Comparison between the measured and the estimated 1-D heave of expansive soils for seven case studies results using a simple technique

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ABSTRACT
Vanapalli et al. (2010) proposed a simple technique for estimating the 1-D heave in expansive soils. This technique requires only the information of plasticity index I_p, initial void ratio, e_0, and variation of natural water content, Δw for the expansive soils in the active zone. In the present study, the validity of the proposed technique was tested using the data of 7 published case studies from various regions of the world. The results of the study show that there is a reasonably good comparison between the measured and the estimated 1-D heave for all the case studies.

RÉSUMÉ
Vanapalli et al. (2010) ont proposé une technique simple pour estimer le soulèvement unidimensionnel de sols expansifs. Cette technique ne requière que l'indice de plasticité, I_p, l' indice des vides initial, e_0, et la variation de la teneur en eau naturelle, Δw, pour les sols expansifs dans la zone active. Dans la présente étude, la validité de la technique proposée a été mise à l'épreuve d'après les résultats publiés de 7 études de cas de diverses régions du monde. Les résultats de cette étude indiquent qu'il existe une concordance raisonnable entre les soulèvements unidimensionnels mesurés et estimés pour toutes les études de cas.

1 INTRODUCTION

Vanapalli et al. (2010) proposed a simple technique (hereafter referred to as proposed technique) to estimate the 1-D heave of natural expansive soils by deriving a new equation based on the Fredlund (1983) and the Hamberg and Nelson (1984) methods. The proposed technique requires three parameters; namely, corrected swelling index, C_s, suction modulus ratio, C_w, and correction parameter, K which is a function of water content change, Δw and plasticity index, I_p. The parameters C_s and C_w can be determined from laboratory tests; however, these tests are time-consuming and require elaborate testing procedures. Due to this reason, they proposed empirical equations to estimate C_s and C_w that are function of plasticity index, I_p.

The proposed technique was tested on a case study results for Regina clay (i.e., Yoshida et al., 1983). The results showed that the proposed technique can provide more reasonable 1-D heave estimations compared to the Fredlund (1983) and the Hamberg and Nelson (1984) methods.

In the present study, the proposed technique was extended to additional 7 case studies from several regions of the world, which include Australia, Canada, Sudan and the United States. The estimated 1-D heave values using the proposed technique were greater than the measured values for all the case studies. However, the differences between the measured and the estimated heave values were less than 30%.

The analysis of the results suggest that the technique proposed by Vanapalli et al. (2010) is reliable and can be used in geotechnical engineering practice to estimate 1-D heave of natural expansive soils of various regions of the world.

2 ESTIMATION OF 1-D HEAVE IN EXPASIVE SOILS

2.1 Proposed technique

Fredlund (1983) proposed a method to estimate 1-D heave of expansive soils using the changes in void ratio that is a function of corrected swelling pressure, C_s and initial and final stress states (Eq. [1]).

$$\Delta H = \frac{\Delta e(= e_f - e_0)}{1 + e_0} H = C_s \frac{H}{1 + e_0} \log\left(\frac{P_f}{P'_s}\right) \qquad [1]$$

where:

H = thickness of the soil layer
P_f = final stress state
P'_s = corrected swelling pressure (i.e., initial stress)
C_s = corrected swelling index
e_0, e_f = initial and final void ratio, respectively

Hamberg and Nelson (1984) used the concept of suction modulus ratio, C_w to determine the 1-D heave in expansive soils (Eq. [2]).

$$\Delta H = C_w \frac{H}{1 + e_0} \Delta w \qquad [2]$$

where:

C_w = suction modulus ratio (= $\Delta e/\Delta w$)
Δe = change in void ratio
Δw = change in water content

The suction modulus ratio, C_w represents the variation of void ratio (i.e., volume in 1-D heave) of soil specimens with respect to water content (Eq. [3]; Figure 1).

$$C_w = \frac{e_f - e_0}{w_f - w_0} = \frac{\Delta e}{\Delta w} \quad [3]$$

where:

w_f, w_0 = water contents corresponding to e_f and e_0, respectively

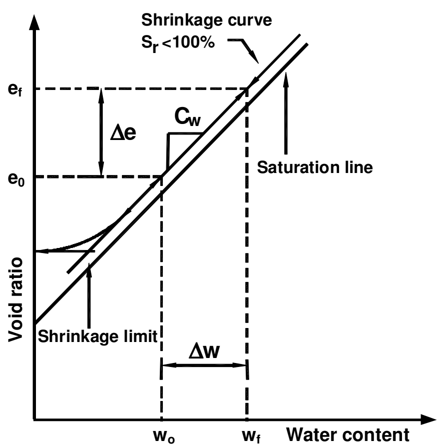

Figure 1. Procedure for determining C_w from water content versus void ratio relationship (modified after Hamberg, 1985).

