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Optimum Replacement Depth to Control Heave of Swelling Clays

Dr. Hisham Arafat,Dr. Ahmed M. Ebid

Abstract*—The behavior of unsaturated swelling soils under changing of moisture content was intensively studied by many researchers since the 1950's. Many proposed formulas and techniques were used to classify, describe and predict the swelling behavior and parameters of such type of soil. On the other hand, many techniques are used to allow structures to be founded on swelling soils without suffering any damages due to the soil heave. Replacing the swelling soil with granular mixture is one of the most famous and cheapest techniques especially in case of light structures on shallow layer of swelling soil. The aim of this research is to develop a simplified formula to estimate the heave of swelling soil considering the effect of replaced layer. The developed formula is used to estimate the required replacement depth to avoid damage due to excessive heave.

***Index Terms*— swelling soil, expansive clay, heave, replacement thickness, sand cushion**

I. INTRODUCTION

The swelling behavior of unsaturated clays was studied by many researches to understand the reasons, mechanism and the parameters affecting this phenomenon. Generally, increasing water content of swelling soil sample causes increasing sample volume due to the chemical reaction between the water and the active clay minerals in sample. The amount of sample heave depends on the quantity and type of active minerals as well as the initial and final degree of saturation. Also, the external applied stresses on the sample have a significant effect on the heave. The aim of this research is to develop a simplified analytical formula abide of sophisticate numerical techniques to estimate the heave of swelling soil considering the effect of replaced layer. The developed formula is used to estimate the required replacement depth to avoid damage due to excessive heave.

A. Swelling clay behavior

The relation between clay water content and its ability to swell is highly nonlinear as shown in Fig.(1). Generally, dry clay ability to swell (measured as metric suction) decreases rapidly with increasing the water content. As water content increases the rate of suction decreases and the swelling ability vanishes when the clay becomes fully saturated as well. As show in Fig.(1), initial water content of clay has major effect on its ability to swell, while the final water content is less important because the final case is always assumed fully saturated to estimate maximum possible heave. Drier clays have more ability to swell than wetter clays; that is why swelling risk is almost trivial in sites with high ground water table, even if the clay has high percentage of active minerals, clay layer above ground water table will be nearly saturated due to capillary action while clay layer below ground water table is already submerged.

External applied compressive stresses on unsaturated clay sample causes consolidation and decreasing sample volume. The more applied stress, the more decreasing in volume. The stress required to decrease the volume of swelled sample to its original volume is called "Swelling Pressure" (Ps). Swelling pressure could be measured experimentally from the odometer test as shown in Fig.(2). By definition, if the external applied stress from the structure is equal to or more than the swelling pressure, than this structure will not suffer any heave. Heavier buildings will suffer less heave than lighter buildings. The heave effect is worst for the weightless structures like pavement, pipe lines, railways and transmission towers.

Swelling pressure depends on the amount of heave to be reduced, that heave depends on the initial water content, type and content of active menials, and hence, there is a relation between swelling pressure, water content, clay content and clay activity. This relation is too complicated to be derived mathematically, so, many researches tried to approximate this relation based on experimental results using different techniques such as multi-variable regression and artificial neural networks (ANN). The most used formulas are shown in Fig.(3)

B. Identification of swelling soil

Estimating the soil ability to swell (swelling potential) was intensively studied by many researches. Each researcher suggested a scale to classify the soils according to its swelling potential based on some basic laboratory tests. Earlier researchers used simple consistency limits and free swelling tests to classify the swelling soil. As soil mechanics laboratory becomes more developed and equipped, more advanced tests are used to classify the swelling soils odometer, mineralogy and cation exchange tests. Most researches classify swelling soils according to its swelling potential into four categories: low, medium, high and very high. Some of the most famous classification methods are summarized in Fig.(4).

