J.King Saud Univ., Vol. 2, Eng. Sci. (2), pp. 355–377 (1410H/1990)

REVIEW PAPER

Heave Prediction Techniques and Design Consideration on Expansive Soils

Abdulmohsin W. Dhowian
College of Engineering, King Saud University,
P.O.Box 800, Riyadh 11421, Saudi Arabia

Abstract. Expansive soil has been responsible for many structural damages that result in great financial losses in many parts of the world including Saudi Arabia. Reliable prediction of ground heave is essential for the development of more effective and economical design of structures on expansive soil. Three procedures are commonly used for this purpose; oedometer test method, suction technique and empirical relationships. Available methodology for the prediction of swell potential in expansive soils and techniques proposed for quantitative analysis of heave are presented in this paper. Also, alternative soil treatment methods to minimize swell potential of expansive soils or foundation design criteria to safely accomodate ground movements are discussed herein.

Introduction

Damages to structures from swell and shrinkage of foundation soils due to changing moisture contents are a common problem in many parts of the world including Saudi Arabia. Heave problems account for more economic loss than all other problems. In the United States the losses due to structure damages caused by expansive soil amount to $2.3 billions annually in terms of 1973 [1]. The swelling of soils in general is caused by the presence of expanding clay minerals, hydration of cations on clay surfaces, and release of intrinsic stresses caused by overconsolidation or dessication of soils. In Saudi Arabia the swelling soils responsible for structural and road damages are mostly shales which do not contain expandible minerals. Thus factors other than mineralogy play an important role in the formation of expansive soils. The shales of the region are in a very dry and intact condition due to hot and arid climate, having in situ water contents even lower than their shrinkage limits. This results in enormously high water intake potential. Upon the infiltration of water as a result of land occupation, high capillary stresses and bonding in the laminated shale structure are destroyed leading to considerable heave movements.

Reliable prediction of in situ heave is a prerequisite in developing more effective and economical design of structures on expansive soils. The choice of stabilization procedures or soil treatment technique for the purpose of minimizing the effect of soil volume change on the integrity of structure may also be guided by the magnitude of predicted heave.

This paper presents an evaluation on heave prediction methods, appropriate foundation methodology on expansive soils, and soil stabilization and treatment techniques to counteract the damaging effects of volume change due to soil expansion.

Identification and Classification of Expansive Soils

A preliminary step in investigation associated with expansive soils is qualitative characterization or classification of potential volume change. The qualitative characterization serves the purpose of warning of potential problems. In the identification methods, consistency limits and shrinkage properties are taken as the basis for the swell classification. The magnitude of soil suction is also considered as an important factor controlling the swell. The classification techniques used in determining the swell potential for a given soil are summarized in Table 1. None of these techniques, however, include parameters that reflect the in situ soil conditions such as natural dry density, soil fabric, and stress conditions. Thus, a meaningful estimation of field behaviour cannot be obtained by the classification systems, other than possible existence of problematic swelling soil conditions.

Heave Prediction

Once potentially expansive soil is identified and a qualitative indication of swell is made, quantitative characterization of swelling soil is performed to estimate the amount of anticipated volume change. Techniques available for this purpose include, oedometer method, empirical relationship, and suction analysis.

Oedometer Methods

To study the magnitude of possible swell in a plastic clay or shale, laboratory oedometer tests are usually conducted on undisturbed specimens. It should be noted that proper sampling of expansive soils is essential to preserve the natural water con-

Table 1. Expansive soil classification systems

a) ***Usaewes method*** **[2]**

Liquid limit	Plasticity index	Potential swell %	Soil suction (tsf)	Potential swell classification
<50	<25	<0.5	<1.5	Low
50–60	25–35	0.5–1.5	1.5–4	Marginal
>60	>35	>1.5	>4.0	High

Potential Swell = Vertical swell under a pressure equal to overburden pressure

b) ***Usbr method*** **[3]**

Colloid content %< 2 μm	Plasticity index	Shrinkage limit	Probable Expansion %	Expansion
<15	<18	>15	<10	Low
13–23	15–28	10–16	10–20	Medium
20–31	25–41	7–12	20–30	High
>28	>35	<11	>30	Very High

tents and to minimize sampling disturbance. Auger drilling for expansive clays of medium consistency, and air drilling with core barrel sampling in firm clays and shales are recommended in the literature in order to preserve the natural water contents by avoiding circulation of drilling water.

In testing undisturbed samples of expansive soil , it is vital to follow as closely as possible the expected stress sequence to which the soils will be subjected in the field. There are different opinions in the published methods concerning the simulation of field conditions in the oedometer tests. Four alternative methods are commonly used in the quantitative analysis of swell by oedometer techniques. All testing methods require that a soil specimen be restrained laterally and loaded axially in a consolidometer with access to free water.

Method (1)

In this method, the sample is allowed to swell freely under a seating load, and then loaded to overburden pressure plus the simulated foundation stress (P_o), [4]. The corresponding stress path is denoted by ISO and the volume change is given by the vertical interval (ae) in Fig. 1.

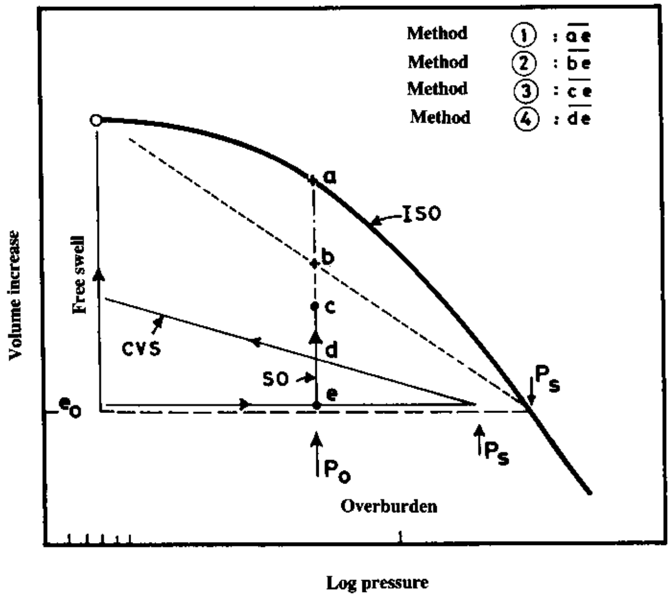

Fig. 1. Swell prediction methods

Method (2)

The sample is soaked in oedometer at a low confining pressure (*i.e.* 7 KPa), and the amount of swell is determined. The sample is then loaded to a stress level which is also referred to as the swell pressure (P_s) to attain the original void ratio. The swell under load behaviour of the soil is then predicted by a straight line which is defined by the swell at low confining stress and zero swell condition on swell versus log pressure curve, as shown by the dotted line in Fig. 1 [5]. The anticipated volume change under the stress, P_o, is defined by the vertical segment (be) in Fig. 1. The slope of the straightline will be referred to as swell index (C_s).