Eq. [1] can be re-written as below:

$$\Delta H = C_s \frac{H}{1+e_0} \log P_f - C_s \frac{H}{1+e_0} \log P_s' \quad [4]$$

The positive (i.e., first term) and the negative (i.e., second term) sign in Eq. [4] indicate compression and heave due to overburden and swelling pressure, respectively. Hence, the heave calculated using the second term of Eq. [4] is proportional to the heave estimated using Eq. [2] as below:

$$C_s \frac{H}{1+e_0} \log P_s' \propto C_w \frac{H}{1+e_0} \Delta w \quad [5]$$

Eq. [5] can be re-written as Eq. [6] by introducing correction parameter, K.

$$P_s' = \frac{10^{\left(\frac{C_w}{C_s}\Delta w\right)}}{K} \quad [6]$$

Substituting Eq. [6] into Eq. [1] yields

$$\Delta H = C_s \frac{H}{1+e_0} \log \left\{ \frac{K P_f}{10^{\left(\frac{C_w}{C_s}\Delta w\right)}} \right\} \quad [7]$$

In the following section, simple empirical methods to estimate the key parameters in the proposed technique (i.e., Eq. [7]) are provided.

2.2 Estimation of the parameters in the proposed technique

The amount of swell in expansive soils is governed by the change in water content, Δw (Chen, 1975), which can be obtained from field investigation studies. However, this information is not available for most of the case studies published in the literature. Hence, in the present study, Δw was calculated using Eq. [8] (Fredlund and Rahardjo, 1993) based on the assumption that the soils attain saturated (i.e., S_f = 100%) condition. Such an assumption provides conservative estimations (i.e., maximum 1-D heave).

$$\Delta w = S_f \Delta e / G_s + e_0 \Delta S / G_s \quad [8]$$

where:

S_f = final degree of saturation
ΔS = change in degree of saturation
G_s = specific gravity

The C_w value can be measured from Clod tests. For silty clay, clayey and expansive soils, the void ratio linearly increases with increasing water content beyond shrinkage limit (Hamberg, 1985; Tripathy et al. 2002) (see Figure 1). Using this concept, an empirical relationship between C_w and I_p was developed using the data published in the literature (see Figure 2).

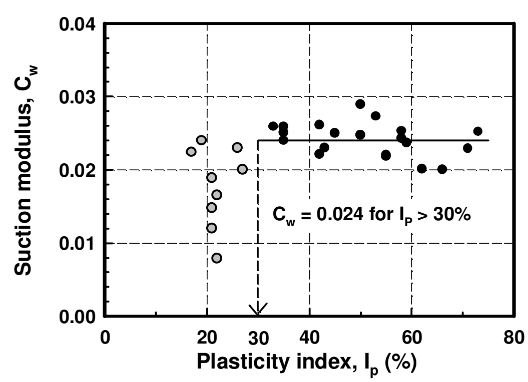

Figure 2. Relationship between I_p and C_w using data published in the literature.

The C_w values show large scatter for the I_p values less than 30%; however, relative constant value (i.e., 0.024) was observed for the I_p values greater than 30%. This indicates that C_w = 0.024 can be used in the estimation of 1-D heave of expansive soils since I_p values of typical expansive soils are generally greater than 30% (Eq. [9]).

$$C_w = 0.024 \quad \text{for} \quad I_p \geq 30\% \qquad [9]$$

The corrected swelling index, C_s can be determined using the procedures detailed by Fredlund (1983) from 1-D Constant Volume Swell (CVS) oedometer test. Determination of the C_s is time-consuming and requires elaborate testing equipment as detailed earlier. It also needs different corrections with respect to swelling pressure, compressibility of the apparatus, and sample disturbance. Hence, a simple empirical relationship between C_s and I_p (Eq. [10]) was developed using the data available in the literature as shown in Figure 3.

