A state-of-the-art review of 1-D heave prediction methods for expansive soils

Article in International Journal of Geotechnical Engineering · January 2012
DOI: 10.3328/IJGE.2012.06.01.15-41

CITATIONS
25

READS
366

2 authors, including:

Sai K. Vanapalli
University of Ottawa
223 PUBLICATIONS 2,552 CITATIONS
SEE PROFILE

Some of the authors of this publication are also working on these related projects:

Methods in Bio-Mediated CaCO3 Production for Improving the Liquefaction Resistance of Loose, Sandy Soils View project

Soil-pipeline system behavior extending the principles of saturated and unsaturated soil mechanics View project
A state-of-the art review of 1-D heave prediction methods for expansive soils

ABSTRACT: Expansive soils are typically in a state of unsaturated condition and are widely found in semi-arid and arid regions of the world. The engineering properties of these soils are highly sensitive to changes in water content and suction in the active zone depth. One of the key problems associated with expansive soils is their swelling characteristics due to an increase in their natural water content. Practicing geotechnical engineers consider reliable prediction of the heave behavior of expansive soils and understanding its impact on the structures as one of their greatest challenges. Several researchers and practitioners from all around the world have made significant contributions over the past 60 years to better understand these problematic soils. This paper synthesizes information of various techniques available in literature for estimating the swelling pressure and the 1-D heave behavior of expansive soils. In addition, the limitations of using them in geotechnical engineering practice are discussed.

KEYWORDS: Unsaturated soils, expansive soils, suction, swelling pressure, 1-D heave prediction.

1. INTRODUCTION

Conventional soil mechanics used for interpreting the engineering behavior of saturated soils in practice cannot be applied for expansive soils as they are typically in a state of unsaturated condition. The engineering behavior of expansive soils is sensitive to both the changes in water content and as well as suction. Swell pressures associated with changes in water content and suction of unsaturated expansive soils are responsible to undesirable heave movements contributing to significant damages to the structures. Expansive soils cause damages to structures, pavements, pipelines, slopes, irrigation channels and other constructions and contribute to significant financial losses that far exceed other natural losses such as earthquakes and tornados (Jones and Holtz, 1973). The heave in an expansive soil deposit typically continues until the degree of saturation is equal to 100%.

Practicing engineers rely on using a variety of design, construction, and stabilization techniques to reduce losses associated with expansive soils (Puppala et al. 2006). The success of these techniques or procedures depends on key information such as the reliable determination or estimation of the swelling pressure and the soil heave. Reliable determination or estimation of the swelling pressure and the soil heave however has proven to be difficult to date. Due to these reasons, the design of foundations in expansive soils has been one of the greatest challenges to the practicing geotechnical engineers (Nelson & Chao 2003).

Reliable design of foundations on expansive soils should include analyses of expected heave and consequences associated with the foundation movement over the design life of the structure. The heave prediction or estimation techniques in unsaturated expansive soils have been studied both using the 3-D (for example, Vu & Fredlund 2004; Masia et al. 2004; Wray et al. 2005) and the 1-D analysis (for example, Johnson & Snethen 1978, Fredlund 1983, Hamberg & Nelson 1984, Mitchell & Avalle 1984, Vanapalli et al. 2010a, b). The 1-D heave is more commonly used in the estimation, prediction and measurement in engineering practice applications. Most of the available methods in the literature for the prediction of heave generally make use of the linear relationship between void ratio (vertical strain) and the net normal stress or soil suction (Fredlund 1983, Dhowian 1990 and Nelson et al. 2006).

Several millions of dollars have been invested towards collecting valuable field data to estimate heave in natural expansive soils (for example, McKeen 1980; Snethen 1980;
Fredlund 1983; Hamberg & Nelson 1984; Dhowian 1990; McKeen 1992; Nelson & Miller 1992; Fityus & Smith 1998). This paper provides a comprehensive summary of expansive soil behavior in general and 1-D heave prediction and estimation methods in particular that were developed during the last 60 years and published in the literature.

2. CONSTITUTIVE RELATIONSHIPS FOR VOLUME CHANGE BEHAVIOR

Several constitutive relationships are available in the literature using the mechanics of unsaturated soils to interpret the volume change behavior by using stress state variables such as net normal stress, \((\sigma - u)\) and matric suction, \((u_a - u_w)\) (Matyas & Radhakrishna 1968, Alonso et al. 1990, Fredlund & Rahardjo 1993). Figure 1 shows these constitutive relationships which can be visualized in the form of volume-mass constitutive surfaces on three-dimensional plots. The constitutive surface can also be shown by plotting the volume-mass parameters versus the logarithm of the stress state variables (Chao 2007). These relationships are summarized below.

The basic constitutive relationships can be written in three different forms; which include the elasticity parameters, the compressibility parameters and the volume-mass form (Fredlund & Rahardjo 1993). These relationships are summarized below.

2.1 Elasticity form

The elasticity form of the volume change constitutive relationships for an unsaturated soil (Fredlund & Rahardjo 1993) can be formulated as an extension of the equations conventionally used for saturated soils. The soil is assumed to behave as an isotropic, linear elastic material and the form for soil structure is in below:

\[
d\varepsilon = \frac{3}{E_w} d(\sigma_{mean} - u_a) + \frac{3}{H_w} d(u_a - u_w)
\]

where, \(\sigma_{mean} = (\sigma_x + \sigma_y + \sigma_z) / 3\) is the average total normal stress; \(\sigma_x\) is total normal stress in the \(x\)-direction; \(\sigma_y\) is total normal stress in the \(y\)-direction; \(\sigma_z\) is total normal stress in the \(z\)-direction; \(\varepsilon\) is volumetric strain, \(d\varepsilon\) is volumetric strain change for each increment, \(\mu\) is Poisson’s ratio; \(E\) is modulus of elasticity or Young’s modulus for soil structure; \(H\) is modulus of elasticity for the soil structure with respect to a change in matric suction, \((u_a - u_w)\).

The water phase constitutive relationship proposed by Fredlund & Rahardjo (1993) for a complete description of the volume change behavior in an unsaturated soil can be summarized as:

\[
\frac{dV_w}{V_0} = \frac{3}{E_w} d(\sigma_{mean} - u_a) + \frac{d(u_a - u_w)}{H_w}
\]

where, \(V_w\) is volume of water; \(V_0\) is initial overall volume of an unsaturated soil element; \(E_w\) is the water volumetric modulus associated with a change in net normal stress, \((\sigma - u)\); \(H_w\) is the water volumetric modulus associated with a change in matric suction, \((u_a - u_w)\).

2.2 Compressibility form

The constitutive equations can also be written in a compressibility form which is more commonly used in conventional soil mechanics. The equation for the soil structure of an unsaturated soil to represent general three dimensional loading is given below:

\[
d\varepsilon = \frac{3}{E_w} d(\sigma_{mean} - u_a) + m_1^e d(u_a - u_w)
\]

where, \(m_1^e = 3(1 - 2\mu) / E\) is the coefficient of compressibility with respect to net normal stress; \(m_2^e = 3/H\) is the coefficient of compressibility with respect to matric suction.

The compressibility form for the water phase in terms of stress state variables \((\sigma - u)\) and \((u_a - u_w)\) is:

\[
\frac{dV_w}{V_0} = m_1^w d(\sigma_{mean} - u_a) + m_2^w d(u_a - u_w)
\]

where, \(m_2^w = 3/E_w\) is the coefficient of water volume change with respect to net normal stress; \(m_1^w = 3/H_w\) is the coefficient of water volume change with respect to matric suction.

2.3 Volume-mass form

The volume-mass form for the constitutive relationships for an unsaturated soil involves the use of void ratio, \(e\), water content, \(w\), and/or degree of saturation, \(S\). Since the void ratio change can be independent of the water content change for an unsaturated soil, the volume change constitutive equation for three-dimensional loading is written in terms of void ratio as below (Fredlund & Rahardjo 1993):

\[
d\varepsilon = a_1 d(\sigma_{mean} - u_a) + a_m d(\sigma - u)
\]

where, \(a_1\) is the coefficient of compressibility with respect to a change in net normal stress, \(d(\sigma - u)\); \(a_m\) is the coefficient of compressibility with respect to a change in matric suction, \(d(u_a - u_w)\) (Figure 1).

For a complete volume-mass characterization, a second constitutive relationship is required. Such a relationship can
be written as a water content relationship for three-dimensional loading as below:

\[ dw = b_t d (\sigma_{\text{mean}} - u_a) + b_m d (u_a - u_w) \]  

(6)

where, \( b_t \) is the coefficient of water content change with respect to a change in net normal stress, \( d(\sigma - u_a) \); \( b_m \) is the coefficient of water content with respect to a change in matric suction, \( d(u_a - u_w) \).

The above relationships are rigorous and valuable in the interpretation of the expansive soils; however, experimental determination of such constitutive relationships are both time consuming and difficult to develop. Therefore, the focus of research by several investigators from all over the world has been directed towards developing simple 1-D heave prediction methods.

### 3. HEAVE PREDICTION METHODS

Lightly loaded structures such as pavements and residential buildings constructed on expansive soils as discussed earlier are often subjected to severe distress subsequent to construction over a period of time as a result of changes in natural water content and associated pore-water pressures (i.e., suction) in the soil. The most commonly damaged structures are near ground surface structures such as roadways, airport runways, pipe lines, small buildings, irrigation canals and spillway structures. This section provides a brief background of the heave prediction methodologies commonly used in engineering practice.