Method (3)

The sample is loaded to vertical stress, P_o, in one increment and then water is added to saturate the sample under the stress P_o. The amount of volume change is given by the vertical interval (ce) and the corresponding stress path is denoted by SO, in Fig. 1 [6].

Method (4)

The sample is saturated at a constant volume in the oedometer, followed by a reduction of load to vertical stress, P_o. The corresponding volume change is given by (cd) and stress path is denoted by CVS, in Fig. 1 [7]. The slope of the unloading portion of the volume change versus log stress curve is denoted as the swell index (C_s).

In Method (1), the volume changes under an overburden stress are estimated as the difference between initial void ratio and void ratio corresponding to P_o on the e-log P curve in the swell tests, whereas in the prediction of heave by methods 2, 3 and 4 three swell parameters characterize the volume change behaviour: free swell (S_F), swell pressure (P_s) and swell index (C_s). Analytically, the swell can be calculated by the following equation:

$$\Delta H = \frac{C_s}{1 + e_o} H \log \frac{P_s}{P_o} \tag{1}$$

where ΔH is the volume change, H is the height of the expansive layer, and e_o is the initial void ratio. Volume change behaviour and the swell parameters are dependent on the stress path and the wetting sequence in the oedometer tests. For an expansive shale predominantly encountered in Saudi Arabia, the magnitude of the swell parameters obtained, namely, free swell, swell pressure, and swell index obtained from ISO and CVS tests are compared in Figs. 2 to 4 respectively [8]. Invariably ISO method reveals higher magnitudes for the swell parameters than CVS method. The discrepancies in the swell parameters are due to differences in loading and wetting conditions in the following way; in ISO tests, soaking the sample, which is already disturbed due to relaxation of the in situ stresses during sample recovery, under a low confining stress promotes water penetration most efficiently. Therefore, the swell parameters are relatively high and indicate higher magnitudes of potential swell as compared to CVS tests where water entry is restricted by relatively high value of vertical stress which also restores the influence of sampling defects.

A comparison of in situ measured heave with the predictions from oedometer data, as collected from the published literature is given in Fig. 5 [9,10,11,12]. Unfortunately the details of the laboratory testing and analytical procedures used by the original authors were not specified in the publications. The data given in Fig. 5 reflects wide range of discrepancies between the predicted and measured heave when oedometer techniques are used. These discrepancies may arise from differences between laboratory and in situ conditions regarding state of stresses, soaking conditions and, lateral confinement.

Empirical Methods

In an effort to reduce the amount of time required to conduct oedometer tests to obtain data for estimating anticipated volume change, researchers began collect-

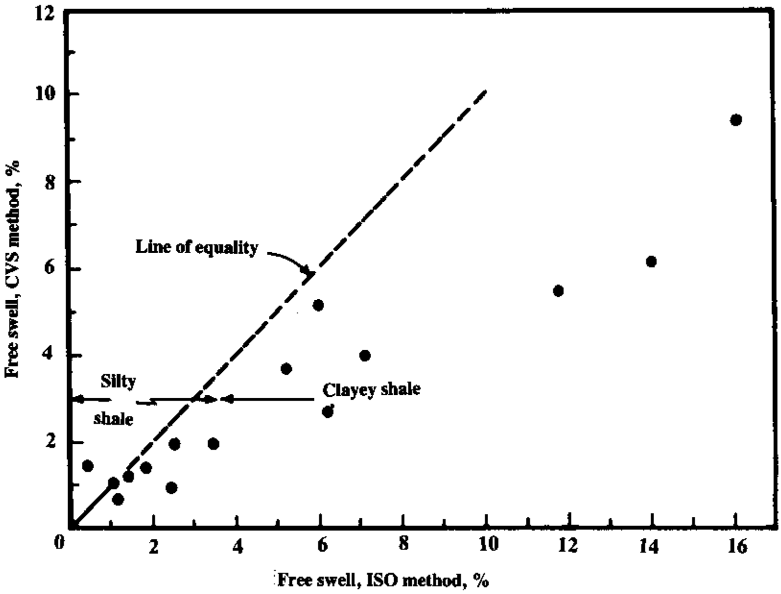

Fig. 2. Comparison of free swell for Saudi shale (Al-Ghatt shale)

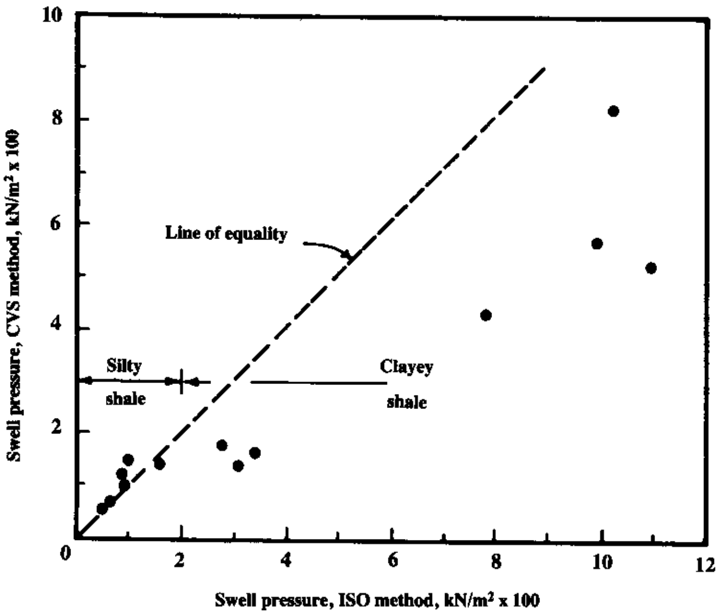

Fig. 3. Comparison of swell pressures for Saudi shale (Al-Ghatt shale)