$$C_s = 0.0193\, e^{0.0343(Ip)} \qquad [10]$$

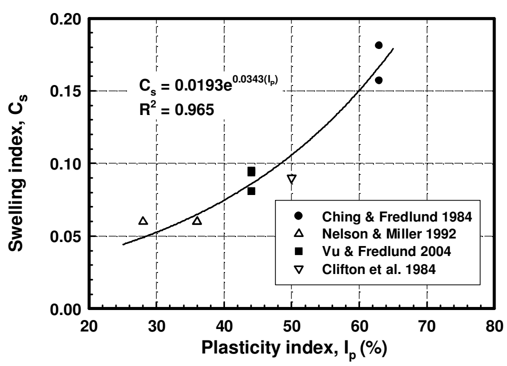

Figure 3. Relationship between C_s and I_p.

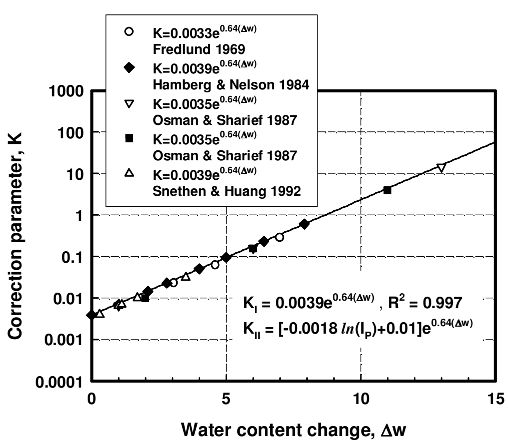

Figure 4. Relationship between K and Δw.

Figure 4 shows the empirical relationship between correction parameter, K and Δw for the data from five case studies. The analysis showed that K is not a constant value but instead varies with I_p (Eq. [11]). The best-fitting curve for the all data was estimated as Eq. [12] with relatively high R-squared value. The correction parameters, K in Eq. [11] and [12] are hereafter referred to as K_I and K_{II}, respectively and both were used in the analysis.

$$K_I = \left[-0.0018\ln(I_p) + 0.01\right] e^{0.64(\Delta w)} \qquad [11]$$

$$K_{II} = 0.0039\, e^{0.64(\Delta w)} \qquad [12]$$

3 SUMMARY OF CASE STUDIES (I)

In this section, an attempt is made to compare the measured and the estimated 1-D heave values for the five case studies used in the development of the correction parameter, K (Eqs. [11] and [12]). The study sites are from Canada, Sudan and the United States.

3.1 Case Study A (Regina Clay, Canada)

Hamilton (1963, 1968) studied the swelling behaviour of expansive soil beneath an industrial building in Regina, Saskatchewan. The swelling in the building was due to the flooding associated with a break in the water line, which occurred during the summer of 1962. Several investigations were undertaken to study the swelling of the building slab floor. The investigations included conducting 1-D consolidation and swelling tests on samples collected from three different depths below the concrete slab floor of the industrial building. In addition, ground movement gauges were also installed at three depths and precise elevation readings were measured over a period of time. The investigation studies are summarized in Fredlund (1969). The measured 1-D heave of the industrial building was 84 mm.

The heave calculated using Eq. [1] was 151 mm (Fredlund, 1969). The values required for performing the calculation includes C_s and P'_s that were measured from CVS tests. The ratio of the calculated heave to the measured value was 1.79.

The initial water content at the site for three different depths was available. However, the final water content was not available. Therefore, the water content change, Δw was calculated using Eqs. [1] and [8]. The final degree of saturation, S_f was assumed to be 100%. The initial and final water content distribution with depth is shown in Figure 5. The average plasticity index, I_p of the Regina clay in the active zone depth was 42%. The C_w (Eq. [9]) and the C_s (Eq. [10]) values were estimated as 0.024 and 0.094, respectively. The estimated heave values using K_I ($\Delta H(K_I)$) and K_{II} ($\Delta H(K_{II})$) were 110 mm (Ratio = 1.31) and 103 mm (Ratio = 1.23), respectively. The details of the Case Study A are summarized in Table 1.