Heave prediction methods were first developed in the late 1950s, and originated as an extension of methods used to estimate volume change due to settlement in saturated soils using results of one-dimensional oedometer (consolidation) tests (Chao 2007). Heave prediction methodologies have been refined continuously as knowledge and understanding of unsaturated soil behavior has increased.

Taylor (1948) proposed a mathematical model describing settlement of a layer of saturated expansive soil. The term “swelling pressure” was coined and defined by Palit (1953), as the pressure in an oedometer test required to prevent a soil sample from swelling after being saturated. Jennings & Knight (1957) first proposed the extension of settlement theory to heave prediction using oedometer tests. Salas & Serratosa (1957) presented the oedometer heave prediction model in terms of the logarithmic pressure, and incorporated the “swelling pressure” of a soil into the equation. Their equation was of the same form as that presented by Taylor (1948).

Aitchison (1973) was one of the earliest pioneers to propose a method for calculating moisture-related ground movements taking account of change in pore-water pressure (i.e., suction model). Fredlund et al. (1980) was probably the first of the investigators to provide a theoretical framework to include soil suction into the prediction of heave in expansive soils.

There are several techniques or procedures used in geotechnical engineering practice to estimate the swelling pressure, swell potential and the 1-D heave in expansive soils. These techniques can be divided into three main categories: (i) empirical methods; (ii) oedometer test methods; and (iii) soil suction methods.

#### 3.1 Empirical methods

Empirical methods use soil classification parameters to predict the swelling behavior of expansive soils. These methods are developed based on the limited data collected on local
soils or soils from a certain region. Table 1 summarizes the list of several empirical methods from the literature.

### 3.2 Oedometer test methods

Prediction methods based on oedometer tests are more widely used when compared to the other two methods. The swelling pressure determined from oedometer test methods is one of the key parameters used in the determination of the 1-D heave. Table 2 and Table 3 summarize these methods.

Depending on the loading procedure, several methods are developed such as the free swell tests (FSV), the overburden swell tests, and the constant volume swell (CVS) tests using conventional oedometer test methods. The analysis of the 1-D oedometer tests should also take into account the loading and unloading sequence, surcharge pressure, sample disturbance and apparatus compressibility for reliable determination of the swelling pressure. In many of the conventional consolidometer test procedures only the total stress is controlled.

The evolution of heave prediction methodologies using oedometer tests has been largely related to determination of the index parameters (i.e., swelling index, \( C_s \); heave index \( C_h \) or \( C_{sh} \)) and use them in the heave prediction equations. Burland (1962) first proposed using the slope of the rebound portion of the consolidation swell curve for estimation of heave. Fredlund (1983) indicated that the slope of the unloading curve from consolidation-swell tests is approximately the same as the slope of the rebound curve determined from the constant volume tests.

Fredlund (1983) method and Nelson & Miller (1992) method used oedometer test results from both the consolidation-swell test and the constant volume test to determine the index parameters (i.e., swelling index, \( C_s \) and heave index, \( C_h \)). Nelson & Miller (1992) method (Equation (21)) uses the same equation as Fredlund (1983) method (Equation (19)). Feng et al. (1998) presented a comprehensive comparison study of swell pressure from different oedometer test methods. Nelson et al. (1998) and Bonner (1998) presented a method of estimating the index parameter, \( C_{sh} \) using test results from only consolidation-swell tests. Nelson et al. (2006) refined the analysis and developed the methodology for determining the percent swell as a function of the inundation pressure.

#### 3.3 Fredlund (1983) method:

Fredlund (1983) proposed an equation that can be used to calculate the 1-D heave in expansive soils using the constant volume swell (CVS) oedometer test results (see Equation (20) in Table 3).

The CVS test procedure involves inundating the sample in the oedometer while seated under a nominal load, which is typically 7 kPa (i.e., 1 psi). The load on the sample was increased to prevent any volume increase or swelling of the sample. The maximum applied stress required for maintaining constant volume condition was defined as the swell pressure. When the specimen no longer exhibits a tendency to swell, the applied load is further increased in a series of increments in a manner similar to that of a conventional consolidation test. Once the recompression branch of the consolidation curve has been established, the sample was allowed to rebound by complete load removal in order to establish the swelling index, \( C_s \).

The test has two main measurements; namely, corrected swelling pressure, \( P_s' \), and the swelling index, \( C_s \). The unsaturated undisturbed soil specimen collected from in-situ condition is subjected to overburden stress (total stress). The total stress plane is a combination of total stress and matric suction that provides an indication of the initial stress state in the soil specimen. The volume change with stress state can be computed by the change in stress state and the swelling index, \( C_s \).

The matric suction is brought to zero during inundation, but is not measured prior to this condition. However, the total stress on the specimen is increased to keep the specimen from increasing in volume. The swelling pressure represents the sum of the in-situ overburden stress and the matric suction of the soil translated into the total stress plane (Figure 2). Therefore, the swelling pressure is dependent on the in-situ soil suction.

The measured swelling pressure is underestimated unless the effect of “sampling disturbance” and “apparatus compressibility” are taken into account (Fredlund 1969). The interpretation of the CVS test must include a correction for the compressibility of the consolidation apparatus, the compressibility of filter paper, and the seating of the porous stones and the soil specimen. Sample disturbance will result in a measured swelling pressure that is lower than the in-situ value. Therefore, to determine the corrected swelling pressure, \( P_s' \), the laboratory data should be corrected to account for the compressibility of the apparatus. The correction of sampling disturbance is also used in order to establish \( P_s' \). The “uncorrected” swelling pressure is typically low to use in the prediction of total heave. Predictions using “corrected” swelling pressures may often be twice the magnitude of those computed when no correction is applied (Fredlund 1983).

The following procedure is suggested for obtaining \( P_s' \) from CVS test results. When interpreting the laboratory data, an adjustment should be made to the data in order to account for the compressibility of the oedometer apparatus. Desiccated, swelling soils have a low compressibility, and the
<table>
<thead>
<tr>
<th>Author</th>
<th>Equation/Description</th>
<th>Eqn.</th>
</tr>
</thead>
</table>
| Seed et al. (1962)            | $SP = 0.00216I_p^{2.44}$  
$SP = $ swelling potential, %;  
$I_p =$ plasticity index.                                                   | (7)  |
| Van der Merve (1964)          | $\Delta H = Fe^{-0.377D}(e^{-0.377H}-1}$  
$H =$ volume change;  
$\Delta H =$ total heave;  
$F =$ correction factor for degree of expansiveness;  
$D =$ the thickness of non expansive layer;                       | (8)  |
| Ranganathan & Satyanarayana (1965) | $SP = 0.000413I_s^{2.67}$  
$I_s =$ shrinkage index, $(LL-SL)$.  
$LL =$ liquid limit  
$SL =$ shrinkage limit                                                  | (9)  |
| Nayak & Christensen (1971)    | $SP = 0.00229I_s^{2.67}(1.45c)/w_i + 6.38$  
$P_s(pi) = [(3.58 \cdot 10^2) I_p^{1.12} c^2/w_i^2] + 3.79$  
$w_i =$ initial water content;  
$P_s =$ the swelling pressure;  
$c =$ clay content.                                                   | (10) |
| Vijayvergiva & Ghazzaly (1973)| $SP = 1/12 (0.4LL - w_i + 5.55)$  
$log SP = 0.0526\gamma_d + 0.033LL - 6.8$  
$LL =$ liquid limit.                                                   | (12) |
| Schneider & Poor (1974)       | $log SP = 0.9(I/p_w) - 1.19$  
$P'_s =$ corrected swelling pressure;                                                                               | (13) |
| Chen (1975)                   | $SP = 0.2558e^{0.0381I_s}$                                                                                       | (14) |
| Weston (1980)                 | $SP = 0.0411LLw^{4.17}\sigma_y^{-3.86}w_i^{-2.33}$  
$LLw =$ weighted liquid limit                                                      | (15) |
| Picornell & Lytton (1984)     | $\Delta H = \sum f_i(V/V_i)H$  
$H =$ the stratum thickness;  
$\Delta V_i/$ = volumetric stain.  
$F =$ factor to include the effects of the lateral confinement;                            | (17) |
| Dhowian (1990)                | $\Delta H = (SP\%)H$                                                                                       | (18) |
| Vanapalli et al. (2010a)      | $\Delta H = C_s H + e_0 \log \left\{ \frac{KP'_s}{10C_w} \right\}$  
$H =$ thickness of the soil layer;  
$P'_s =$ corrected swelling pressure;  
$K =$ correction parameter;  
$C_s =$ swelling index;  
$\sigma'_y =$ total overburden pressure;  
$\Delta \sigma'_y =$ change in total stress;  
$u_w =$ final pore-water pressure;  
$e_0 =$ initial void ratio.  
$C_w =$ suction modulus ratio;  
$\Delta w =$ change in water content.                            | (19) |
Table 2. Oedometer tests used for heave prediction (modified and expanded after the original contribution of Nelson & Miller 1992)

