low Atterberg limits and P002 results, especially the latter.

As gypsum is a well-known flocculant (e.g. Hall & Robinson 1945, Donor & Lynn 1989) – actually all the $Ca^{2+}$, Fe $^{2+}$and Al $^{3+}$ polyvalent cations (Mitchell 1993, Brady & Weil 1999) – affecting particle size analysis:

Even if the bassanite had rehydrated to gypsum during the tests it would probably still have caused flocculation.

In the current method of soil profiling (Jennings et al. 1973, Brink & Bruin 2006) the former requirement of the recording of inclusions (such as calcrete or ferricrete concretions, gypsum, roots, artefacts etc.) was inexplicably dropped, despite the comments of Netterberg (1973) on the necessity thereof, and should be reinstated.

In the testing of the particle size distribution by any method (in South Africa the hydrometer method is usual in engineering and the more accurate pipette method in soil science) it should be noted that it is not rare for some or all of the swelling component to occur in the silt- sand sand-size fractions, not only in soils derived from granite (as in Bühmann 1990), but also those derived from basalt, and as in the little-known Rosebank case in Cape Town semi-consolidated decomposed sediment, and the expansive mineral was an illite-smectite or illite-vermiculite and not the discrete smectite expected (Bühmann et al. 1988).

In the Rosebank case the LL was 62, the PI 24 and the LS 8.0, all plotted below the A-line, the mean heave was 18 mm and was still rising and in the Cape Town area linear shrinkages in excess of 8 associated with cracked structures were seldom found, 6.5 being typical (Kantey 1955), suggesting that the minimum LS criteria of 8 be lowered to 6.0.

Although vermiculite is potentially expansive (Mitchell 1993, Davis 1999) and can absorb large amounts of water (De Bruijn 1955) it is not usually a problem (Nelson & Miller 1999):

- "Kaolinite group – generally non-expansive.
- Mica-like group – includes illites and generally do not pose significant problems.
- Smectite group – includes montmorillonites, which are highly expansive and the most troublesome "clay minerals"

Most clay mineral flakes can exceed 2 µm effective diameter e.g kaolites only up to 2 µm but illites and montmomllonite up to 10µm, whilst the so-called clay fraction up to 2 µm may also contain non-clay minerals (Michell 1993) and thus confirmed for some South Africa soils (e.g. Stephens & Dunvely 1999, Fey 2010). 1993).

The clay minerals at the Roodepan site near Kimberley and the Rosebank site both lacked discrete smectite and the expansive mineral was an illite-smectite or illite-vermiculite interstratification which in the Rosebank case was present only in the fraction coarser than 2 µm (Bühmann et al. 1988).

As both the PI and hydrometer tests have poor reproducibility it makes sense to stop using one of them if we can. From numerous opinions expressed from Wilson in 1964 to Stott & Theron in 2015 it is clear that this is the hydrometer test, which is both more troublesome and expensive.

From the information already discussed and a study of Oloo et al. (1987) it seems safest to eliminate the vertical boundary at 12 % clay and to lower the PI limit from 12 to 10.

A Van der Merwe method which eliminates the necessity for a hydrometer analysis and only uses the PI of the whole sample and the plasticity ratio (i.e. LL/PL) has been proposed by Savage (2007) and an improved method which takes the original in-situ moisture content and the locality into account by Sparks (2011).

As, apart from general dissatisfaction with the hydrometer test, the Van der Merwe (1964, 1976, TRH: 1982) method appears to have withstood the test of time and usage over some 50 years it should be left unchanged unless there is good evidence for any changes.

### 5.2 *The clay content boundaries*

The original boundaries for the P002 "clay" fractions of Williams (1958) are the minama of Holtz & Gibbs (1956), i.e. 12, 18 and 27 (28 in Fig. 1) for the P001 fraction, later revised to 13, 20 and 28 (Holtz 1959).

As the P002 content of a clay might be about 1.1 to 1.5 times that of the P001, these revisions are trivial by comparison, and it is suggested that the current boundaries be left unchanged.