The $\Delta H(K_I)$ and $\Delta H(K_{II})$ values using the measured C_s values with depth were estimated as 98 mm (Ratio = 1.17) and 107 mm (Ratio = 1.27), respectively.

The reason for the estimated heave values being greater than measured values may be attributed to the assumption that the soil is saturated. Such an assumption may not be valid in practice since the entire active zone of expansive soils may not have attained a state of saturated condition.

3.2 Case Study B (Fort Collins, USA)

The field test site is located in Fort Collins, Colorado and has a cool and semi-arid climate. The 1-D heave data were monitored on lightly loaded plastic barriers along with the changes in water content measured using nuclear moisture gauges for a period of twenty months (Miller et al., 1995). The initial and final water contents with depth are shown in Figure 6.

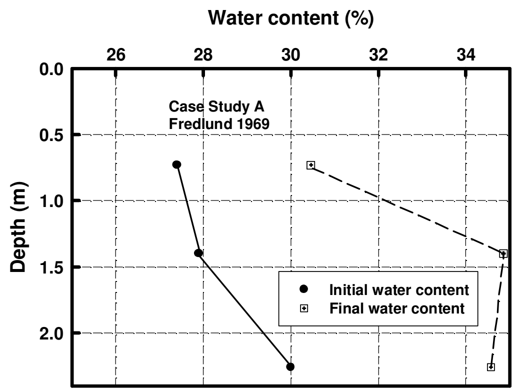

Figure 5. Water content variation with respect to depth for the Case study A (modified after Fredlund, 1969).

Table 1. Summary of Case Study A.

Depth (m)	Δw** (%)	e_0*	C_s**	C_w**	$\Delta H(K_I)$ (mm)	$\Delta H(K_{II})$ (mm)
0.73	3.10	0.859	0.094	0.024	110	103
1.40	6.99	0.983				
2.26	4.58	0.975				
Measured heave (mm)					84	
Ratio (estimated/measured)					1.31	1.23
Fredlund (1969) method (Eq. [1]) (mm)					151	

*measured; **estimated
$\Delta H(K_I)$, $\Delta H(K_{II})$: estimated heave using KI and KII, respectively (Available for other tables)

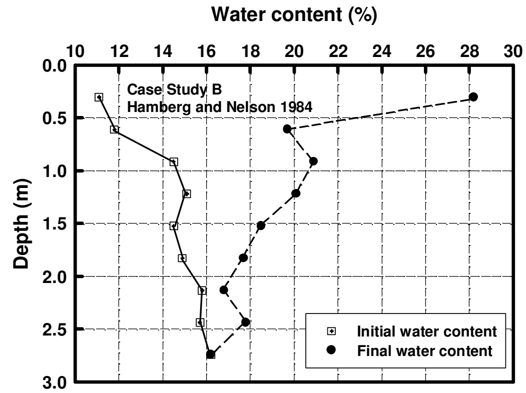

Figure 6. Water content variation with depth for the Case Study B (Hamberg and Nelson, 1984).

The active zone of the test site mainly consists of Pierre Shale and the average I_p value is 28%. The measured (using Clod test) and the estimated (Eq. [9]) C_w values are 0.016 and 0.024, respectively. The estimated C_s value (Eq. [10]) is 0.05. The details of the Case Study B are summarized in Table 2. The $\Delta H(K_I)$ and $\Delta H(K_{II})$ values provide reasonable good comparison with the Ratio of 1.29 and 1.28, respectively, while the heave estimated using the Hamberg and Nelson (1984) method was significantly overestimated.

The $\Delta H(K_I)$ and $\Delta H(K_{II})$ values using the measured C_w were estimated as 88 mm (Ratio = 1.22) and 87 mm (Ratio = 1.21), respectively. This indicates that 1-D heave can be more accurately estimated with the measured C_w.

Table 2. Summary of Case study B.