<table>
<thead>
<tr>
<th>Tests</th>
<th>Location</th>
<th>Description</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Double oedometer method</td>
<td>South Africa</td>
<td>Two tests performed on adjacent samples; a consolidation-swell test under a small surcharge pressure and a consolidation test, performed in the conventional manner but at natural moisture content. Analysis accounts for sample disturbance and allows simulation of various loading conditions and final pore-water pressures.</td>
<td>Jennings &amp; Knight (1957)</td>
</tr>
<tr>
<td>Volumenometer method</td>
<td>South Africa</td>
<td>Uses specialized apparatus, air-dried samples were inundated slowly under overburden pressure.</td>
<td>DeBruijn (1961)</td>
</tr>
<tr>
<td>Sampson, Schuster &amp; Budge method</td>
<td>Colorado, USA</td>
<td>Two tests performed on adjacent samples to simulate highway cut conditions; a consolidation-swell test under overburden surcharge, and constant volume-rebound upon load removal test.</td>
<td>Sampson et al. (1965)</td>
</tr>
<tr>
<td>Noble method</td>
<td>Canada</td>
<td>Consolidation-swell tests of remolded and undisturbed samples at various surcharge loads to develop empirical relationships for Canadian prairie clays.</td>
<td>Noble (1966)</td>
</tr>
<tr>
<td>Sullivan and McClelland method</td>
<td>USA</td>
<td>Constant volume test, samples initially under overburden pressure on inundation.</td>
<td>Sullivan &amp; McClelland (1969)</td>
</tr>
<tr>
<td>Komornik, Wiseman &amp; Ben-Yacob method</td>
<td>Israel</td>
<td>Constant volume tests at various depths and swell-consolidation tests at various initial surcharge pressures representing overburden plus equilibrium pore water suction, used to develop swell versus depth curves.</td>
<td>Komornik et al. (1969)</td>
</tr>
<tr>
<td>Navy method</td>
<td>USA</td>
<td>Swell versus depth curves determined by consolidation-swell tests at various surcharge pressures representing overburden plus structural loads.</td>
<td>Navy (1971)</td>
</tr>
<tr>
<td>Wong &amp; Yong Method</td>
<td>England</td>
<td>Swell versus depth is determined as in Komornik, Wiseman &amp; Ben-Yacob method and Navy method; but surcharge loads of overburden plus hydrostatic pore water pressures are used.</td>
<td>Wong &amp; Yong (1973)</td>
</tr>
<tr>
<td>USBR method</td>
<td>U.S.A.</td>
<td>Double sample test, a consolidation-swell under light load, and a constant volume test.</td>
<td>Gibbs (1973)</td>
</tr>
<tr>
<td>Direct model method</td>
<td>Texas, USA</td>
<td>Consolidation-swell tests on samples inundated at overburden or end-of-construction surcharge loads.</td>
<td>Smith (1973)</td>
</tr>
<tr>
<td>Simple Oedometer</td>
<td>South Africa</td>
<td>Improved from double oedometer test. Single sample loaded to overburden, then unloaded to constant seating load, inundated and allowed to swell, followed by usual consolidation procedure.</td>
<td>Jennings et al. (1973)</td>
</tr>
<tr>
<td>Mississippi State Highway Dept. method</td>
<td>Mississippi, USA</td>
<td>Consolidation-swell tests on remolded or undisturbed samples inundated at overburden surcharge loads.</td>
<td>Teng et al. (1972; 1973)</td>
</tr>
<tr>
<td>Controlled strain test</td>
<td>Colorado, USA</td>
<td>Constant volume swell pressure obtained on inundation followed by incremental, strain-controlled pressure reduction.</td>
<td>Porter &amp; Nelson (1980)</td>
</tr>
<tr>
<td>University of Saskatchewan</td>
<td>Canada</td>
<td>Constant volume test. Procedure includes sample disturbance and apparatus deflection.</td>
<td>Fredlund et al. (1980)</td>
</tr>
<tr>
<td>Sridharan, Rao &amp; Sivapullaiah method</td>
<td>India</td>
<td>Tests results from three methods, namely, a) conventional consolidation tests, b) equilibrium void ratios for different consolidation loads, and c) constant volume method are combined to study the swelling pressure of expansive soils. Results show that method a) gives a upper bound value; method b) gives the least value; and method c) gives the intermediate value.</td>
<td>Sridharan et al. 1986</td>
</tr>
<tr>
<td>Erol, Dhowian &amp; Youssef method</td>
<td>Saudi Arabia</td>
<td>Assessment of the various oedometer test methods of ISO (improved swell oedometer test), CVS (constant volume swell test) and SO (swell overburden test) is used for heave prediction.</td>
<td>Erol et al. (1987)</td>
</tr>
<tr>
<td>Shanker, Ratnam &amp; Rao method</td>
<td>India</td>
<td>Studying the multi-dimensional swell behavior by testing cubic soil samples in oedometers. Swelling of samples is allowed to occur in 1-, 2- or 3-dimensions under a token surcharge.</td>
<td>Shanker et al. (1987)</td>
</tr>
<tr>
<td>Al-Shamrani &amp; Al-Mhaidib method</td>
<td>Saudi Arabia</td>
<td>The stress path triaxial cell and oedometer are used to evaluate the vertical swell of expansive soils under multi-dimensional loading conditions. Several series of triaxial swell tests were conducted in which the influence of confinement on the predicted vertical swell was evaluated.</td>
<td>Al-Shamrani &amp; Al-Mhaidib (1999)</td>
</tr>
<tr>
<td>Basma, Al-Homoud &amp; Malkawi method</td>
<td>Jordan</td>
<td>Two commonly used methods, zero swell test and the swell-consolidation test; and two relatively new techniques, “restrained swell test” and “double oedometer swell test” are using to study the swell pressure of the expansive soil. The restrained swell test is believed to give more reasonable results for swell pressure determination and thus is considered to more closely resemble field conditions.</td>
<td>Basma, et al. (2000)</td>
</tr>
<tr>
<td>Subba Rao &amp; Tripathy method</td>
<td>India</td>
<td>One-dimensional oedometer is used to study the swell-shrinkage behavior of the compacted expansive soils. The compression-rebound tests were conducted on aged and un-aged compacted specimens by incrementally loading them up to a certain surcharge and then unloading. The cyclic swell-shrinkage tests were carried out in fixed ring oedometers with the facility for shrinking the specimens at fixed temperature under constant surcharge pressure.</td>
<td>Subba Rao &amp; Tripathy (2003)</td>
</tr>
</tbody>
</table>
compressibility of the apparatus can significantly affect the evaluation of \textit{in situ} stresses and the slope of the rebound curve (Fredlund 1969). Because of the low compressibility of the soil, the compressibility of the apparatus should be measured using a steel plug substituted for the soil specimen. The measured deflections should be subtracted from the deflections measured when testing the soil. The adjusted void ratio versus pressure curve can be sketched by drawing a horizontal line from the initial void ratio, which curves downward and joins the recompression curve adjusted for the compressibility of the apparatus (Fredlund & Rahardjo 1993). Second, a correction can be applied for sampling disturbance after determining the equipment compressibility. Sampling disturbance increases the compressibility of the soil, and does not permit the laboratory specimen to return to its \textit{in situ} state of stress as its \textit{in situ} void ratio. More detailed testing procedures of this technique are available in ASTM D4546-2000.

Casagrande (1936) proposed an empirical construction on the laboratory curve to account for the effect of sampling disturbance when assessing the preconsolidation pressure of a soil. A modification of Casagrande’s construction was extended for determining $P_{s}$. In this method, the point of maximum curvature where the void ratio versus pressure

| Table 3. Summary of oedometer tests methods |

<table>
<thead>
<tr>
<th>Author</th>
<th>Equation/Description</th>
<th>Eqn.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fredlund (1983)</td>
<td>$\Delta H = C_{s} \frac{H}{1 + e_{0}} \log \frac{P_{f}}{P'_{s}}$</td>
<td>(20)</td>
</tr>
<tr>
<td></td>
<td>$H_{i}$ = thickness of the $i_{th}$ layer;</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$P_{f} = \sigma_{y} + \Delta \sigma_{y} - u_{wf}$ = final stress state;</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$P'_{s}$ = corrected swelling pressure;</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$C_{s}$ = swelling index;</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\sigma_{y}$ = total overburden pressure;</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\Delta \sigma_{y}$ = change in total stress;</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$u_{wf} = $ final pore-water pressure;</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$e_{0} = $ initial void ratio.</td>
<td></td>
</tr>
<tr>
<td>Dhowian (1990)</td>
<td>$\Delta H = H \frac{C_{s}}{1 + e_{0}} \log \frac{P}{P'_{0}}$</td>
<td>(21)</td>
</tr>
<tr>
<td></td>
<td>$C_{s}$ = swell index;</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$P$ = swelling pressure;</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$P'_{0}$ = effective overburden pressure.</td>
<td></td>
</tr>
<tr>
<td>Nelson &amp; Miller (1992)</td>
<td>$\Delta H = H \frac{C_{p}}{1 + e_{0}} \log \left( \frac{\sigma'<em>{f}}{\sigma'</em>{cv}} \right)$</td>
<td>(22)</td>
</tr>
<tr>
<td></td>
<td>$C_{p}$ = heave index;</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\sigma'_{f}$ = swelling pressure from constant volume swell test;</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\sigma'_{cv}$ = vertical stress at the midpoint of the soil layer for the conditions under which heave is being computed.</td>
<td></td>
</tr>
<tr>
<td>Nelson et al., (2006)</td>
<td>$\Delta H = H \frac{C_{H}}{\log \frac{\sigma'<em>{cv}}{\sigma'</em>{vo}}}$</td>
<td>(23)</td>
</tr>
<tr>
<td></td>
<td>$C_{H} = \frac{%S_{A}}{\log \frac{\sigma'<em>{cv}}{(\sigma')</em>{a}}}$</td>
<td>(24)</td>
</tr>
<tr>
<td></td>
<td>$C_{H}$ = heave index;</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\sigma'_{cv}$ = swelling pressure from constant volume swell test;</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\sigma'_{vo}$ = vertical stress at the midpoint of the soil layer for the conditions under which heave is being computed.</td>
<td></td>
</tr>
</tbody>
</table>
curve bends downward onto the recompression branch is determined (Figure 3). At the point of the maximum curvature, a horizontal line and a tangential line are drawn. The “corrected” swelling pressure is designated as the intersection of the bisector of the angle formed by these lines and a line parallel to the slope of the rebound curve which is placed in a position tangent to the loading curve (Fredlund & Rahardjo 1993).