As the clay fraction is always reported on a whole sample basis no correction is necessary for the P425.

### 5.3 *The plasticity index boundaries*

The PI boundaries of 12, 24 and 32 are the minima of Holtz & Gibbs (1956) – (with the 12 coincident with that of Kantey & Brink (1952) – later (Holtz 1959) revised to 15, 25 and 35, and unchanged by BUREC until at least 2004.

In all cases Williams (1958) conservatively selected the minima of a range of results for the P001 and PI only whereas it was intended that all the criteria of the P001, PI and SL be considered together. For example, if the PI alone is used a PI of up to 19 would result in a classification of 'low'.

Shattering due to shrinkage on drying indicates a minimum LL of about 32, PI of 12 or LS of 5.5 and fissuring and slickensiding a minimum LL of 35, PI of 17 or LS of 6.5 (Netterberg 2019), supporting minimum criteria for a potentially expansive soil of a LL of about 32, PI of 12, or LS of 5.5.

In the absence of any further local information it is suggested that the PI boundaries only be conservatively adjusted downwards to a whole sample basis according to the presumed 85–95 % P 420 in both the South African and American samples, i.e. Kantey–Brink criteria on a whole sample basis: LL > 28 and PI 10 and LS > 5.0.and the Williams – Van der Merwe PI boundaries : Low < 10, Medium 10–22, High 22–30, and Very high >30.

In order to avoid confusion between the soil constants as measured and those adjusted for the P425 it is suggested that either the term weighted PI (WPI) as used by Weston (1979) or, better still the 'modulus' obtained by simply multiplying the two to obtain an unambiguous number as is often done in Australia. The criteria would then become:

Revised and adjusted Kantey & Brink criteria: LLM > 2800 and PIM > 1000 and BLSM > 500.

Revised and adjusted Williams–Van der Merwe boundaries: Low < 1000, Medium 1000 – 2 200, High 2 200 – 3 000 Very high > 3 000

## 6 CONCLUSIONS

Apart from general dissatisfaction with the reliability of the hydrometer test both the Skempton–Williams–Van der Merwe chart and Van der Merwe heave prediction appear to have generally withstood the test of some 50 years of usage.

As the boundaries between the different degrees of potential expansiveness are essentially the minima of those of the Bureau of Reclamation for the Western United States they should be almost globally applicable.

The boundaries for clay content are conservative and should probably be left unchanged.

The Kantey–Brink criteria for a potentially expansive soil should be adjusted to a whole sample basis by multiplying by the assumed fraction of 0.9 passing 420μm to become LL > 28 and PI > 10 and, based also on experience., the LS to >5.5 or, conservatively, to > 5.0.

The use of the 'modulus' i.e. simply multiplying by the percentage now passing 425μm rather than the fraction, should be used in order to avoid confusion between the measure and adjusted soil constants.

The adjusted Kantey-Brink criteria for a potentially expansive soil then become LLM > 2 800 and PIM > 1 000 and BLSM > 500.

It is uncertain whether the linear shrinkage calculated from the volumetric shrinkage as in the ASTM test or the linear shrinkage by the direct bar method as commonly used for roads in South Africa since about 1950 was used for the result reported by Kantey & Brink (1952), but the former is likely.

Linear shrinkages determined by the bar method should be designated BLS to avoid confusion.

Using the modulus adjusted plasticity index the boundaries on the chart then become:

Low < 1 000, Medium 1 000 – 2 200, High 2 200 – 3 000 and Very high > 3 000.

The use of the PI (and PIM) boundaries between the degrees of expansiveness alone are probably generally more reliable than the use of the chart and the determination of the percentage passing 2 μm is probably unnecessary.

An ordinary PI exceeding 12 or 10 if adjusted to a whole sample basis is probably a sufficient indicator of a potentially expansive soil.

The bar linear shrinkage from the liquid limit or possibly from the field moisture equivalent may be a better indicator of swell than the plasticity index.

Road indicator test methods must not be used for clay soils, but be compatible with the methods used to derive the classification parameters used.

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