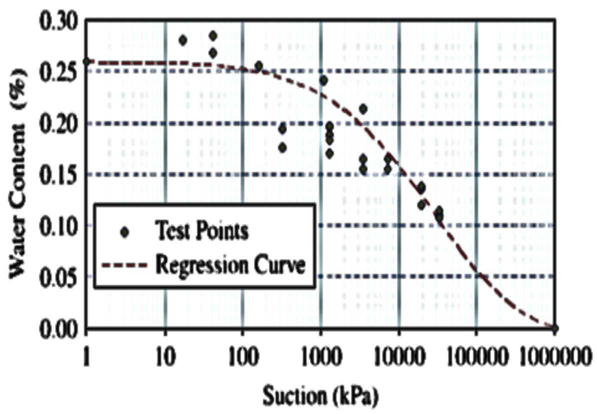

Fig.(1): Typical relation between water content and metric suction [1]

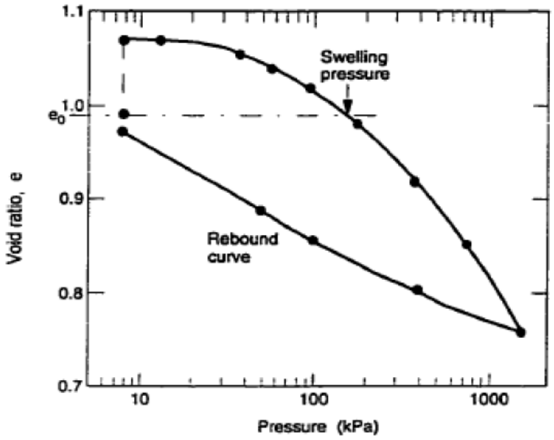

Fig.(2): Measuring swelling pressure using odometer test [2]

Reference	Emprical relationships	Remarks
Seed et al.	$SP = 0.00216PI^{2.44}$ for undisturbed $SP = 0.0036PI^{2.44}$ for disturbed soils	±35 % approximation soils
Komornik and David	$\text{Log}P_s = -2.132 + 0.0208LL + 0.000665\gamma_d - 0.0269w_0$	For undisturbed soils, $\gamma_d = \text{kg/m}^3$, $P_s = \text{kg/cm}^2$
Vijayvergia and Ghazzaly	$\text{LogSP} = (0.44LL - w_0 + 5.5)/12$	For undisturbed soils
Vijayvergiya and Sullivan	$\text{LogSP} = 0.0526\gamma_d + 0.033LL - 6.8$	For undisturbed soils, $\gamma_d = \text{lb/ft}^3$
Nayak and Christensen	$SP = (0.0229PI^{1.45}C)/w_0 + 6.38$	For soils compacted to the maximum standard AASHTO unit weight at optimum water content by free swell test, $P_s =$ Psi
	$P_s = 0.035817PI^{1.12}C^2/w_0^2 + 3.7912$	
Schneider and Poor	$\text{LogSP} = 0.9PI/w_0 - 1.19$	For undisturbed soils
Johnson	$SP = 23.82 + 0.7346PI - 0.1458H - 1.7w_0 + (0.0025PI)w_0 - (0.00884PI)H$	For undisturbed soils, at PI ≥ 40 %
	$SP = -9.18 + 1.5546PI + 0.08424H + 0.1w_0 - (0.0432PI)w_0 - (0.01215PI)H$	For undisturbed soils, at PI ≤ 40%
Weston	$SP = 0.00411(LL_w)^{4.17}q^{-3.86}w_0^{-2.33}$	$LL_w = (\% < 0.425\ \text{mm}/100)LL$
Chen	$SP = 0.2558e^{0.0838PI}$	Compacted soils with initial conditions at $\gamma_d = 15.7 - 17.3\ \text{kN/m}^3$, $w_0 = 15 - 20\,\%$ by free swell test
Basma	$SP = 0.00064PI^{1.37}C^{1.37}$	For soils compacted to the maximum standard AASHTO unit weight at optimum water content by free swell test
Erguler and Ulusay	$P_s = -227.27 + 2.14w_0 + 1.54LL + 72.49\gamma_d$	For remoulded samples, ASTM Method B, $P_s = \text{N/cm}^2$, $\gamma_d = \text{gr/cm}^3$
Erzin and Erol	$\text{Log}P_s = -4.812 + 0.01405PI + 2.394\gamma_d - 0.0163w_0$	
	$\text{Log}P_s = -5.020 + 0.01383PI + 2.356\gamma_d$	For constant volume swell test, $\gamma_d = \text{gr/cm}^3$, $P_s = \text{kg/cm}^2$
Sabtan	$SP = 1.0 + 0.06(C + PI - w_0)$	
	$P_s = 135.0 + 2.0(C + PI - w_0)$	For undisturbed samples, ASTM Method A, $P_s = \text{kPa}$