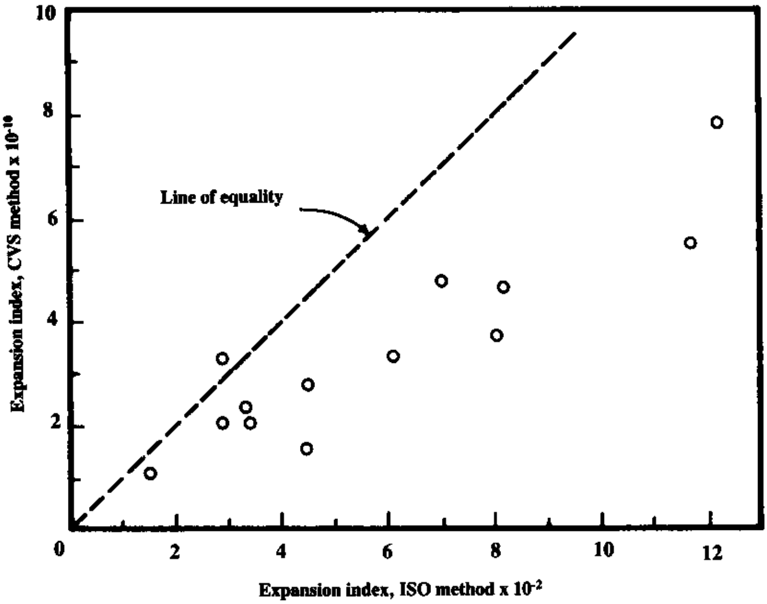

Fig. 4. Comparison of swell index for Saudi shale (Al-Ghatt shale)

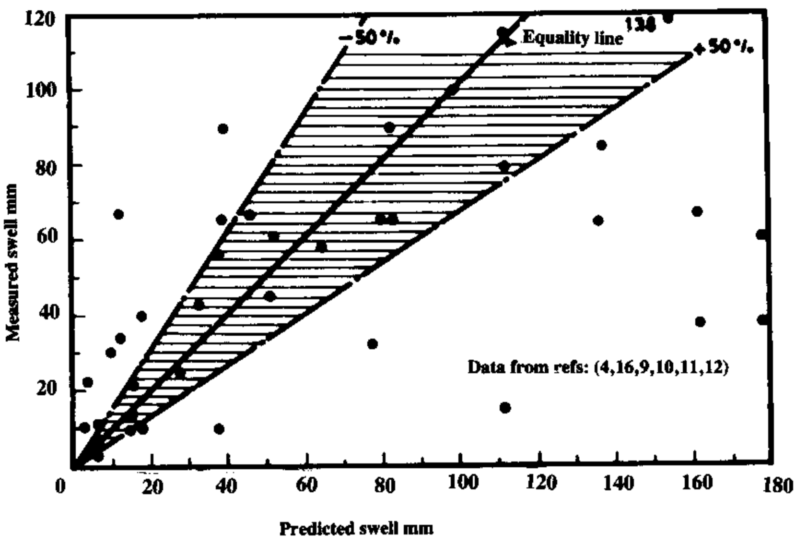

Fig. 5. Comparison of measured heave with the predictions based on oedometer data

ing oedometer test data and correlating it with physical and index properties of soils. The results of the correlation studies are equations relating swell or swell pressure to such index properties as liquid limit, plasticity index, natural water content, etc. It should be noted that, the proposed relationships are limited in application to soils outside the geographical area of consideration. This was substantiated by using Vijayvergiya *et al.* models [13,14] to predict the swell behavior of Saudi Arabia expansive shale. The models proved to be unsatisfactory when the calculated swell is compared with the measured values. On the other hand, the model suggested by Dhowian *et al.* [15] for the same soil tends to give better approximation as reflected by Fig. 6

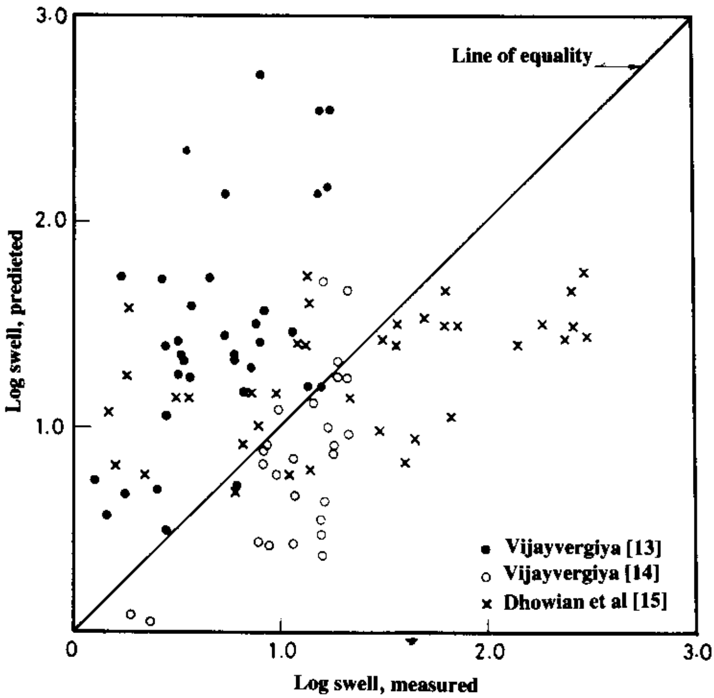

Fig. 6. Comparison of measured log swell with log swell predicted by empirical models for Saudi shale (Al-Ghatt shale)

As previously mentioned the swell of soil is caused by the increase in water content and the depth in a soil profile up to which periodic changes of moisture occur is usually referred to as the active zone. In arid regions, such as Saudi Arabia, the swell is not due to seasonal changes in soil moisture, but it is substantiated by the leakage of utility water to the foundation soil. In such environment the active zone is defined as the depth of ground which is exposed to wetting after the land utilization; and it is confined to upper few meters of the subsoil profile.

Table 2. Emprical method for predicting heave

No.	Description	Reference
1	$S_p = 0.00411\,(LL_w)^{4.17}\,\sigma_v^{-3.86}\,w_o^{-233}$	Weston [12]
2	$\text{Log}\,S_p = \frac{1}{12}\,(0.44\,LL - w_o + 5.5)$	Vijayvergiya *et al.* [13]
3	$\text{Log}\,S_p = 0.0526\,\gamma_d + 0.033\,LL - 6.8$, ($\gamma_d$ in lb/ft^3 units)	Vijayvergiya *et al.* [14]
4	$S_F = 0.925\,(0.43\,LL - \omega_i)^{0.51} + 1.19\,PI^{0.40} - 0.74\,(100-C)^{0.33}$	Dhowian *et al.* [15]
5	$S_p = 0.00216\,PI^{2.44}$	Seed *et al.* [16]
6	$\Delta H = Fe^{-0.377D}\,(e^{-0.377H} - 1)$	Van der Merve [17]
7	$S_p = [(0.00229\,PI)(1.45C)/w_o] + 6.38$	Nayak *et al.* [18]
8	$P_s = [(3.58 \times 10^{-2})\,PI^{1.12}\,C^2/w_o^2] + 3.79$, ($P_s$ in psi)	Nayak *et al.* [18]
8b	$\text{Log}\,S_p = 0.9\,(PI/w_o) - 1.19$	Shneider *et al.* [19]
9a	$S_p = 23.82 + 0.7346\,PI - 0.1458\,H - 1.7\,w_o + 0.00225\,PI\,w_o - 0.0088\,PI\,H$	Johnson [20]
9b	$S_p = -9.18 + 1.5546\,PI + 0.08224\,H + 0.1\,w_o - 0.0432\,PI\,w_o - 0.0215\,PI\,H$	
10	$\text{Log}\,P_s = -2.132 + 0.0208\,LL + 0.000665\,\gamma_d - 0.0269\,w_o$	Komornik *et al.* [21]