Depth (m)	Δw (%)*	e_0*	C_w**	$\Delta H(K_I)$ (mm)	$\Delta H(K_{II})$ (mm)
0.305	17.1	1.19	0.024	92	93
0.610	7.9	0.98			
0.915	6.4	0.98			
1.220	5.0	0.86			
1.525	4.0	0.78			
1.830	2.8	0.78			
2.135	1	0.79			
2.440	2.1	0.83			
2.745	0	0.82			
Measured heave (mm)				72	
Ratio (estimated/measured)				1.29	1.28
Hamberg and Nelson (1984) method (mm)				116	

3.3 Case Study C and D (Sudan)

Osman and Sharief (1987) measured 1-D heave values of expansive soil deposits at two different locations in Sudan. The properties of the collected soil samples at the two test site sub-soils were determined from laboratory tests. Field data collected include soil surface movements and water content distributions with depth after long term flooding in the both sites.

The measured heave values at the sites were 142 mm and 150 mm, respectively. The initial and final water content distribution with depth for the Case Study C and D are shown in Figure 7(a) and (b), respectively. The average I_p values in the active zone for the two different sites were 35% (Case Study C) and 34% (Case Study D), respectively. The C_w = 0.024 for the both sites (Eq. [9]) and the C_s = 0.064 and 0.062 (Eq. [10]) for the Case Study C and D, respectively.

Osman and Sharief (1987) conducted oedometer test (i.e., swell overburden load test, CVS test and free swell test) to estimate the 1-D heave, which resulted in the maximum heave of 295 mm for the two different sites. This indicates that the laboratory oedometer test can significantly overestimate the 1-D heave compared to the field conditions.

For the Case Study C, the measured heave was 142 mm and the $\Delta H(K_I)$ and $\Delta H(K_{II})$ values were estimated as 157 mm (Ratio = 1.11) and 154 mm (Ratio = 1.08), respectively. For the Case Study D, the measured heave was 150 mm and the $\Delta H(K_I)$ and $\Delta H(K_{II})$ values were estimated as 159 mm (Ratio = 1.06) and 155 mm (Ratio = 1.03), respectively. The details of the Case Study C and D are summarized in Table 3 and Table 4, respectively.

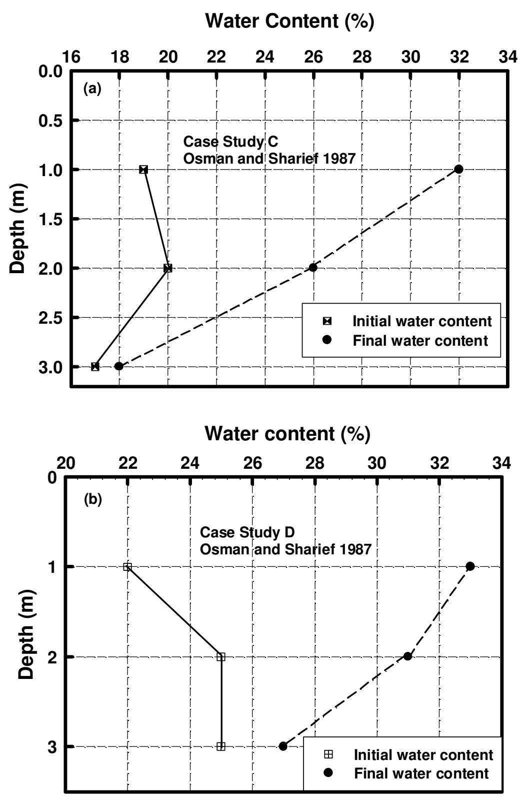

Figure 7. Water content variation with depth for the Case Study a) C and b) D (Osman and Sharief, 1987).

Table 3. Summary of Case Study C.

Depth (m)	Δw^* (%)	P_f * (kPa)	$\Delta H(K_I)$ (mm)	$\Delta H(K_{II})$ (mm)
1	13	18	154	157
2	6	38		
3	1	56		
Measured heave (mm)			142	
Ratio (estimated/measured)			1.08	1.11
Oedometer test (mm) (Osman and Sharief, 1987)			295	

Table 4. Summary of Case study D.

Depth (m)	Δw^* (%)	P_f * (kPa)	$\Delta H(K_I)$ (mm)	$\Delta H(K_{II})$ (mm)
1	11	17.5		
2	6	38	155	159
3	2	56		
Measured heave (mm)			150	
Ratio (estimated/measured)			1.03	1.06
Oedometer test (mm) (Osman and Sharief, 1987)			295	

3.4 Case Study E (Oklahoma City, USA)

The Case Study E is based on the studies by Snethen and Huang (1992) for the test site near Wynnewood, Oklahoma City. The climate in this region is classified as moist sub-humid to dry sub-humid.