Fredlund (1983) method has been tested to predict 1-D heave on Regina soil. This method provided a good agreement between the estimated heave and the in-situ measured heave. More details of this case study are available in Yoshida et al. (1983) and Fredlund & Rahardjo (1993).

3.4 Soil suction methods

The swelling pressure and the 1-D heave in expansive soils can be more reliably measured or calculated using soil suction methods as they are based on the information of the stress state (i.e., suction). In these methods, the influence of suction is taken into account through the use of different parameters. Several heave prediction formulations based on soil suction methods proposed by various researchers are summarized in this section.

3.5 Hamberg & Nelson (1984) methods

Hamberg & Nelson (1984) presented an approach (Equation (41), see Table 3) that can be used to predict heave of expansive soils using the relationship between water content and volume change in the range of shrinkage limit to liquid limit. The parameter, \( C_w \) in Equation (42) represents the variation of volume of soil specimens with respect to water content (see Figure 4) and can be obtained using the Clod test which is the modified form of COLE (coefficient of linear extensibility; Brasher et al. 1966) test. The COLE test is originally developed to determine the heave beneath airfield pavements (McKeen 1981, McKeen & Hamberg 1981).
The test procedure involves coating soil samples with a liquid resin (i.e., DOW Saran F310) that allows for volume measurements at different moisture conditions (Nelson & Miller 1992). Once the resin dries on the soil sample, it acts as a flexible membrane, containing the soil material with its natural soil fabric intact. The resin is essentially waterproof when exposed to liquid water for a short time, but it permits gradual water vapor flow to and from the sample. The volume of a soil sample of any shape may be determined by weighing the soil clod while it is submerged underwater on a balance. The reading of the balance, adjusted for the weight of the pan and water, is a direct measurement of buoyant force on the sample. Sample volume can then be determined by Archimedes’ principle.

In the original COLE procedure, each resin-coated sample was brought to 33 kPa suction in a pressure plate device. The sample volume was determined at the initial adjusted moisture condition using the weighing procedure as described above. The samples were then oven dried for 48 hours, followed by another volume determination (Nelson & Miller 1992).

A COLE value was defined as the normal strain that occurs from the moist to the dry condition, defined with reference to the dry dimension, as Equation (55) (Grossman et al. 1968).

\[
COLE = \frac{L_M - L_D}{L_D} = \frac{L_M}{L} - 1 = \left(\frac{\gamma_dM}{\gamma_dD}\right)^{0.33} - 1 \quad (55)
\]

where, \(L_M\) is the length of moist sample at 33 kPa suction; \(L_D\) is the length of oven dried sample; \(\gamma_dM\) is the dry density of moist sample at 33 kPa suction; and \(\gamma_dD\) is the dry density of oven dried sample.

The basic difference between the Clod test and the COLE test procedures is that in the Clod test, volume changes are monitored along a gradually varying moisture change path. This results in a smooth shrinkage or swelling curve for each sample (Nelson & Miller 1992). The basic Clod test procedure to develop a shrinkage curve is as follows: (i) coat a sample with resin according to the COLE procedure specifications and measure its volume; (ii) allow sample to dry slowly in air; (iii) when the sample reaches a constant weight under constant humidity conditions in a laboratory, volume and weight measurements are taken; (iv) samples are oven dried for 48 hours and final volume and weight measurements are made.

The tests data provide void ratio and water content at various points. Void ratio and water content are both directly related to soil suction. This relationship is valid only at water contents greater than the shrinkage limit; below the shrinkage limit, changes in water content are not accompanied by changes in volume, by definition (Hamberg 1985). For silty clay soils, the \(C_u\) versus water content relationship shows linear behavior for the water content greater than shrinkage limit (Hamberg 1985; Figure 4).

The limitations of Equation (41) are: (i) the equation does not take into account the effect of applied load; and (ii) there are difficulties in estimating the \(C_u\) values for the water contents close to shrinkage limit due to the nonlinearity in that range. Table 4 lists the most commonly used soil suction methods.

4. DETAILS OF HEAVE PREDICTION TECHNIQUES

Early swelling behavior studies were based on limited studies that focused on developing empirical relationships useful for explaining the local expansive soil behavior. Presently, oedometer tests methods (Table 3) and soil suction methods (Table 4) are more commonly used in engineering practice. Fredlund (1983) method and Hamberg & Nelson (1984) methods were discussed in greater detail in the earlier section. This section briefly provides the details of other heave prediction methodologies in the literature.

4.1 Aitchison (1973) method

This is probably the one of the earliest methods published in the literature that uses soil suction as a tool in the estimation of soil suction heave (Jaksa et al. 1997). This method was originally proposed for estimating heave in residential foundations built on expansive soils. The surface movement, \(\Delta H\) can be estimated using Equation (25) (see Table 4).

The ratio of vertical strain to suction change, \(\Delta u\) was experimentally observed and defined as instability index, \(I_n\) by Aitchison & Woodburn (1969), Aitchison (1970), and Lytton & Woodburn (1973). This index is equivalent to the suction index, \(C_i\) used by Snethen (1980) and Johnson (1979), or suction compression index, \(y_s\) used by McKeen & Hamberg (1981). The Australian Standard for the design and construction of residential slabs and footing, AS 2870-1996 (Standards Australia 1996), specifies three methods for the estimation of instability index, \(I_n\):

1. Laboratory tests. Three such tests are suggested; the shrink-swell test, AS 1289.7.1.1 – 1992; the loaded shrinkage test, AS 1289.7.1.2 – 1992; and the core shrinkage test, AS 1289.7.1.3 – 1992 (Standards Australia 1992);
2. Correlations between the instability index, \(I_n\) and other clay index tests;
Table 4. Summary of soil suction methods.

<table>
<thead>
<tr>
<th>Author</th>
<th>Equation/Description</th>
<th>Eqn.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aitchison (1973)</td>
<td>[ \Delta H = \frac{1}{100} \int_{z}^{\infty} I_n \Delta u \Delta h ] \hspace{1cm} \Delta H = \text{surface movement}; \hspace{1cm} I_n = \text{instability index of the soil}; \hspace{1cm} \Delta u = \text{change in suction, in pF units, at depth } z \hspace{1cm} z \text{ below the ground surface}; \hspace{1cm} \Delta h = \text{thickness of the soil layer under consideration}; \hspace{1cm} H_s = \text{depth of the design suction change} (25)</td>
<td></td>
</tr>
<tr>
<td>Lytton (1977)</td>
<td>[ \Delta H = \gamma h H \log_{10}(h_f / h_i) - \gamma \sigma H \log_{10}(\sigma_f / \sigma_i) ] \hspace{1cm} \hspace{1cm} h_f, h_i = \text{final and initial water potentials}; \hspace{1cm} \sigma_f = \text{applied octahedral normal stress}; \hspace{1cm} \sigma_i = \text{is the octahedral normal stress above which overburden pressure restricts volumetric expansion}; \hspace{1cm} \gamma_h, \gamma_\sigma = \text{two constants characteristic of the soil} (26)</td>
<td></td>
</tr>
<tr>
<td>Johnson &amp; Snethen (1978)</td>
<td>[ \Delta H = H \frac{C_r}{1 + e_0} \log \frac{h_0}{h_f + a \sigma_f} ] \hspace{1cm} \hspace{1cm} C_r = \alpha G_s / (100 B) \hspace{1cm} \log h_0 = A - B w_0 \hspace{1cm} (27) \hspace{1cm} (28) \hspace{1cm} (29)</td>
<td></td>
</tr>
<tr>
<td>Snethen (1980)</td>
<td>[ \Delta H = H \frac{C_r}{1 + e_0} (A - B w_0) - \log (\tau_{mf} + \alpha \sigma_f) ] \hspace{1cm} \hspace{1cm} C_r = \alpha G_s / 100 B \hspace{1cm} \log \tau_m = A - B w_0 \hspace{1cm} (30) \hspace{1cm} (31) \hspace{1cm} (32)</td>
<td></td>
</tr>
</tbody>
</table>

\( C_r \) = suction index; \( \alpha \) = compressibility factor; \( e_0 \) = initial void ratio; \( h_0 \) = matric suction without surcharge pressure, kPa; \( h_f \) = final matric suction, kPa; \( \sigma_f \) = final applied pressure, (overburden + external load), kPa; \( h_f \) = final matric suction, kPa; \( \sigma_f \) = final applied pressure (overburden + external load); \( \alpha \) = compressibility factor; \( A, B = \text{constants (y-intercept and slope of soil suction versus water content curve, respectively).} \)
### Table 4. (continued) Summary of soil suction methods.