SP swelling potential (%), P_s swelling pressure, *PI* plasticity index (%), *LL* liquid limit (%), w_0 initial water content (%), γ_d dry unit weight, C clay content (%), H depth of expansive layer (feet), LL_w weighted liquid limit (%), q surcharge load

Fig(3): Summary for the most famous empirical formulas to estimating swelling potential (SP) and swelling pressure (Ps) [3]

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Based on Plasticity Index, Chen (1988)

Plasticity Index (PI)	Swelling Potential
< 15	Low
10 – 35	Medium
35 – 55	High
> 55	Very high

Based on Plasticity and Shrinkage Index, (Raman, 1967)

Plasticity Index (PI)	Shrinkage Index (SI)	Swelling Potential
< 12	< 15	Low
12 – 23	15 – 30	Medium
23 – 32	30 – 40	High
>32	>40	Very high

Based on Colloid Content, Plasticity Index and Shrinkage Limit (Holtz, 1959)

Colloid Content (%)	Plasticity Index (PI)	Shrinkage Limit (SL)	Expansion (%)	Swelling Potential
< 15	< 18	> 15	< 10	Low
15 – 23	15 – 28	10 – 16	10 – 20	Medium
20 – 31	25 – 41	7 – 12	20 – 30	High
>28	>35	<11	>30	Very high

Based on Clay Content, Plasticity Index (Van Der Merwe, 1964)

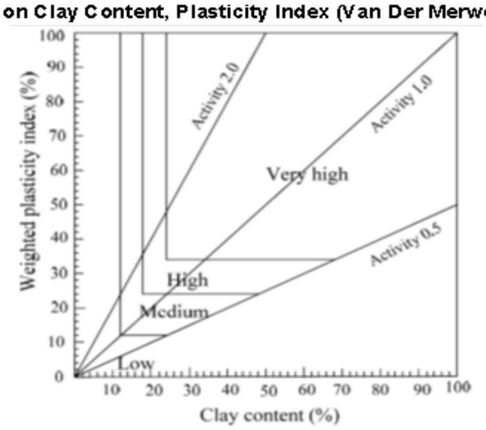

Fig.(4): Summary for the most famous methods to classify swelling soils [4]

$$Sp = -100\, \gamma_h \log (\tau_f / \tau_o)$$ **McKeen and Lytton's Model (1981)**

Where:
γ_h : suction compression index.
τ_f : final in situ soil suction .
τ_o : initial in situ soil suction.

$$Sp = C_h \Delta h\, f\, s$$ **McKeen's Model (1992)**

Where:
C_h: suction compression index.

$$= -0.02673\left(\frac{\Delta h}{\Delta w}\right) - 0.388704$$

Δw : change in soil water content
Δh : total suction change.
f : lateral restraint factor($0.5<f<0.8$ for clays).
s : load effect coefficient (typically, s = 0.9).

$$Sp = 0.022\left(\frac{\gamma_d}{\gamma_w}\right)^2 \log\left(\frac{h_s^o}{h_s^f}\right)$$ **Hafez's Model (1994)**

Where:
γ_d : dry unit weight of soil.
γ_w : unit weight of water.
h_s^o : initial soil suction head.
h_s^f : final soil suction head.

Fig.(5): Summary for the most famous advanced empirical formulas to estimating swelling potential (SP) [2]

C. Earlier methods to predict heave

Predicting amount of heave is one of main goals of studying swelling soils. Earlier researchers used experimental results to form empirical formulas to estimate the heave based on basic soil parameters such as consistency limits, water content and clay content. Some of the most famous empirical formulas to predict heave value are Vijayvergiya and Sullivan's formula (1973), Schneider and Poor's formula (1974), Johnson's formula (1978) and Weston's formula (1980) as shown in fig(3). Where heave equals swelling layer thickness multiplied by swelling potential. Although those formulas are easy to apply and require only the basic soil properties but they have wide range of error (about 35%).

This approach was developed by using more advanced laboratory tests to enhance the accuracy of the empirical formulas. Some examples of those formulas are McKeen and Lytton's Correlation (1981), McKeen's Model (1992), Hafez's Model (1994) as shown in Fig.(5) Although those formulas are more accurate but they are still data regression without scientific base.

Another approach to estimate heave value is analytical methods which depends on principals of soil mechanics and uses laboratory measured specified parameters to calculate heave. Based on the measured parameters, those methods could be classified into two types, methods depended on constant volume odometer test and methods depended on soil suction test (controlled suction odometer test).