S_p	Percent swell, %	LL	Liquid limit
PI	Plasticity index	w_o	Initial water content
ΔH	Total heave	γ_d	Dry unit weight
F	Correction factor for degree of expansiveness	C	Clay percent
D	Thickness of nonexpansive layer	σ_v	Surcharge load
H	Thickness of expansive layer	LL_w	Weighted liquid limit
P_s	Swell pressure		

Among the numerous methods published in current literature the most commonly referred correlations are summarized in Table 2. More details have been presented in Dhowian *et al.* [22]. Using the empirical methods, swell under any surcharge load can be predicted by a straight line which is defined by the swell at low confining stress and zero swell condition.

Once, the depth of active zone is known, and the swell parameters are determined from equations listed in Table 2, the procedure described in Fig. 7 can be used to predict the heave at the ground level. According to Brackley [5] the swell-log pressure relationship can be obtained by joining the points A and B by a straight line, in Fig. 7. Then, at any depth, Z_i, the percent swell S_{Li} can be obtained from line AB on Fig. 7. Then, S_L versus depth plot can be established as shown in Fig. 7. The Surface heave ΔH is calculated as follows:

$$\Delta H = \sum_{i=1}^{n} (S_{Li}\,\%)\,\frac{H_i}{100} \qquad (2)$$

where

ΔH = surface heave,

H_i = thickness of n^{th} layer,

S_{Li} = percent swell in the n^{th} layer,

n = number of layers,

P_o = overburden stress plus the vertical stress from foundation loading on n^{th} layer.

The surface heave can also be calculated as the area under percent swell S_{Li} versus depth curve, as illustrated in Fig. 7.

The swell data obtained from oedometer tests correspond to fully restrained case which does not simulate the in situ conditions. In other words the specimens are laterally restrained, and the volume change occurs in vertical direction only. However, there are intensive cracks and fissures in natural expansive soils as a result of dessication and/or periodic shrink-swell mechanism taking place over long period of time. These cracks provide room for the natural soils to expand in lateral direction also. Therefore, in situ heave is reflected as a vertical component of the volumetric swell, rather than the entire volume change. In order to predict the real in situ vertical heave in a soil profile, some assumptions concerning the lateral restraint factor that accounts for the presence of cracks and fissures are to be introduced.

Suction Methods

Soil suction or negative pore pressure is important in controlling mechanical properties of partially saturated soils. Recently, various methods for determining

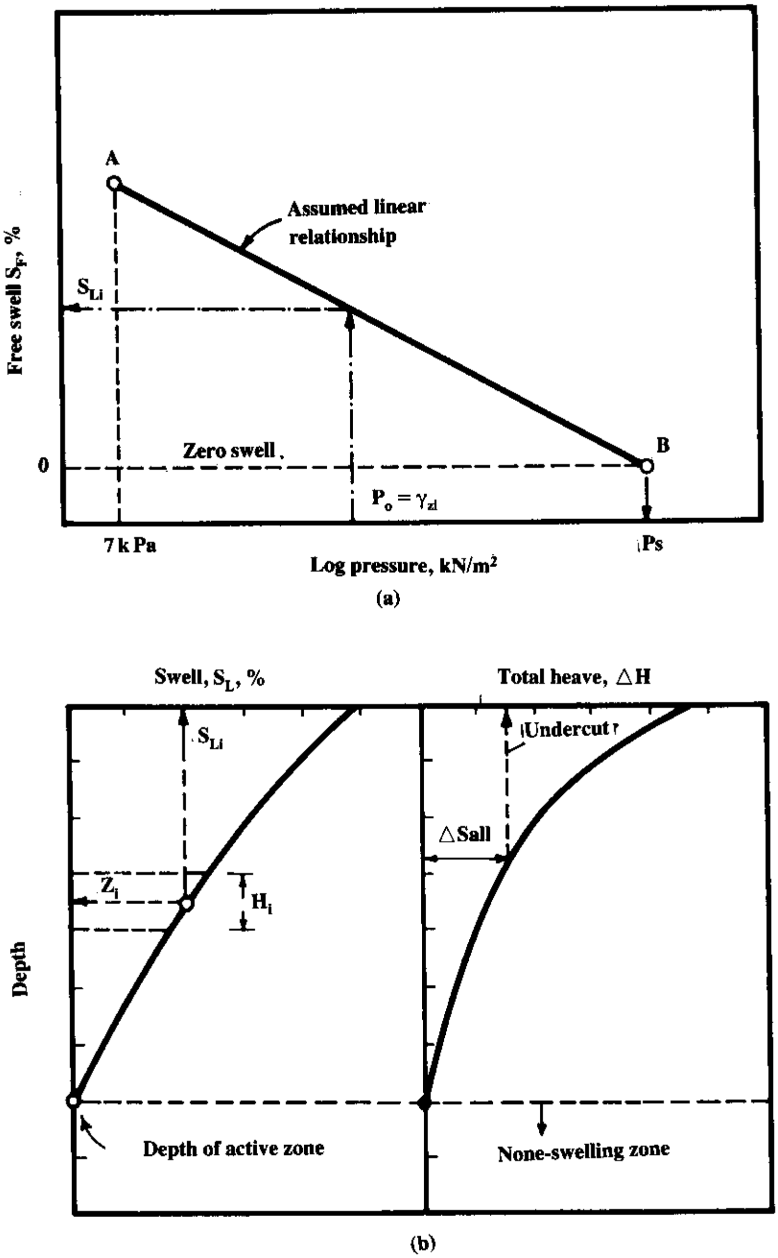

Fig. 7. Method of heave computation

swell potential and predicting in situ volume changes based on soil suction have been proposed. The soil suction measurements became a routine procedure after the application of thermocouple psychrometers to soil suction measurements. These methods eliminate the need for oedometer tests, take little time, and require inexpensive equipment [6].