<table>
<thead>
<tr>
<th>Author</th>
<th>Equation/Description</th>
<th>Eqn.</th>
</tr>
</thead>
<tbody>
<tr>
<td>McKeen (1992)</td>
<td>[ \Delta H = - \gamma_h H \log \frac{h_f}{h_i} ]</td>
<td>(33)</td>
</tr>
<tr>
<td></td>
<td>[ \gamma_h = - \frac{\Delta \nu/v_i}{\log_{10} h_f/h_i} ]</td>
<td>(34)</td>
</tr>
<tr>
<td></td>
<td>[ \Delta H = H \cdot C_h \log \Delta \tau f s ]</td>
<td>(35)</td>
</tr>
<tr>
<td></td>
<td>[ C_h = (-0.02673)(d_i/d_w) - 0.38704 ]</td>
<td>(36)</td>
</tr>
<tr>
<td></td>
<td>[ f = (1 + K_0)/3 ]</td>
<td>(37)</td>
</tr>
<tr>
<td></td>
<td>[ s = (1 - 0.01(%SP)) ]</td>
<td>(38)</td>
</tr>
<tr>
<td></td>
<td><em>\gamma_h</em> = suction compression index;</td>
<td></td>
</tr>
<tr>
<td></td>
<td><em>h_f, h_i</em> = final and initial weighted suction, respectively.</td>
<td></td>
</tr>
<tr>
<td></td>
<td><em>\Delta \nu/v_i</em> = volume change with respect to initial volume.</td>
<td></td>
</tr>
<tr>
<td></td>
<td><em>C_h</em> = suction compression index, is the slope of volume change-</td>
<td></td>
</tr>
<tr>
<td></td>
<td>soil suction curve;</td>
<td></td>
</tr>
<tr>
<td></td>
<td><em>\Delta \tau</em> = the suction change in pF;</td>
<td></td>
</tr>
<tr>
<td></td>
<td><em>f</em> = the lateral restraint factor;</td>
<td></td>
</tr>
<tr>
<td></td>
<td><em>K_0</em> = the coefficient of earth pressure at rest, equal to 1;</td>
<td></td>
</tr>
<tr>
<td></td>
<td><em>s</em> = the coefficient for load effect on heave;</td>
<td></td>
</tr>
<tr>
<td></td>
<td><em>SP</em> = the percent of swell pressure applied.</td>
<td></td>
</tr>
<tr>
<td>Mitchell &amp; Avalle (1984)</td>
<td>[ \Delta H = I_{ps} \Delta u \cdot H ]</td>
<td>(39)</td>
</tr>
<tr>
<td></td>
<td>[ I_{ps} = \frac{\Delta L/L}{\Delta w} \cdot \frac{\Delta w}{\Delta u} ]</td>
<td>(40)</td>
</tr>
<tr>
<td></td>
<td><em>I_{ps}</em> = instability index;</td>
<td></td>
</tr>
<tr>
<td></td>
<td><em>\Delta u</em> = soil suction change.</td>
<td></td>
</tr>
<tr>
<td>Hamberg &amp; Nelson (1984)</td>
<td>[ \Delta H = H \cdot \frac{C_w}{1 + \varepsilon_0} \cdot \Delta w ]</td>
<td>(41)</td>
</tr>
<tr>
<td></td>
<td>[ C_w = \frac{\Delta e}{\Delta w} ]</td>
<td>(42)</td>
</tr>
<tr>
<td></td>
<td>[ \Delta H = H \cdot \frac{C_w}{1 + \varepsilon_0} \log (h) ]</td>
<td>(43)</td>
</tr>
<tr>
<td></td>
<td>[ C_h = C_w \cdot D_h ]</td>
<td>(44)</td>
</tr>
<tr>
<td></td>
<td><em>C_w</em> = suction modulus ratio;</td>
<td></td>
</tr>
<tr>
<td></td>
<td><em>\Delta w</em> = change in water content;</td>
<td></td>
</tr>
<tr>
<td></td>
<td><em>C_h</em> = suction index with respect to void ratio;</td>
<td></td>
</tr>
<tr>
<td></td>
<td><em>D_h</em> = suction index with respect to moisture content.</td>
<td></td>
</tr>
</tbody>
</table>
3. Visual-tactile identification of the soil by an engineer or engineering geologist having appropriate expertise and local experience.

The method of estimating the instability index, $I_{pt}$, has been widely used throughout the geotechnical engineering community (Mitchell 1989, Jaksa et al. 1997, Fityus & Smith 1998). The method was named as visual-manual method (Mitchell, 1989). This technique involves a visual inspection of the soil and manually moulding and kneading the soil in order to estimate its plasticity index, $I_p$ (Jaksa et al. 1997). Mitchell (1979) presented an approximate relationship between plasticity index $I_p$ and instability index, $I_{pt}$, which is often used in the visual-tactile method to estimate the instability index, $I_{pt}$. The relationship for estimating the

### Table 4. (continued) Summary of soil suction methods.

<table>
<thead>
<tr>
<th>Author</th>
<th>Equation/Description</th>
<th>Eqn.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dhowian (1990)</td>
<td>$\Delta H = H \cdot \frac{C_v}{1 + e_0} \log \frac{\psi_i}{\psi_f}$</td>
<td>(45)</td>
</tr>
<tr>
<td></td>
<td>$C_v = \frac{\alpha G}{100B}$</td>
<td>(46)</td>
</tr>
<tr>
<td></td>
<td>$\Delta H = H \cdot \frac{\alpha G}{1 + e_0} (w_f - w_i)$</td>
<td>(47)</td>
</tr>
<tr>
<td></td>
<td>$\Delta H = H \cdot C_w (w_f - w_i)$</td>
<td>(48)</td>
</tr>
<tr>
<td></td>
<td>$C_w = \alpha G_i / (1 + e_0)$</td>
<td>(49)</td>
</tr>
<tr>
<td></td>
<td>$C_v = \text{suction index; }$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\psi_i = \text{initial suction; }$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\psi_f = \text{final suction; }$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\alpha = \text{volume compressibility factor} $</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$B = \text{slope of suction versus water content relationship; }$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$G_s = \text{specific gravity of solid particles; }$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$C_w = \text{moisture index} $</td>
<td></td>
</tr>
<tr>
<td>Fityus &amp; Smith (1998)</td>
<td>$\Delta H = H \cdot I_v \cdot \alpha (w_i - w_f)$</td>
<td>(50)</td>
</tr>
<tr>
<td></td>
<td>$I_v = \text{volume index; } $</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\alpha = \text{empirical factor accounting for confining stress differences }$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\text{in lab and field; }$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$w_i, w_f = \text{average initial water content and the average final }$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\text{water content, respectively; } $</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\sigma_v = \text{vertical stress at the midpoint of layer.} $</td>
<td></td>
</tr>
<tr>
<td>Briaud et al. (2003)</td>
<td>$\Delta H = H_f (\Delta w - E_w)$</td>
<td>(52)</td>
</tr>
<tr>
<td></td>
<td>$E_w = \Delta w (\Delta V/V_0)$</td>
<td>(53)</td>
</tr>
<tr>
<td></td>
<td>$f = (\Delta H/H_0) (\Delta V/V_0)$</td>
<td>(54)</td>
</tr>
<tr>
<td></td>
<td>$E_w = \text{shrink–swell modulus, slope of the water content versus }$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\text{the volumetric strain line; }$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$f = \text{shrinkage ratio, ratio of the vertical strain to the volumetric }$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\text{strain} $</td>
<td></td>
</tr>
</tbody>
</table>
instability index, \( I_{pt} \), is detailed in the Mitchell & Avalle (1984) method (see Section 4.6).

However, by its nature, the method is highly classifier-dependent (Jaksa et al. 1997). In addition, the laboratory and in-situ tests performed to obtain the required parameters (i.e., instability index, \( I_{pt} \); soil suction change, \( \Delta u \)) for the approach are both time consuming, expensive and difficult.

### 4.2 Lytton (1977) method

The method is used for estimating the heave and rate of heave of foundations resting on expansive soil deposits. The approach is based on simple laboratory tests and some field observations of the shrinkage crack network. The ground surface heave experienced by an elemental volume of soil due to a change in water potential can be calculated using Equation (26) (see Table 4). The volumetric strain can be represented as Equation (56):

\[
\frac{\Delta V}{V_o} = \gamma_h \log_{10} \left( \frac{h_f}{h_i} \right) - \gamma_s \log_{10} \left( \frac{\sigma_f}{\sigma_i} \right)
\]

The first term (i.e., \( \gamma_h \log_{10} \left( \frac{h_f}{h_i} \right) \)) is the volumetric strain due to a change in water potential. The contribution of the second term (i.e., \( -\gamma_s \log_{10} \left( \frac{\sigma_f}{\sigma_i} \right) \)), which has the contrary sign, is only considered with increasing depth until the strains become zero (Picornell & Lytton 1984).

### 4.3 Johnson & Snethen (1978) method

Soil heave is predominantly induced by suction change within the active zone depth. When an expansive soil is wetted, its soil suction decreases and the soil volume increases. This concept was used as a tool in the estimation of unit heave which is expressed in Equation (27) (see Table 4).

The initial suction can be measured by thermocouple psychrometer or filter paper method. Soil suction measurements include preparing several undisturbed soil cubes and modifying the moisture contents of the samples to fit the range of field conditions of moisture variation (Snethen & Huang 1992). The resulting soil suction versus moisture content relationship is used to establish the required parameters. The parameters \( A \) and \( B \), and compressibility factor \( \alpha \) are determined from the plotted results of the soil suction test procedure. \( A \) is the soil suction value (logarithmic scale) at zero moisture content; \( B \) is the slope of soil suction versus moisture content curve. The compressibility index, \( \alpha \), is the slope of specific volume, \( (1+e)/G_s \), versus moisture content.
The suction index, $C_{\tau}$, reflects the rate of change of void ratio with respect to soil suction. The initial soil suction, $h_0$, is measured during suction testing and the final suction profile is assumed based on one of the four criteria suggested by Snethen (1980): (i) zero throughout the depth of active zone; (ii) linearly increasing with depth through the active zone; (iii) saturated water content profile; (iv) constant at some equilibrium value.