Undisturbed soil samples were collected from five continuously sampled borings within a 1 m radius to a depth of approximately 4 m to obtain reliable initial natural water contents. The soils at the site consist of tan and reddish brown, which are moderate to high plasticity clay. The ground water table was observed to be at a depth of 3 m from site investigation studies. The measured heave was 180 mm.

Snethen and Huang (1992) used soil suction method to estimate 1-D heave. The initial soil suction at the test site was estimated using the filter paper technique (ASTM D5298 - 03, 2007). The final soil suction can be estimated using one of the assumptions as follows as per the guidelines provided by Russam and Coleman (1961), Russam and Dagg (1965), Richards (1966), and Johnson and Snethen (1978): (i) suction linearly increases with depth in the active zone, (ii) soil suction corresponding to saturated water content, (iii) soil suction is constant with a certain equilibrium value and (iv) soil suction is equal to zero throughout the depth of active zone. The assumptions (i), (iii) and (iv) are neither realistic with respect to field behaviour observations nor useful in the prediction or estimation of the 1-D heave. Assumption (ii) requires the information of saturated water content to estimate the final suction value; this is probably the most realistic and practical approach for estimating potential heave since it involves measured physical properties of the soils rather than assumed relationships (Snethen, 1980). The 1-D heave estimated based on the assumption (ii) was 157 mm (Ratio = 0.87) (Snethen and Johnson, 1978).

The information related to the initial and final water content was not available in Snethen and Huang (1992). Therefore, the water content change, Δw was back-calculated based on the ratio of water content change to soil suction change (i.e., Δw/Δψ) for each soil layer. These details are summarized in Table 5.

Snethen and Huang (1992) provided the details of final stresses at the test site. The maximum I_p determined using the soil samples in the active zone was 33%. The estimated C_w (Eq. [9]) and C_s (Eq. [10]) values were 0.024 and 0.06, respectively. The ratio of estimated heave to measured heave was estimated as 1.61 and 1.62 for $\Delta H(K_I)$ and $\Delta H(K_{II})$, respectively (Table 6).

In expansive soils, even small changes in natural water content conditions can contribute to detrimental swelling (Chen, 1975). In the proposed technique, the water content change was back-calculated using the assumed soil suction changes. As such the water content readings were not reliable and may have contributed to the overestimation of the 1-D heave (see Table 6). This discussion demonstrates the proposed technique is sensitive to water content measurements; therefore, utmost care should be taken in collecting water content data for reliable estimation of the 1-D heave.

Table 5. Soil properties from Wynewood site (Snethen and Huang, 1992).

Depth (m)	Suction change Δψ* (kPa)	Δw/Δψ*	Δw** (%)
0.5	1899	2.8	12
1	1716	4.44	19.65
1.5	4159	3.66	16.94
2	5941	4.78	22.86
2.5	1956	3.47	14.92

Table 6. Summary of Case study E.

Depth (m)	e_0*	P_f * (kPa)	$\Delta H(K_I)$ (mm)	$\Delta H(K_{II})$ (mm)
0.5	0.600	4.5	290	291
1	0.546	12.9		
1.5	0.479	21.1		
2	0.438	29.0		
2.3	0.485	38.0		
Measured heave			180	
Ratio			1.61	1.62
Snethen and Johnson (1978) method (mm)			157	

4 ANALYSIS AND SUMMARY OF CASE STUDIES (I)

Figure 8 provides the comparison between the measured 1-D heave values and those estimated using the proposed technique for the five case studies. As it can be seen, good agreements were observed except the Case Study E (i.e., Snethen and Huang, 1992). This may be attributed to the indirect measurements of water content change, Δw for the Cast Study E, which was derived from the suction data obtained using the filter paper method.