In the swell prediction methods based on soil suction, swell is related to change in the soil suction through a volume change parameter. This parameter is analogous to the compression index (C_c) for consolidation process, and is an intrinsic property of soil. Aitchison [23] gives the following relationship to estimate heave of a soil profile due to change in suction:

$$\Delta H = \frac{\partial \varepsilon}{\partial \log \psi} H \log \psi \tag{3}$$

where ΔH is the heave, ε is the vertical soil strain, $\Delta \log \psi$ is the change in the soil suction, and H is the thickness of the expansive layer. In equation (3) the term ($\partial\varepsilon/\partial\log \psi$) is defined as the instability index (I_p) which relates vertical heave to change in soil suction. The measurement of this parameter through laboratory testing is difficult, but an approximate value can be determined by swell-shrinkage tests [24] to represent the in situ soil conditions appropriate to the problem. Snethen and Johnson [25] define a similar parameter (*i.e.* suction index, C_ψ), which is determined in the laboratory by measuring specific volume (*i.e.* reciprocal of dry density) of undisturbed specimens and the corresponding soil suctions independently at various water contents. In this method the swell is given by the following equation:

$$\frac{\Delta H}{H} = \frac{C_\psi}{1 + e_o} \log \frac{\psi_i}{\psi_f + \alpha\sigma_v} \tag{4}$$

where ψ_i and ψ_f are initial and final soil suctions. σ_v is the vertical stress and e_o is the initial void ratio. The suction index (C_ψ) reflects the ratio of change of void ratio with respect to soil suction and can be calculated as follows:

$$C_\psi = \frac{\alpha G_s}{100B} \tag{5}$$

where α is the volume compressibility factor, G_s being the specific gravity of the solid particles, and B is the slope of log suction versus water content curves.

In the absence of laboratory data the compressibility factor (α) may be roughly estimated from plasticity index (PI) by:

$$PI < 5 \qquad \alpha = 0$$
$$PI > 40 \qquad \alpha = 1.0$$
$$5 < PI < 40 \qquad \alpha = 0.0275\,PI - 0.125$$

Otherwise the value of the α factor can be measured from the slope of the reciprocal of the dry density (specific volume) versus water content plots. The anticipated volume change should be calculated for each layer in the profile (*i.e.* within the active zone) and summed to obtain the total surface movement. The Snethen and Johnson [25] procedure is a convenient way to predict field heave without detailed laboratory testing. This method requires evaluating the initial and final soil suction (*i.e.* corresponding to ultimate swell condition). Experimental studies have shown that the log suction versus water content behaviour can be approximated by a straight line as given by the following relationship:

$$\log \psi = A - BW \tag{6}$$

where ψ = soil suction, bars,
A,B = intercept and slope of log suction, water content curve
w = water content.

Thus, for a given expansive soil, the water contents and the corresponding soil suctions are measured from which the values of A and B in Eq. (6) are determined. The initial water content of a soil profile is determined in any routine site investigation program. However, some assumptions should be made concerning the value of final water content which corresponds to ultimate swell condition. It has been found that the swell parameters is practically negligible at water content above the plastic limit [22]. Accordingly, it may be assumed that the final suction where the ultimate swell condition occurs is at water contents close to plastic limit. Hamberg *et al.* [26], Vijayvergiya *et al.* [14] and Weston [12] have suggested similar criteria concerning the final water contents in the process of swell. Hence, the final suction value to be used in Eq. 6 can be calculated using the Plastic limit as the upper limiting water content.

Mitchel *et al.* [27], Hamberg *et al.* [26] described slightly different procedures than Snethen and Johnson [25]. Brackley [28] proposed an empirical expansion index which is similar to suction index (C_ψ) expressed as a function of plasticity index in the following way:

$$C_\psi = \frac{PI - 10}{10} \tag{7}$$

where PI is the plasticity index of the soil. The percent swell is then given by the following equation:

$$\frac{\Delta H}{H} = C_\psi \log \frac{\psi_i}{\sigma_v} \tag{8}$$

A comparison of measured heave with the predicted swell based on soil suction methods is given in Fig. 8. The data given in the figure is collected from published literature and predictions are presented as reported by the original author [6,24,27,28,29]. The distribution of data points in Fig. 8 indicates that suction methods are more reliable as compared to oedometer methods (Fig. 5). The predicted swell mostly lies within ± 50% of measured heave, the majority of the points being in the overestimated zone.

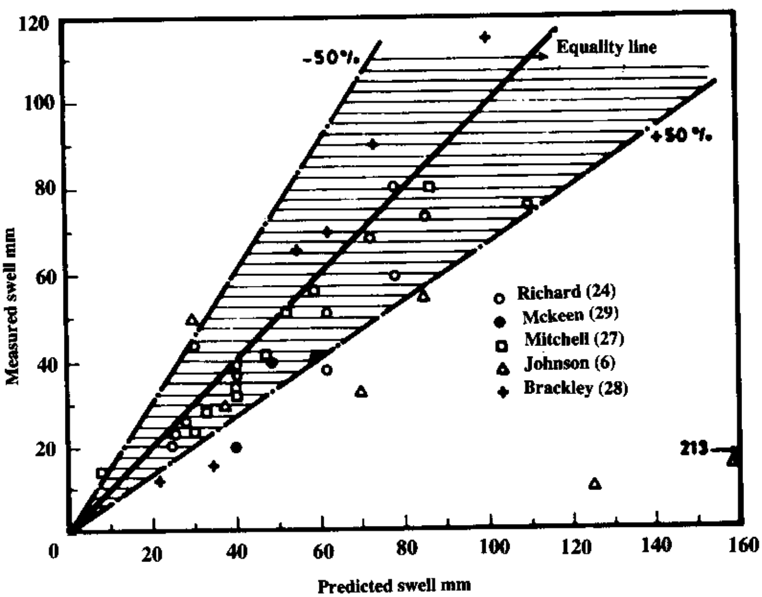

Fig. 8. Comparison of measured heave with the predictions based on suction methods

Direct heave measurements of shale formation in the middle province of Saudi Arabia in an instrumented field station are reasonably represented by the predicted ground movements based on constant volume, oedometer, CVS, test. Improved swell oedometer, ISO, test and suction analysis using Brackley method [28] tend to overestimate the in situ heave whereas Snethen and Johnson suction technique [25] highly overestimates the field swell behavior as shown in Fig. 9.