### 4.4 Snethen (1980) method

This method uses soil suction data (Figure 5) to characterize the expansive soil:

$$\log \tau_m = A - Bw$$

where, $\tau_m$ is the soil suction without surcharge pressure (i.e., atmospheric pressure). The slope of the line, $B$ is determined by calculating the inverse of the change in water content over one cycle of the soil suction scale. The intercept, $A$ is calculated by applying Equation (32) at soil suction equal to 95.8 kN/m². The volume change and swelling pressure of an expansive clay stratum is estimated using Equation (30) (see Table 4) and Equation (58):

$$\log P_s = A - (100Be_0/G_s)$$

The suction index, $C_{\tau}$ reflects the rate of change of void ratio with respect to soil suction. The laboratory data necessary to apply the equation include $G_s$, $e_0$, $A$, $B$, $w_0$ and $\alpha$, all of which (expect $G_s$) can be determined in the soil suction test procedure (Snethen 1980). The remaining two variables, $\tau_{mf}$ and $\sigma_f$ are functions of assumed depth of active zone and the assumed final soil suction profile. The compressibility factor, $\alpha$ for highly compressible clays (i.e., CH) is commonly set equal to one, because the voids of these soils are filled with water within a wide range of moisture contents (Snethen 1980). In the absence of measured data, the compressibility factor may be roughly estimated from the plasticity index, $I_p$, as: (i) $I_p < 5$, $\alpha = 0$; (ii) $I_p > 40$, $\alpha = 1$; (iii) 5 < $I_p$ < 40, $\alpha = 0.0275 I_p - 0.125$.

The equation provides predictions of in-situ volume change of a soil stratum with respect to field conditions of soil composition, structure, initial and equilibrium moisture profiles, and confining pressures.

### 4.5 McKeen (1980, 1992) methods

McKeen (1980, 1992) used the suction compression index $\gamma_s$ introduced by Lytton (1977) as a tool to propose a heave prediction technique as shown in Equation (34) (see Table 4).

A measure of the physical phenomena was needed to facilitate rating, classifying, and discussing the volume...
change behavior of expansive soils. This was addressed by suction compression index, $\gamma_h$, which is volume-response coefficient. The suction compression index, $\gamma_h$, can be determined using different testing methods. The required data are volume change measurement based on drying soil samples and suction change over which the volume change occurred (McKeen 1980). The data collected must cover the range of moisture suction expected in the field environment. The measurements may take place in a 1-D oedometer or a 3-D configuration (i.e., unrestrained soil clods) (McKeen 1980). One test routinely used for this purpose is the COLE (coefficient of linear extensibility) method. All procedures for determining suction compression index, $\gamma_h$, require suction measurements. For convenience, the empirical relationships between this parameter and soil activity and cation exchange activity, or percentage of clay are established by statistical studies (McKeen 1981):

For clay content between 40% and 70% and high activity:

$$\gamma_h = 0.00179C - 0.041$$

(59)

For clay content between 25% and 70% and low activity:

$$\gamma_h = 0.00057C - 0.00057$$

(60)

where, $C = \text{percent } < 2 \mu\text{m}$ (ASTM D422). Thus, the value of the compression index $\gamma_h$ can be determined and McKeen (1980) method can be expressed in Equation (33) (see Table 4) and McKeen (1992) method can be shown in Equation (35).

McKeen (1981, 1985 & 1990) proposed a classification scheme by correlating the suction potential index, $dh/dw$, and the suction compression index, $C_h$ (Figure 6) using a database on Texas soils. In this correlation, a linear relationship is suggested such that 85% of the compression index, $C_h$ value is larger than would be predicted by the relationship (Olsen et al. 2000). The required information includes suction potential, $dh/dw$, for which relatively simple suction and water content measurements are needed. This method provides a useful index for estimating the more fundamental suction compression index, $C_h$, which also requires the Clod test (Olsen et al. 2000). McKeen (1992) proposed quantitative criteria for using both the suction potential index, $dh/dw$ and suction compression index, $C_h$, values to differentiate five categories of swell potential ranging from very high to non-expansive, as shown in Table 5.

McKeen’s suction potential, $dh/dw$ and suction compression index, $C_h$ are recommended to be obtained on undisturbed clods of soil. These values are dependent not only by the type and amount of clay in a soil, but also by the structure and pore-fluid chemistry of the soil, that result from its geologic origin and history (Olsen et al. 2000).

### 4.6 Mitchell & Avalle (1984) method

A simple and relatively quick method is presented to enable the prediction of expansive soil movements from soil suction changes. This method is assumed that vertical strain of the expansive soil is linearly proportional to soil suction. Therefore, the ground heave can be expressed as Equation (39) (see Table 4).

The instability index, $I_{ph}$ is obtained from the core shrinkage test. It involves the measurement of the linear strain versus moisture content relationship, $\varepsilon_v/\Delta w$, and the
moisture characteristic, $c$ (i.e., $c = \Delta w / \Delta u$), of unconfined undisturbed core samples (Figure 7; Equation (40)). Generally, the more clayey the soil, the higher is its moisture characteristic (Morris & Gray 1976, Mitchell 1979). Several soil specimens are required for obtaining information about different initial water contents. The specimens are air dried for two days, during this period; the length and mass of the shrinkage core are measured frequently. It is then oven dried to obtain the moisture content. The moisture characteristic, $c$, can be determined directly through soil suction test using thermocouple psychrometer or the filter paper methods on companion specimens. The instability index, $I_p$, is calculated as the slope of linear dimension change versus moisture content, $\Delta \epsilon / \Delta \omega$, times the moisture characteristic, $c$ (Equation (40)).

Mitchell & Avalle (1984) shows the core shrinkage tests results of 80 samples from 18 sites in South Australia and three samples from one site in Victoria, Australia (Figure 8). Considerable scatter exists in the relationship between the instability index, $I_p$ and plasticity index, $I_p$. A simple relationship does not exist; however, $I_p$ value is higher for soils with higher $I_p$. If the plasticity of a soil is assessed in a qualitative way (i.e. visual-tactile method), then from experience with a particular geological location, a reasonable accurate value of instability index, $I_p$, may be adopted without testing (Mitchell & Avalle 1984).

### 4.7 Hamberg & Nelson (1984) method:

Heave is related to soil suction change and soil suction is dependent on the moisture content of the soil; therefore, heave may be predicted by measuring changes in moisture content. Hamberg & Nelson (1984) extended this philosophy and proposed a method to determine the heave of expansive soils using suction modulus ratio, $C_w$ (Equation (42), see Table 4). The amount of heave can be calculated using Equation (41) (see Table 4).

If the final soil suction profile is available instead of moisture content change profile $\Delta H$ can be calculated using Equation (43) (see Table 4). Actual ground heave may be adjusted depending on the confinement situation:

$$\Delta H_{act} = f \Delta H \quad (61)$$

where, $f$ is the correction factor, 0.33-1.0.

### 4.8 Dhowian (1990) method

Dhowian (1990) estimated heave of expansive shale formation based on soil suction change. The model described the field volume change where the anticipated heave is compared with direct measurement obtained from the field station. For the purpose of obtaining in-situ heave measurements, an instrumented field station was established in the central region of Saudi Arabia (Figure 9). Expansive shale predomi-
nates near the ground surface in this area. Undisturbed shale samples were obtained from the field and used for the free swell test, the constant volume test, and the swell overburden test. The swell parameters obtained from results of oedometer tests are used to predict the anticipated field heaves using Equation (21) (see Table 3). The method uses the same equation (Equation (20), see Table 3) as Fredlund (1983). In this equation, $C_s$ is the swelling index; $P_s$ is the swelling pressure; $P_0$ is the effective overburden pressure.

Amongst the oedometer tests, free swell test gave the highest swelling pressure value, swell overburden test gave the least value, and the constant volume test is in between. The swelling index, $C_s$ value from swell overburden tests was significantly higher than free swell test and constant volume test. The estimated heaves calculated from Equation (20) using tests results from three types of oedometer tests are compared with the measured field heave values. The agreement between the predictions based on constant volume test and the field heave is quite well (Al-Shamrani & Dhowian 2002) (Figure 10).

Linear relationship is observed between swell and suction (Figure 11) for most homogeneous undisturbed shale samples tested under controlled conditions. Such a trend is analogous to the $e$–log $P$ curve in the consolidation; hence, the suction method can be presented as Equation (45) (see Table 4). The equation tends to overestimate field heave considerably. The discrepancy between the predicted and measured heave may be attributed to the experimentally determined parameters volume compressibility factor, $\alpha$ and slope of suction versus water content relationship, $B$. For this purpose, the average change in soil suction and water content coupled with the average ultimate swell measured at the end of the observation period are back-calculated to obtain the parameters $\alpha$ and $B$. The values are presented in Table 6 with the laboratory data. The discrepancy between the field and laboratory value of $\alpha$ is attributed to the measurement of volumetric swell rather than the vertical swell because of the lateral restraint in the oedometer chamber. Therefore, one third of the experimentally obtained value is considered as the field value of $\alpha$ and substituted in the suction method.

The slope of the linear part (Figure 12) is defined as the compressibility factor $\alpha$, hence

$$\Delta e = \alpha G_s (w_f - w_i)$$

where, $w_i$, $w_f$ are the initial and final moisture contents, respectively. The log-suction versus swell relationship has been found out to be as

$$\Delta e = C_{\psi} \log(\psi_f / \psi_i)$$

where, $C_{\psi}$ = suction index; and $\psi_i$ and $\psi_f$ are the initial and final suction.