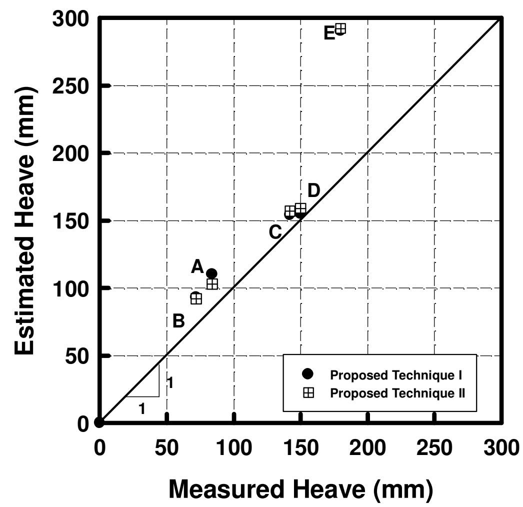

Figure 8. Comparison between the measured and estimate 1-D heave of the five case studies using the proposed technique.

The Fredlund (1983) method requires corrected swelling index, C_s and corrected swelling pressure, P'_s to calculate the 1-D heave. Both these properties can be measured from the CVS tests. Although C_s can be estimated using Eq. [10], Fredlund (1983) method cannot be used for providing comparisons between the measured and the estimated heave values without the information of measured P'_s. The measured C_s values from CVS tests were available only for one case study (i.e., Fredlund, 1969). Hamberg and Nelson (1984) were the only investigators who measured the suction modulus ratio, C_w using Clod test. The C_w and C_s values were estimated using Eq. [9]and Eq. [10] when the information was not available.

The proposed technique provides reasonable results for all the four case studies. Especially, excellent comparisons were observed for the case studies that the water content variation with depth data were directly available from field investigation studies (i.e., Case Studies B, C and D).

The Hamberg and Nelson (1984) method overestimated the 1-D heave for all the five case studies. This may be attributed to ignoring the effect of overburden pressure.

The ratios (= estimated heave/measured heave) using the proposed technique were greater than unity for all the case studies. However, the differences between the measured and estimated heaves were less than 30%. In other words, the 1-D heave estimated using the proposed technique provides conservative values with reasonable accuracy. The reason for the significant discrepancy between the measured and the estimated 1-D heave for the Case Study E (i.e., Snethen and Huang, 1992) was already discussed earlier.

5 SUMMARY OF CASE STUDIES (II) (FROM MARYLAND, AUSTRALIA)

The results of the previous five case studies suggest that the proposed technique is simple and reliable to use. The estimated heave values however were highly sensitive to water content change readings. In this section, data obtained from two more case studies are summarized and comparison between the measured in-situ surface heave and the estimated 1-D heave using the proposed technique are provided. The soil properties of these case studies were not used in deriving the empirical relationships such as C_w, C_s, and K. In other words, they are independent case studies. The objective of using these additional case studies is to further understand the limitations of the proposed technique.

The test site (called Maryland site in Newcastle, Australia) was used to measure the long-term (i.e., 7 years) behaviour of 1-D heave of the regional expansive soils in both open and covered areas. The region has near coastal climate with an annual rainfall typically between 1000 and 1200 mm per year. The open area (Case Study F) test site facilities the measurement of free field heave, while the test in covered area (with a surcharge of approximate 3 kPa; Case Study G) provides valuable data to understand the differences in 1-D heave characteristics in comparison to the open area.

The test field site was extensively instrumented and data was collected over a long period of time. The instrumentation includes 154 surface movement indicators, 28 subsurface movement indicators, 9 neutron probe for in-situ measurement of soil water content and 6 in-situ filter paper devices for measurement of soil suction. In spite of all the care, it was reported that the investigators had difficulties in collecting reliable data sometimes due to problems associated with instrumentation. The predicted ground movement by using the Fityus and Smith (1998) method for the open area was 41 mm, which is underestimated compared to the measured value (i.e., 75 mm) (Fityus et al., 2004).

The depth of the active zones for the open area and cover area are approximately 1.5 m and 0.5 m, respectively. The plasticity index, I_p of the clay specimen from the active zone is 45%. The average dry unit weight is 15.52 kN/m^3. The suction modulus ratio, C_w and the corrected swelling index, C_s can be estimated to be 0.024 (Eq. [9]) and 0.09 (Eq. [10]). The distribution of initial and final water content with depth for the open (Case Study F) and the covered area (Case Study G) of the test sites are shown in Figure 9 and Figure 10, respectively. Comparisons between the measured heave and that estimated using the proposed technique for the Case Study F and G are summarized in

Table 7 and Table 8, respectively.