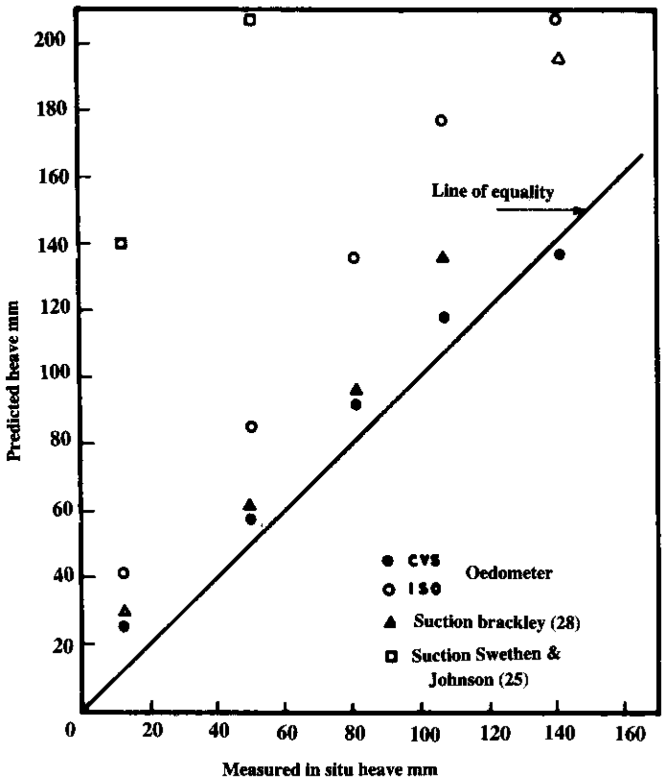

Fig. 9. Measured versus predicted in situ heave of Saudi shale (Al-Ghatt shale)

Foundation Methodology and Stabilization of Expansive Soils

If a soil is classified as having a low swell potential, standard construction practices may be followed. However, if the soil possesses marginal or high swell potential, precautions need to be taken. Several treatment procedures have been successfully used to counteract the adverse effects of expansive soils. One particular method may not be adequate in every situation. Instead, it may be necessary to combine more than one technique for this purpose. In addition, the local construction experience should always be taken into consideration whenever designing or treating swelling formation. In the proceeding paragraphs the techniques found effective in dealing with expansive soils will be presented in brief.

Replacement of Expansive Soil

When moderately expansive soils of low thickness are present near the ground surface, they can be removed and replaced by less expansive soils and then compacted properly. Thorough investigation should be carried out to determine the thickness and extension of expansive strata prior to replacement. A typical example of soil replacement is illustrated in Fig. 10.

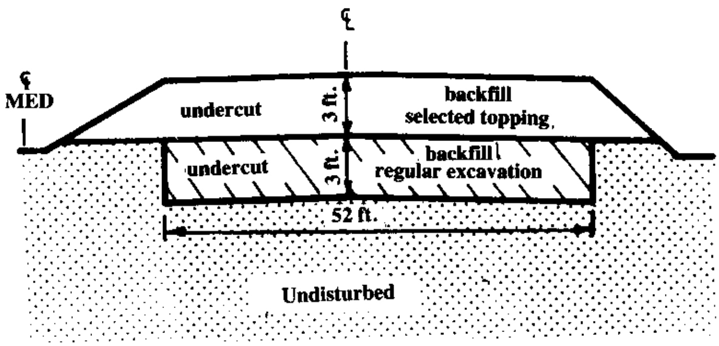

Fig. 10. Typical example of sub-excavation and replacement for an interstate highway (after Snethen, 1979)

Changing the Nature of Exapansive Soils

Certain measures can be taken to change the nature of the expansive soil whereby its swelling potential is greatly reduced. Such measures may include the followings.

Compaction

Heave of expansive soils is known to decrease substantially when the soil is compacted to a lower unit weight on the high side of the optimum water content. Even under such conditions, a slab on grade type construction should not be encouraged where the total heave may reach some undesirable magnitudes.

Prewetting

Prewetting by ponding or submerging an area in water allows desiccated foundation soil to swell and nearly reach equilibrium water content prior to construction.

Prewetting can be effective, but may require long time (some months) unless the foundation soil contains an extensive fissure system. Prewetting to about 2–3 percent above the plastic limit provides significant improvement of the performance of slab on ground foundations. Excessive prewetting, however, may be detrimental to foundations where moisture in wetted soil can migrate down into dry deeper soil, and cause high swell.

Installation of a grid of vertical sand wells prior to flooding can significantly reduce the time needed for ponding. Lime mixed with the ponded water helps to increase migration of water, through an increase in soil permeability. After ponding, 4–5% hydrated lime may be added to the top layer of the soil to make it less plastic and more workable [30].

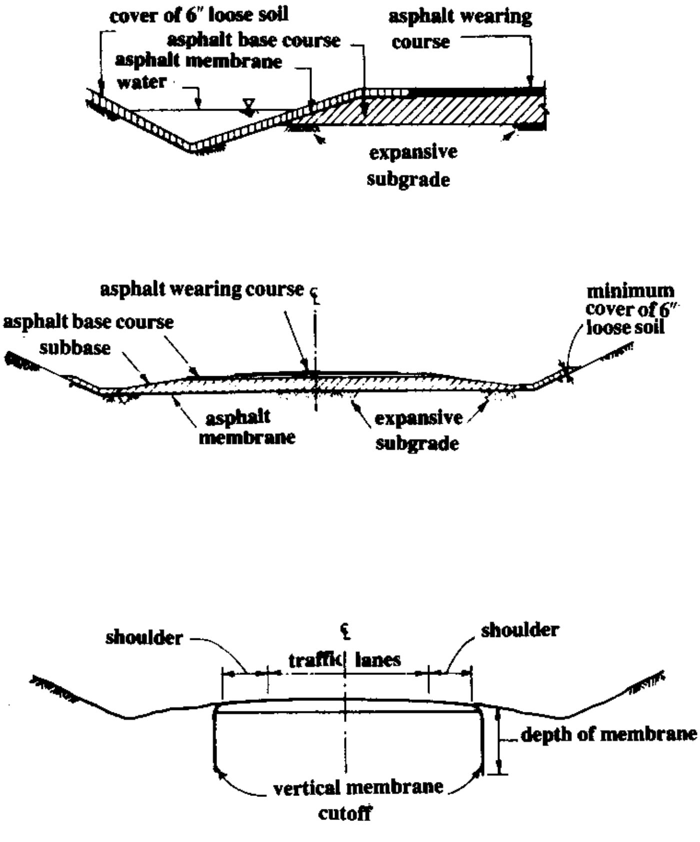

Fig. 11. Typical examples of membrane application in highway construction (after Snethen, 1979)

Use of Drainage System and Moisture Barriers

The long term effect of differential heave can be reduced by controlling the moisture variation in the soil through the use of membrane as shown in Fig. 11 where typical examples of membrane application in highway construction are illustrated. Moisture variation can also be achieved by providing horizontal and vertical (up to 1.5 m deep) moisture barriers around the perimeter of the slabs for the slab-on-grade type construction. These moisture barriers may be constructed as drainage trenches filled with sand and gravel with the provision of perforated pipes and impervious membranes as shown in Fig. 12. Moreover, sewer and water lines near the structure should be constructed with watertight and flexible joints. The vertical membranes should be placed about 1 m from the foundation to avoid disturbance of foundation soil, and be extended to the bottom of the active zone [31].