Thus, the relationship for $C_{\psi}$ is equal to

$$C_{\psi} = \frac{\alpha G_s (w_f - w_i)}{\log(\psi_f / \psi_i)}$$

Using the suction method, the heave prediction can be estimated as Equation (47). Equation (48) indicates that heave is linearly proportional to the change in moisture content. This equation is possible to determine the maximum swell by determining the limits of moisture content variation. It must be noted that the moisture index, $C_w$, should be determined using one third of the experimentally determined compressibility factor, $\alpha$. 

\[\text{Figure 12. Specific volume versus water content plot for clayey shale (modified after Dhowian 1990).}\]

\[\text{Figure 13. Estimation of the volume change index, } I_v \text{ for Maryland clay (modified after Fityus & Smith 1998).}\]
4.9 Nelson & Miller (1992) method

The method presented in Nelson & Miller (1992) used test results from both the consolidation-swell test and the constant volume test to determine the index parameter. The method uses the same equation (Equation (20), see Table 3) as Fredlund (1983).

“Free-field” heave is the amount of heave that the ground surface will experience due to wetting of the sub-soils with no surface load applied. Because the surface load applied by slab-on-grade floors is relatively smaller in comparison to the swell pressure generated by an expansive soil, the heave of slabs is essentially the same as the free-field heave (Chao 2007). The general equation for predicting heave using the oedometer methods can be presented as Equation (22) (see Table 3).

4.10 Fityus & Smith (1998) method

The surface movement prediction presented by Fityus and Smith (1998) integrates volume changes which occur in sub-layers of soil over the depth of the active zone to provide an estimate of the total movement at the ground surface. This method requires the information of change in gravimetric water content as a function of depth through the soil profile, as well as an appropriate index to relate moisture change directly to volume change. As the amount of volume change, \( \Delta H \) is affected by the confining stress of the overburden, a volume change index, \( I_v \) is used which takes this into account (Equation (50), see Table 4).

The load dependent volume index, \( I_v \) was determined from a series of simple, one dimensional swell tests, in which a clay sample is allowed to swell under a vertical confining stress that was equal to the calculated vertical stress experienced under field conditions. The tests were conducted on Maryland clay (Figure 13). The approach described here is based on a number of assumptions (Fityus & Smith 1998): (i) it is assumed that a given change in water content will always correspond with the same change in strain (ii) it is assumed that the volume index, \( I_v \), determined from swelling clay tests can be employed to predict both swelling and shrinking behavior (iii) it is assumed that the difference between one and three dimensional volume changes can be reasonably accommodated using a factor, \( \alpha \) of 0.33.

4.11 Briaud et al. (2003) method

Briaud et al. (2003) proposed a water content method to estimate the vertical movement of the ground surface for soil that swells and shrinks due to variations in water content, \( \Delta w \). In the field, the soil suction usually ranges from 30 to 3,000 kPa (McDowell 1956). Within that range, the soil water characteristic curve can often be approximated by a straight
The soil water content change, $\Delta w$, can be calculated by using Equation (65), where $\beta$ is the slope of the soil–water characteristic curve.

$$\Delta w = \log \frac{u_f}{u_i}$$

(65)

Thus, for each layer, $i$ the movement for swelling or shrinkage can be calculated by using Equation (52). The ground surface movement can be calculated by using Equation (66).

$$\Delta H = \sum_{i=1}^{n} \Delta H_i$$

(66)

The procedures for the method include: 1) Determine the depth $Z_{\text{max}}$ of water content fluctuation and break the depth $Z_{\text{max}}$ into an appropriate number of $n$ layers, $h_i$ being the thickness of layer $i$; 2) Collect samples at the site within depth, $Z_{\text{max}}$; 3) Perform shrink tests on the samples; for each sample, determine the shrink–swell modulus, $E_w$, which is the slope of the water content versus the volumetric strain line; and the shrinkage ratio $f$, which is the ratio of the vertical strain to the volumetric strain; 4) Determine the change in water content, $\Delta w$ as a function of depth within $Z_{\text{max}}$.

For remolded conditions, soils dryer than shrinkage limit, $w_{sh}$ are unsaturated whereas soils wetter than swell limit, $w_{sw}$ are saturated (Holtz and Kovacs 1981). For natural conditions, soils dryer than shrinkage limit, $w_{sh}$ are unsaturated and soils wetter than swell limit, $w_{sw}$ are either saturated or close to it (Fredlund et al. 2002). Thus, the water content of a soil ranges from shrinkage limit, $w_{sh}$ to swell limit, $w_{sw}$. Most of the volume change takes place between shrinkage limit $w_{sh}$ and swell limit $w_{sw}$ and the relationship is linear within that; in other words, the volume change is clearly related to the change in water content between this range. Therefore, a direct relationship can be seen between the shrink–swell modulus, $E_w$ and the suction compression index, $\gamma_s$. This water content method is similar to the McKeen (1992) soil suction method (see Equation (35), Table 4), but it makes use of the water content as the governing parameter.

The change in water content, $\Delta w$ can be obtained from various possible sources including databases. The depth of water content variation is estimated from a combination of experience, databases, observations, and calculations. For the actual water content variation, it is as a function of time, weather, vegetation and pipe leaks. There are several methods to predict it (i.e., Wray et al. 2005, Chao 2007).

The shrink tests were used for obtaining links between the water content $w$ and the normal strain, $\varepsilon = (\Delta H/H_0)$. The test procedure which involves four steps are detailed in Briaud et al. (2003): 1) Trim the sample into a cylinder (A 75 mm diameter × 150 mm high sample size is recommended with the height corresponding to the vertical direction at the

![Figure 15. Influence of vertical pressure on the shrink test results (modified after Briaud et al. 2003): (a) $w$ versus $\Delta V/V_0$ curve; (b) $w$ versus $\Delta H/H_0$ curve; (c) $w$ versus $\Delta D/D_0$ curve.](image-url)
2) The weight \( W_0 \), the height \( H_0 \), and the diameter \( D_0 \) of the sample at time \( t = 0 \) is recorded. A minimum of three heights and three diameter measurements for each height is recommended at 120° intervals. 3) The sample is allowed to air dry in a laboratory environment under constant humidity conditions. The variation of weight \( W \), the height \( H \), and the diameter \( D \) of the sample is collected as a function of time, \( t \). The data is collected every hour for the first 8 hours. It is recommended to record a minimum of three height and three diameter measurements for each height at 120° intervals at each reading. Typically, two days of data are usually sufficient. 4) After the last reading is taken, dry the sample in the oven and measure its oven-dried weight \( W_d \). Data reduction consists of calculating the average height, the average diameter, and the average volume for each time step as well as the water content of the sample.

The plots of the water content \( w \) versus the volumetric strain, \( (\Delta V/V_0) \) or the vertical normal strain, \( (\Delta H/H_0) \) are the constitutive laws, which are used in this method. The plot of \( (\Delta H/H_0) \) versus \( (\Delta V/V_0) \) gives the relationship between vertical shrinkage and volumetric shrinkage. Examples of shrink test results are shown in Figure 15. The reason is twofold: 1) most of the volume change occurs when the soil is saturated or near saturation (i.e., shrinkage limit to swell limit); 2) for the tests results presented in Figure 15, the volume change is simply due to water loss or water gain which is independent of the pressure if water and soil grains are considered incompressible (Briaud et al. 2003).

Therefore, if the soil is saturated, the shrink–swell modulus, \( E_w \), can be obtained using Equation (67); where \( \gamma_w \) is unit weight of water, and \( \gamma_d \) is dry unit weight of the soil.

\[
E_w = \frac{\gamma_w}{\gamma_d} \quad (67)
\]

While the soil is not completely saturated, Briaud et al. (2002) developed an equation linking the change in volume, \( (\Delta V/V_0) \) to the change in water content \( \Delta w \) that is valid between the shrink limit \( w_{sh} \) and the swell limit \( w_{sw} \).

\[
\Delta V/V_0 = S_n(\Delta w/w) + 0.8n(S - 1)[\Delta w/(w_{sh} - w_{sw})] \quad (68)
\]

where, \( S \) is degree of saturation; \( n \) is porosity; \( w_{sh} \) is the shrinkage limit obtained in the shrink test; and \( w_{sw} \) is the swell limit obtained in the free swell test (ASTM D4546). The shrink–swell modulus, \( E_w \), can be calculated from Equation (69):

\[
E_w = [S_n(\Delta w/w) + 0.8(S - 1)/(w_{sh} - w_{sw})]^{-1} \quad (69)
\]

The first term in Equation (69) corresponds to Equation (67) (i.e., \( \gamma_w/\gamma_d = w/S_n \)); the second term can quantify the deviation from the saturated case. Parametric calculations show that for degrees of saturation between 0.9 and 1, the \( E_w \)
A state-of-the-art review of 1-D heave prediction methods for expansive soils

Figure 18. Hypothetical oedometer test results for stress states (modified after Nelson et al. 2006).

value for the unsaturated case is within 10% of the \( E_w \) value for the saturated case (Briaud et al. 2003).

The advantages of the proposed shrink test–water content method are as follows (Briaud et al. 2003). First, it is based on the water content as a governing parameter that is simpler than suction to obtain experimentally. Second, larger databases exist for water contents than suction values; as a result obtaining the magnitude of the actual water content variation in the field and the depth of these variations from such databases is much easier than in the case of suction values. Third, the constitutive law is obtained experimentally from a simple shrink test on site-specific samples instead of correlations to index properties. Also, the constitutive law (Equation (53), see Table 4) is independent of the stress level; this is not the case for the suction constitutive law. However, if the soil is highly fractured, shrink tests are difficult to perform; in this case, the Clod tests can be substituted and used as an alternative shrink test.

