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, \( \Delta w \) 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, \( \Delta w \), 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, \( \Delta w \) 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]).

\[
\Delta H = \Delta e \left( \frac{e_f - e_i}{1+e_i} \right) \left( H = C_s \log \left( \frac{P_f}{P_s} \right) \right)
\]

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
- \( e_i, e_f \) = 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]).

\[
\Delta H = C_w \left( \frac{H}{1+e_i} \right) \Delta w
\]
where:
\[ C_w = \text{suction modulus ratio} = \frac{\Delta e}{\Delta w} \]
\[ \Delta e = \text{change in void ratio} \]
\[ \Delta w = \text{change in water content} \]

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).

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

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:
\[ \Delta H = C_s \frac{H}{1 + e_0} \log P_0 - C_s \frac{H}{1 + e_0} \log P_0' \]

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:
\[ C_s \frac{H}{1 + e_0} \log P_0' \propto C_w \frac{H}{1 + e_0} \Delta w \]

Eq. [5] can be re-written as Eq. [6] by introducing correction parameter, \( K \).
\[ P_0' = 10^{\frac{C_w \Delta w}{K}} \]

\[ \Delta H = C_s \frac{H}{1 + e_0} \log \left( \frac{K P_1}{10^{\frac{C_w \Delta w}{C_s \Delta w}}} \right) \]

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, \( \Delta w \) (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, \( \Delta w \) 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).

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

where:
\( S_f \) = final degree of saturation
\( \Delta S \) = 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]).

$$C_w = 0.024 \text{ for } I_p \geq 30\%$$

[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.

$$C_s = 0.0193 e^{0.0343 I_p}$$

[10]

Figure 3. Relationship between $C_s$ and $I_p$.

Figure 4 shows the empirical relationship between correction parameter, $K$ and $\Delta w$ 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.

$$K_i = \left[-0.0018 \ln(I_p) + 0.1 \right] e^{-0.64(\Delta w)}$$

[11]

$$K_{II} = 0.0039 e^{0.64(\Delta w)}$$

[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, $\Delta w$ was calculated using Eqs. [1] and [8]. The final degree of saturation, $S_i$ 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 0.049, 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 6. Water content variation with depth for the Case Study B (Hamberg and Nelson, 1984).](image)

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_v value (Eq. [10]) is 0.05. The details of the Case Study B are summarized in Table 2. The ΔH(K_I) and ΔH(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 ΔH(K_I) and ΔH(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.

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Δw (%)</th>
<th>e_o</th>
<th>C_w</th>
<th>ΔH(K_I)</th>
<th>ΔH(K_II)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.305</td>
<td>17.1</td>
<td>1.19</td>
<td>0.024</td>
<td>92</td>
<td>93</td>
</tr>
<tr>
<td>0.610</td>
<td>7.9</td>
<td>0.98</td>
<td>0.64</td>
<td>0.98</td>
<td></td>
</tr>
<tr>
<td>0.915</td>
<td>5.0</td>
<td>0.86</td>
<td>0.98</td>
<td>0.98</td>
<td></td>
</tr>
<tr>
<td>1.220</td>
<td>4.0</td>
<td>0.78</td>
<td>0.98</td>
<td>0.98</td>
<td></td>
</tr>
<tr>
<td>1.525</td>
<td>2.8</td>
<td>0.78</td>
<td>0.98</td>
<td>0.98</td>
<td></td>
</tr>
<tr>
<td>1.830</td>
<td>2.1</td>
<td>0.83</td>
<td>0.98</td>
<td>0.98</td>
<td></td>
</tr>
<tr>
<td>2.135</td>
<td>0</td>
<td>0.82</td>
<td>0.98</td>
<td>0.98</td>
<td></td>
</tr>
<tr>
<td>2.440</td>
<td>2.1</td>
<td>0.83</td>
<td>0.98</td>
<td>0.98</td>
<td></td>
</tr>
<tr>
<td>2.745</td>
<td>0</td>
<td>0.82</td>
<td>0.98</td>
<td>0.98</td>
<td></td>
</tr>
<tr>
<td>Measured heave (mm)</td>
<td>72</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ratio (estimated/measured)</td>
<td>1.29</td>
<td>1.28</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hamberg and Nelson (1984) method (mm)</td>
<td>116</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### 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 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 \( \Delta H(K_1) \) and \( \Delta H(K_2) \) 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 \( \Delta H(K_1) \) and \( \Delta H(K_2) \) 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.