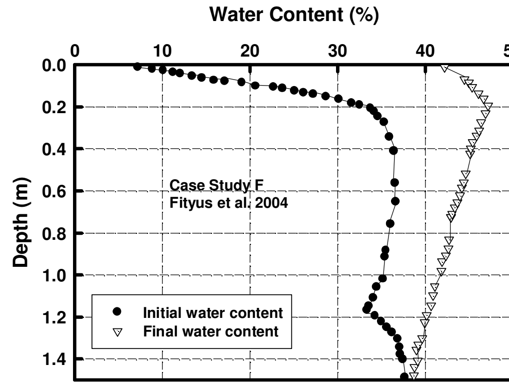

Figure 9. Distribution of in-situ moisture content with depth (open area).

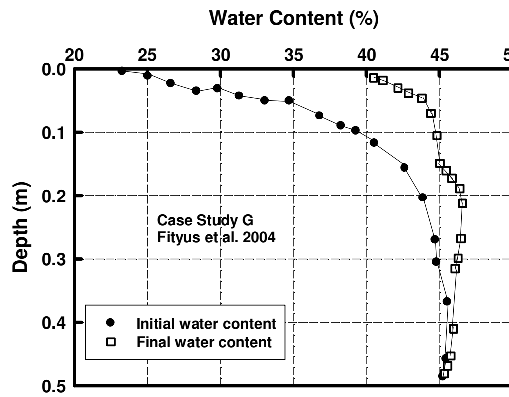

Figure 10. Distribution of in-situ moisture content with depth (cover area).

Table 7. Summary of Case Study F.

Depth (m)	Δw* (%)	e_0*	$\Delta H(K_I)$ (mm)	$\Delta H(K_{II})$ (mm)
0.25	11.89	0.69	101	109
0.50	8.1	0.69		
0.75	7	0.69		
1.00	6.59	0.69		
1.25	4.16	0.69		
1.50	1.01	0.69		
Measured heave			75	
Ratio (estimated/measured)			1.35	1.45
Fityus and Smith (1998) method (mm)			41	

Table 8. Summary of Case study G.

Depth (m)	Δw^* (%)	e_0^*	$\Delta H(K_I)$ (mm)	$\Delta H(K_{II})$ (mm)
0.1	5.2	0.794	40	38
0.2	2.7	0.794		
0.3	1.5	0.794		
0.4	0.6	0.794		
0.5	0.1	0.794		
Measured heave			35	
Ratio			1.14	1.09
Fityus and Smith (1998) method (mm)			41	

There is an excellent comparison between the measured and the estimated 1-D heave for the covered area (see Table 8); however, there are significant differences for open area test site (see

Table 7). Fityus et al. (2004) summarized that the neutron probes used in open area has proven to be an effective means for long-term monitoring of in-situ water content changes. However, the extraction of absolute water content data from neutron probe counts has proven to be problematic. Further, the shrinkage cracks extended from the ground surface down to the subsurface in the open area. While the first reason of the problems associated with collection of water content using neutron probe has been of some concern; secondly, the measurements of water contents in the zone of shrinkage cracks were also a challenge. These two factors may have contributed to errors in the data collection of water content distribution with depth in open area.

6 SUMMARY OF ALL CASE STUDIES

Expansive soils in all regions of the world pose various problems particularly to the lightly loaded structures. Several researchers and practitioners have significantly contributed 1-D or 3-D heave prediction techniques to understand the expansive soils behaviour. However, most of these techniques require time-consuming laboratory and/or in-situ tests, which are expensive and difficult to be performed by geotechnical engineers. Hence, in the present study, a simple and inexpensive 1-D heave prediction technique that can be extended for all natural expansive soils was proposed. This technique requires only the information of plasticity index I_p, the initial void ratio, e_o and the variation in natural water content, Δw with depth in the active zone of natural expansive soils. The proposed technique was tested using the data from 7 case studies published in the literature.

The results of the studies presented in this paper show that the proposed technique can be used to obtain reasonable 1-D heave in natural expansive soils with the aid of reliable information (i.e., Atterberg Limits, original void ratio, in-situ moisture variations in the active zone). The proposed technique is simple and should encourage geotechnical engineers to implement in practice.

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