4.12 Nelson et al. (2006) method

Nelson et al. (2006) describes a methodology to determine heave using oedometer test data. An important constitutive parameter used in the method is the heave index, \( C_{hv} \) which is the ratio of the percent swell observed in the oedometer test to the vertical stress applied to the sample when it was inundated (i.e., the inundation pressure).

When oedometer test data is plotted in a two-dimensional format, the entire stress path is projected onto the plane defined by the \((\sigma - u)\) and \(\varepsilon_v\) axes (Figure 16). The consolidation-swell test data follows the projected path ABCF, whereas the constant volume test data follows the projected stress path ABD. Example test data is shown as below. The slope of the loading portion of the curve shown in the figure is the compression index \( C_c \) and of the rebound portion of the curve is the rebound index \( C_r \). The volumetric strain experienced during inundation is the percent swell.

Figure 17 shows the vertical overburden stress states at three different depths in a soil profile with similar soil throughout. At all points all samples are in a condition of zero lateral strain with a vertical overburden stress equal to \( \sigma'_{vo} \). If a consolidation-swell test is conducted on a sample identical to that at depth, \( Z_A \), at an inundation stress, \( (\sigma'_{i})_A \) the sample will swell by an amount \( %S_A \) as shown in Figure 18. Similarly for a sample at depth \( Z_B \), the sample would be subjected to an inundation stress, \( (\sigma'_{i})_B = (\sigma'_{w})_B \), and the sample would swell by an amount \( %S_B \).

The general equation for predicting heave or settlement in a soil stratum of thickness, \( H \) can be shown as Equation (70):

\[
\Delta H = H \frac{\Delta e}{1 + \varepsilon_0} 
\]

(70)

For uniform vertical strain throughout the stratum the strain is equal to:

\[
\varepsilon_v = \frac{\Delta e}{1 + \varepsilon_0} 
\]

(71)

Substituting Equation (71) into Equation (70):

\[
\Delta H = H \varepsilon_v = H %S 
\]

(72)

At any depth in the soil, the percent swell will fall along the line ABC. For all practical purposes the line can be defined by a straight line connecting point A (the point defined by the percent swell in a consolidation-swell test) and point C (the point corresponding to the constant volume swell pressure, \( \sigma'_{cv} \)). The slope of that line is denoted by the heave index, \( C_{hv} \) which is shown in Equation (24).

If values of heave index, \( C_{hv} \) and constant volume swell pressure, \( \sigma'_{cv} \) are known; the vertical strain, or percent swell, that will occur during inundation at any depth \( z \) in a soil profile can be determined from Equation (23). Equation (23) can be re-written as Equation (73), when the soil at depth \( z \) is inundated, the stress on the soil is the overburden stress, \( (\sigma'_{wo})_z \). This value is therefore used for the inundation stress, \( \sigma'_{i} \).in Equation (73).

\[
\varepsilon_v = %S = C_{hv} \log \left[ \frac{\sigma'_{wo} - e_{z}}{\sigma'_{wo}} \right] 
\]

(73)

Therefore, heave prediction can be calculated using Equation (23) (see Table 4).
Nelson et al. (1998, 2006) indicated that an accurate method to determine the heave index, $CH$, would require several consolidation-swell tests at different inundation pressures and a constant volume test. This is neither practical nor economical. Therefore, the relationship between $\sigma''_{cv}$ and $\sigma''_{f}$ was proposed so that the value of the heave index, $CH$, can be determined from a single consolidation-swell test (Nelson et al. 2006). A relationship between $\sigma''_{cv}$ and $\sigma''_{cs}$ exists that is of the form:

$$\sigma''_{cv} = \sigma''_{i} + \lambda(\sigma''_{cv} - \sigma'')$$ (74)

This method for determination of the heave index is considered to be practical and rational but the actual value of $\lambda$ to be used for different soils should be investigated on a case by case basis. Generally, a value of $\lambda$ in the range of 0.5 to 0.7 may be reasonable.

### 4.13 Vanapalli et al. (2010a) method

Vanapalli et al. (2010a) proposed a simple technique by deriving a new equation based on the Fredlund (1983) and the Hamberg & Nelson (1984) methods for estimating 1-D heave. This method alleviates from the limitations of several methods. Some of the limitations of the available methods include: 1) they are not universally valid as they are proposed using only limited soils data collected locally; 2) they do not use the stress state variables approach that provides a rational basis for interpretation; 3) the various parameters required in these approaches can only be obtained from time consuming laboratory or in-situ tests that are expensive and difficult to be performed.

Vanapalli et al. (2010a) technique requires three parameters; namely, corrected swelling index, $Cs$, suction modulus ratio, $Cw$, and correction parameter, $K$ which is a function of water content change, $\Delta w$ and plasticity index, $IP$. Though the parameters $C_s$ and $C_w$ can be determined from laboratory tests, they proposed empirical equations to obtain the values of the parameters to avoid time-consuming or elaborate testing procedures.
The technique was tested on several case studies and the results showed that the proposed technique can provide reasonable 1-D heave estimations on natural expansive soils from various regions of the world (Vanapalli et al. 2010b).

Fredlund (1983) method (Equation (20), see Table 3) can be re-written as the Equation (75). The positive and the negative sign in the equation indicate compression and heave due to overburden and swelling pressure, respectively. The heave calculated by using the second term is proportional to the heave estimated using Hamberg & Nelson (1984) method (Equation (41), see Table 4), and can be shown as Equation (76).

\[
\Delta H = C_s \frac{H}{1 + e_0} \log \frac{P_s}{C_w} - C_s \frac{H}{1 + e_0} \log \frac{P'}{C_w} \quad \text{for swelling} \\
C_s \frac{H}{1 + e_0} \log \frac{P'}{C_w} C_s \frac{H}{1 + e_0} \Delta w \quad \text{for compression}
\]  

Equation (76) can be re-written as Equation (77) by introducing a correction parameter K. The Equation (19) can be obtained by substituting Equation (77) into Equation (20).

\[
P' = 10^{-\frac{\log \Delta w}{C_s}}
\]

Chen (1975) stated that the amount of swell in expansive soils is governed by the change in water content, \( \Delta w \), which can be obtained from field investigation studies. This information can be calculated using Equation (78) (Fredlund and Rahardjo, 1993) based on the assumption that the soils attain saturated (i.e., \( S_f = 100\% \)) condition as well. Such an assumption provides conservative estimations (i.e., maximum 1-D heave).

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

The \( C_w \) value can be measured from Clod tests (see section 3.3.1). 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). Using this concept, an empirical relationship between suction modulus ratio, \( C_w \), and plasticity index, \( I_p \), was developed using the data published in the literature (see Figure 19).

Though the suction modulus ratio, \( C_w \), values show large scatter for the \( I_p \) values less than 30\%, the relative constant value (i.e., 0.024) was observed for the \( I_p \) values greater than 30\%. This indicates that Equation (79) 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%.

\[
C_w = 0.024 \quad I_p \geq 30\%
\]

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 discussed earlier (see section 3.2.1). It also needs different corrections with respect to swelling pressure, compressibility of the apparatus, and sample disturbance. Hence, a simple empirical relationship between the corrected swelling index, \( C_s \), and plasticity index, \( I_p \), was developed using the data available in the literature as shown in Equation (80) and Figure 20.

\[
C_s = 0.019e^{0.0343(I_p)}
\]

Figure 21 shows the empirical relationship between correction parameter, \( K \), and \( \Delta w \) for the data from five case studies in literature. The analysis in Figure 21 showed that \( K \) is not a constant value but instead varies with \( I_p \) (Equation (81)). The best-fitting curve for the all data was estimated as Equation (82) with relatively high R-squared value (i.e. 0.997). The correction parameters, \( K \) in Equation (81) and (82) are hereafter referred to as \( K_i \) and \( K_{II} \) respectively and both were used in this technique.

\[
K_i = [-0.0018\ln(I_p) + 0.01]e^{0.64(\Delta w)}
\]

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

5. SUMMARY

Empirical relationships that were developed using limited studies for explaining the expansive soils behavior during the period from 1950’s thru 1900’s were found to be not universally valid but only useful for explaining the local expansive soil behavior. Aitchison (1973) was the earliest investigators who introduced the soil suction models introducing the concept of instability index (i.e., \( I_p \)) for calculating moisture-related ground movements. This instability index, \( I_p \), has been widely used by geotechnical researchers (Mitchell 1989, Jaksa et al. 1997, Fityus & Smith 1998).

Several other techniques have been proposed using oedometer test methods or soil suction methods for estimating the surface heave in expansive soil with lightly loaded structures (see Table 4). Some techniques are based on stress state variables approach (i.e., Snethen (1980) method, Fredlund (1983) method) and others do not use the stress state variables approach (i.e., Hamberg & Nelson (1984) method, Nelson & Miller (1992) method). However, in all these methods various parameters are required to estimate the 1-D heave by time consuming laboratory and/or in-situ tests. Most of the tests are expensive and difficult to be performed by conventional geotechnical engineers. The method proposed by Vanapalli et al. (2010a) is a much simpler and inexpensive for estimating the 1-D heave that can be used.
universally on expansive soils from all regions of the world with a reasonable degree of accuracy. This method requires only the information of plasticity index, \( I_p \), and variation in water content with respect to depth in the active zone.

**REFERENCES**


Teng, T.C.P., Mattox, R. M. & Clisby, M. B. (1972). A study of active clays as related to highway design. Research and Development Division, Mississippi State Highway Department, Engineering and Industrial Research Station, Mississippi State University, MSHD-RD-72-045, 134.


Van Der Merwe, D.H. (1964). “The prediction of heave from the plasticity index and percentage clay fraction of soils,” Civil Engineers in South Africa, 6, 337–42.