### Table 3. Summary of Case Study C.

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>( \Delta w^\ast ) (%)</th>
<th>( P_i^\ast ) (kPa)</th>
<th>( \Delta H(K_1) ) (mm)</th>
<th>( \Delta H(K_2) ) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>13</td>
<td>18</td>
<td>154</td>
<td>157</td>
</tr>
<tr>
<td>2</td>
<td>6</td>
<td>38</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>56</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Measured heave (mm)</td>
<td>142</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ratio (estimated/measured)</td>
<td>1.08</td>
<td>1.11</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Oedometer test (mm)</td>
<td>295</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(Osman and Sharief, 1987)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Table 4. Summary of Case Study D.

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>( \Delta w^\ast ) (%)</th>
<th>( P_i^\ast ) (kPa)</th>
<th>( \Delta H(K_1) ) (mm)</th>
<th>( \Delta H(K_2) ) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>11</td>
<td>17.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>6</td>
<td>38</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>2</td>
<td>56</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Measured heave (mm)</td>
<td>150</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ratio (estimated/measured)</td>
<td>1.03</td>
<td>1.06</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Oedometer test (mm)</td>
<td>295</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(Osman and Sharief, 1987)</td>
<td></td>
<td></td>
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</tr>
</tbody>
</table>

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, $\Delta w$ was back-calculated based on the ratio of water content change to soil suction change (i.e., $\Delta w/\Delta \psi$) 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).

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Suction change $\Delta \psi^*$ (kPa)</th>
<th>$\Delta w/\Delta \psi^*$ (%)</th>
<th>$\Delta w^*$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>1899</td>
<td>2.8</td>
<td>12</td>
</tr>
<tr>
<td>1</td>
<td>1716</td>
<td>4.44</td>
<td>19.65</td>
</tr>
<tr>
<td>1.5</td>
<td>4159</td>
<td>3.66</td>
<td>16.94</td>
</tr>
<tr>
<td>2</td>
<td>5941</td>
<td>4.78</td>
<td>22.86</td>
</tr>
<tr>
<td>2.5</td>
<td>1956</td>
<td>3.47</td>
<td>14.92</td>
</tr>
</tbody>
</table>

Table 6. Summary of Case study E.

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>$e_0^*$</th>
<th>$P'_s^*$ (kPa)</th>
<th>$\Delta H(K_I)$ (mm)</th>
<th>$\Delta H(K_II)$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>0.600</td>
<td>4.5</td>
<td>290</td>
<td>291</td>
</tr>
<tr>
<td>1</td>
<td>0.546</td>
<td>12.9</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.5</td>
<td>0.479</td>
<td>21.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>0.438</td>
<td>29.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.3</td>
<td>0.485</td>
<td>38.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Measured heave</td>
<td></td>
<td></td>
<td>180</td>
<td></td>
</tr>
<tr>
<td>Ratio</td>
<td>1.61</td>
<td>1.62</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

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.

Table 7. Summary of Case Study F.

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>$\Delta w^*$ (%)</th>
<th>$e_0^*$</th>
<th>$\Delta H(K_I)$ (mm)</th>
<th>$\Delta H(K_{II})$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25</td>
<td>11.89</td>
<td>0.69</td>
<td>101</td>
<td>109</td>
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<td>0.50</td>
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<td>0.75</td>
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<td>1.25</td>
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<td>1.50</td>
<td>1.01</td>
<td>0.69</td>
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<td></td>
</tr>
</tbody>
</table>

Measured heave 75

Ratio (estimated/measured) 1.35 1.45

Fityus and Smith (1998) method (mm) 41
void ratio, in-situ moisture variations in the active zone).

aid of reliable information (i.e., Atterberg Limits, original
reasonable 1-D heave in natural expansive soils with the
proposed technique can be used to obtain
literature.

using the data from 7 case studies published in the
expansive soils. The proposed technique was tested
effective means for long-term monitoring of in-situ water
neutron probes used in open area has proven to be an
differences for open area test site (see
Table 8); however, there are significant

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, ∆w 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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<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Δw* (%)</th>
<th>e_o*</th>
<th>ΔH(K_I) (mm)</th>
<th>ΔH(K_II) (mm)</th>
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<tr>
<td>0.5</td>
<td>0.1</td>
<td>0.794</td>
<td></td>
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</tr>
<tr>
<td>Measured heave</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Ratio</td>
<td>1.14</td>
<td>1.09</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Fityus and Smith (1998) method (mm) 41


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