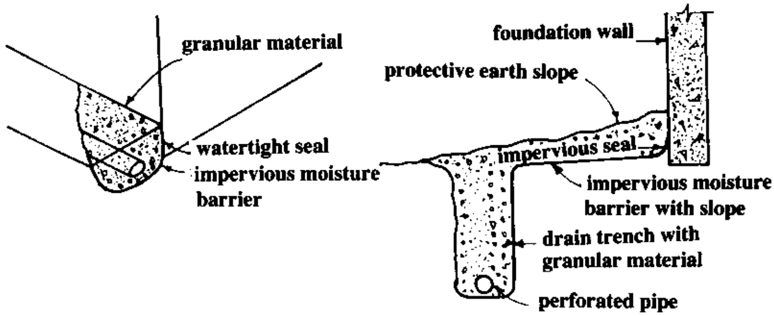

Fig. 12. Construction details of moisture barriers

Membranes are usually made of impervious plastic materials such as polyvinyl choloride (PVC), polyethylene, asphaltic fiberglass sheets or sprayed bitumen.

Stabilization of Soil

Chemical stabilization with the aid of lime and cement has often proved useful. Small addition of lime from 2 to 8 percent usually decreases the plasticity index and swell, and increases the permeability and shear strength of expansive clays. This type of stabilization can be done to a depth of about 1–1.5 m, and the effectiveness of lime treatment depends on the thoroughness of mixing. Pressure injection of lime may be effective in soils containing extensive fissures and cracks such as shales into which the slurry can be injected. The injected slurry deposited in fissures appears to provide an effective lime barrier against moisture flow as well as prewet the soil from absorption of the slurry [32]. This method has been effectively used to reduce the swelling potential of dry shale in the northern part of Saudi Arabia where a lime slurry was injected into the shale formation to restore the integrity of severely cracked wall bearing structures [33].

Construction Practices on Expansive Soils

The design of superstructure and foundation should be chosen to satisfy most economically the functional requirements of the structure, minimize soil differential movement, and minimize damages that may occur to the structure from soil movement. Usually the type of structure and foundation system are selected according to a criterion which limits the deflection/length ratio to less than a critical value. Framed structures, for example, are known to be able to withstand considerable movement without exhibiting noticeable damage. This is in contrast to wall bearing buildings which are more sensitive to soil volume change.

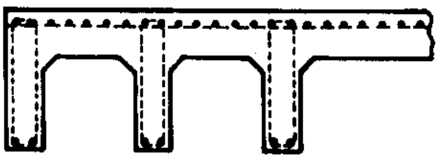

Fig. 13a. Stiffened mat (Waffle slab)

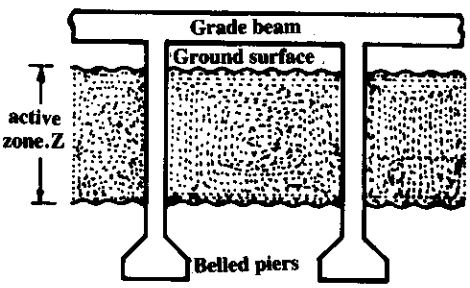

Fig. 13b. Pier foundations with suspended slabs

The use of waffle slabs (stiffened mats) is suggested as an alternative in designing rigid buildings capable of tolerating ground movements. A schematic diagram of a waffle slab is shown in Fig. 13a. In this type of construction, the ribs hold the structural load, and the waffle voids allows the expansion of soil. Also, the use of foundation piers with a suspended slab for the construction of structures independent of movement is suggested whenever a swelling stratum of reasonable thickness is encountered. Fig. 13b shows a schematic diagram of such an arrangement. The bottom of the piers should be placed below the active zone of the expansive soil. Some constructional details have always been helpful to counteract the effect of soil heave on the structure. Placement of compressible joint fillers or open blocks beneath the grade beams can effectively reduce the swell pressures. A typical example of such constructional details is shown in Fig. 14.

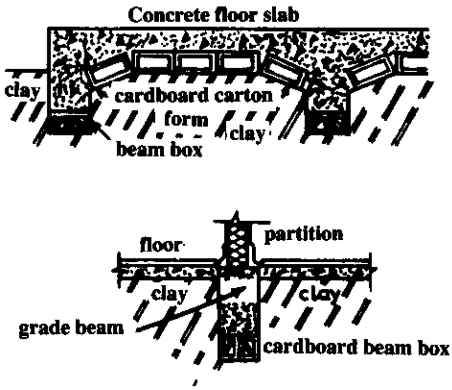

Fig. 14. Construction details of open blocks beneath the grade beam

Conclusions

Volume change due to soil expansion has been recognized as one of the main causes of structural distressing and utility damages. Based on the discussion presented in this paper the following conclusions may be advanced:

1. Swelling of soil is a complex phenomenon and several methods have been proposed for the purpose of heave prediction including the oedometer test method, empirical relationships, and suction technique. Unfortunately there is a lack of agreement between these methods.

2. Most of the empirical relationships are derived from data for specific climatic and enviornmental conditions and hence, should be used with caution.

3. Data from published literature indicates that suction method gives more realistic estimation of in situ heave as compared with oedometer technique. However, experience of the author with expansive shale shows that analysis based on constant volume oedometer test results in reasonable heave prediction.

4. There are several alternative methods to counteract expansive soil problem falling into two broad categories; soil treatment, and special foundation

design. None of the methods have been found fully satisfactory. Case histories are needed to establish sound criterion for the selection of proper treatment and design methods.

Reference

[1] Johns, D.E.Jr. and Holt, W.G. "Expansive Soils – the Hidden Disaster·" *Civil Engineering (ASCE)* 43, No.CE8 (1973), 49.

[2] Snethen, D.R. "Evaluation of Expedient Methods for Identification and Classification of Potentially Expansive Soils." *5th Int. Conf. on Expansive Soils,* (1985), pp. 22-27.

[3] Holtz, W.G. "Expansive Clays: Properties and Problems·" *Quarterly, Colorado School of Mines,* 54, No.4 (1969), 89-125.

[4] Jennings, J.E. "The Prediction of Amount and Rate of Heave Likely to be Experienced in Engineering Construction." *Proc.* 2nd *Int. Conf. on Expansive Soils,* (1969), pp. 99-105.

[5] Brackley, I.J.A. "Swell under Load." 6th *Regional Conference for Africa,* Durban, S.A. Vol.2 (1975), pp. 65-70.

[6] Johnson, L.D. and Snethen, D.R. "Prediction of Potential Heave of Swelling Soils." *ASTM Geotechnical Testing Journal* 1, No.3 (1978), 117-127.

[7] Sullivan, R.A. and McClelland, B. "Predicting Heave of Buildings on Unsaturated Clays." *Proc.* 2th *Int. Conf. on Expansive Soils,* Texas (1969), pp. 404-420.

[8] Erol, A.O.; Dhowian, A.W. and Youssef, A. "Assessment of Oedometer Methods for Heave Prediction." 6th *Int. Conf. on Expansive Soils,* New Delhi, India, 1-4 Dec. (1987), pp. 1-5.

[9] Snethen, D.R. "Technical Guidelines for Expansive Soils in Highway Subgrades." Dept. of Transporation, U.S.A., *Final Report* No. FHWA-RD-79-51 (1979).

[10] Justo, J.L. "Design Parameters for Special Soil Conditions." *Proc. of* 7th *European Conference on Soil Mechanics,* Vol.4 (1979), pp. 127-158.

[11] Holland, J.E.; Pitt, W.G.; Lawrence, C.E. and Cimino, D.J. "Behaviour and Design of Housing Slabs on Expansive Clays." *Proc. of* 4th *Int. Conf. on Expansive Soils,* Vol. 1 (1980), pp. 448-467.

[12] Weston, D.J. "Expansive Roadbed Treatment for Southern Africa." *Proc. of* 4th *Int. Conf. on Expansive Soils,* Vol.1 (1980), pp. 339-360.

[13] Vijayvergiya, V.N. and Gazzaly, O.I. "Prediction of Swelling Potential for Natural Clays." *Proc. of* 3rd *Int. Conf. of Expansive Soils* (1973), pp. 227-234.

[14] Vijayvergiya, V.N. and Sullivan, R.A. "Simple Technique for Identifying Heave Potential." *Proc. of Workshop on expansive Clays and Shales,* Vol. 2 (1973), pp. 149-154.

[15] Dhowian, A.W.; Erol, A.O. and Youssef, A. "Evaluation of Expansive Soils and Foundation Methodology in the King of Saudi Arabia." 6th *Progress Report* SANCST AT-5-88 (1985).

[16] Seed, H.B.; Woodward, R.J. and Lungren, R. "Prediction of Swelling Potential for Compacted Clays." *Proc. of ASCE, JSMFD,* Vol. 88, SM 3 (1962), pp. 58-67.

[17] Van der Merve, D.H. "The Prediction of Heave from the Plasticity Index and Percent Clay Fraction." *Trans. of South Af. Inst. of CE.,* Vol.6 (1964), pp. 103-107.

[18] Nayak, N.V. and Christensen, R.W. "Swelling Characteristics of Compacted Expansive Soils." *Clay and Clay Minerals,* 19 (1974), 251-261.

[19] Schneider, G.L. and Poor, A.R. "The Prediction of soil Heave and Swell Pressures Developed by an Expansive Clays." *Research Report* TR-9-74, Construction Research Center, Univ. of Texas (1974).

[20] Johnson, L.D. "Predicting Potential Heave and Heave with Time in Swelling Foundation Soils." *Technical Report* S-78-7, USAWES (1978).

[21] Komornik, A. and David, D. "Prediction of Swelling Potential of Clays." *Proc. ASCE, JSMFD,* Vol. 95, SM 1, (1969), pp. 209-225.

[22] Dhowian, A.W.; Erol, A.O. and Youssef, A. "Evaluation of Expansive Soils and Foundation Methodology in the Kingdom of Saudi Arabia." *Final Report,* SANCST AT-5-88 (1985).

[23] Aitchison, G.D. "A Quantitative Description of the Stress Deformation Behaviour of Expansive Soils." *Proc. of 3rd Int. Conf. on Expansive Soils,* Vol. 2 (1973), pp. 79-82.

[24] Richard, B.G.; Peter, P. and Emerson, W.W. "The Effects of Vegetation on the Swelling and Shrinking of Soils in Australia." *Geotechnique,* 33 (1983), 127-139.

[25] Snethen, D.R. and Johnson, L.D. "An Investigation of Natural Microscale Mechanism that Causes Volume Changes in Expansive Clays." Dept. of Transporation, U.S.A. *Research Report* No.FHWA-RD-77-75 (1977).

[26] Hamberg, D.J. and Nelson, J.D. "Prediction of Floor Slab." *Proc. of 5th Int. Conf. on Expansive Soils,* (1985), pp. 137-140.

[27] Mitchell, P.W. and Avalle, D.L. "A Technique to Predict Expansive Soil Movements." *Proc. of 5th Int. Conf. on expansive Soils* (1985), pp. 124-129.

[28] Brackley, I.J.A. "Prediction of Soil Heave from Suction Measurements." *7th Regional Conf. for African, SMFE,* (1980), pp. 159-166.

[29] McKeen, G.R. "Field Studies of Airport Pavements on Expansive Clay." *Proc. of 4th Int. Conf. on Expansive Soils,* Vol. 1 (1981), pp. 241-261.

[30] Das, B.M. "Principles of Foundation Engineering." *PWS Engineering Publications,* (1984), 452-479.

[31] Johnson, D.L. "Overview for Design of Foundations on Expansive Soils." *USAE, WES, Report* No. AT.40-EO-004 (1979).

[32] O'Neill, M.W. and Poormoayed, N. "Methodology for Foundations on Expansive Clays." *Proc. ASCE, JGED,* Vol. 106, GT.12 (1980), pp.1345-1367.

[33] Dhowian, A.W. "Characteristics of Expansive Soils in the Northern Region of Saudi Arabia." *5th Int. Conf. on Expansive Soils,* Adelaide, South Australia, 21-23 May (1984),, pp. 316-320.

طرق التنبوء بانتفاخ التربة والاحتياطات التصميمية على التربة المتنفخة

عبدالمحسن وني الضويان

قسم الهندسة المدنية، كلية الهندسة، جامعة الملك سعود، ص.ب ٨٠٠،
الرياض ١١٤٢١، المملكة العربية السعودية

ملخص البحث. يعد انتفاخ التربة مسئولا عن كثير من الأضرار التي تحدث في المنشآت ويترتب عليها خسائر مالية كبيرة. ولتطوير تصميم فعال للمنشآت لكي تتحمل آثار الانتفاخ لابد من تقدير حجم هذا الانتفاخ، وهناك ثلاث طرق يمكن استخدامها لهذا الغرض وهي : اختبار الضغط «الأودومتري» وطريقة الامتصاص وطريقة العلاقات التجريبية. كما أوضح البحث أساليب معرفة قابلية التربة للانتفاخ والحساب التحليلي الكمي له بالإضافة إلى طرق الاختيار المختلفة لمعالجة التربة وطرق التصميم عليها.