Methods that depend on constant volume odometer test, calculate the heave based on the measured swelling index assuming that the initial stress state is the corrected swelling pressure and the final stress state is the effective vertical stress. The basic formula is show in Fig.(6).

Methods that relied on soil suction test calculate the anticipated heave based on the measured swelling and compressibility indexes using principals of unsaturated clay behavior shown in Fig.(7)

Increasing the computational capacities of computers allow recent researches to use more sophisticated techniques such as coupled and uncoupled finite element models to predict the heave.

Studying the stress state parameters of

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$$\Delta H = \frac{H_s}{1+e_0}\left[C_s \Delta \log\left(\frac{\sigma_f'}{\sigma_{sc}'}\right)\right]$$

Where:

H_s : depth of soil layer
e_0 : initial void ratio
C_s : swelling index
σ_f' : final effective stress
σ_{sc}' : Corrected swelling pressure

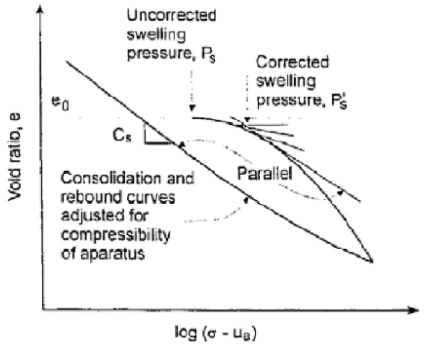

Fig.(6): Basic formula for the constant volume odometer test based methods [5]

$$\Delta H = \frac{H_s}{1+e_0}\left[C_m \Delta \log(u_a - u_w) + C_s \Delta \log(\sigma - u_a)\right]$$

Where:

u_a-u_w : soil suction.
σ-u_a : net normal stress.
e_o : initial void ratio
C_m : volume change index with respect to soil suction.
C_s : volume change index with respect to net normal stress.

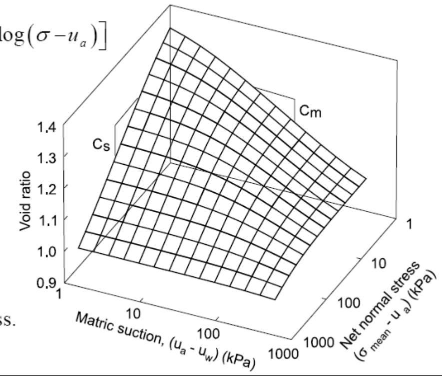

Fig.(7): Basic formula for the soil suction test based methods [6]

$$e = e_o - C_t \log\frac{(\sigma - u_a)}{(\sigma - u_a)_o} - C_m \log\frac{(u_a - u_w)}{(u_a - u_w)_o}$$

$$w = w_o - D_t \log\frac{(\sigma - u_a)}{(\sigma - u_a)_o} - D_m \log\frac{(u_a - u_w)}{(u_a - u_w)_o}$$

Where:

e_o : initial void ratio
C_t : volume change index with respect to net normal stress
C_m : volume change index with respect to matric suction
$(\sigma - u_a)_o$: initial net normal stress
$(u_a - u_w)_o$: initial matric suction
w_o : initial water content
D_t : water content index with respect to net normal stress
D_m : water content index with respect to matric suction

Fig.(8): Basic formulas change in volume and water content due to both effective stresses and soil suction [2]

unsaturated clays started in the 1960's and 1970's with Bishop, (1959); Aitchison, (1961); Jennings, (1961); Richards, (1967); Aitchison, (1973); Fredlund and Hasan, (1979), their researches indicated that negative pore water pressure (soil suction) is independent from effective stresses, which means that volume change is the sum of two independent phenomena, effective stresses and soil suction, which also means that there are two volume change indexes one with respect to effective stresses (Ct) and the other with respect to soil suction (Cm).

On the other hand, water content change also will be affected by the two phenomena, which means that there are other two indexes to describe the water content change with respect to effective stresses and soil suction which are (Dt),(Dm) respectively. Those four indexes could be measured by laboratory tests or estimated using many available empirical formulas. Basic formulas for both phenomena are shown in Fig. (8).

The independency of the two governing phenomena allows for two modeling concepts, the first is to model the two phenomena in the same model and solve them simultaneously which called Coupled Model, this concept requires complicated and nonstandard elements that can simulate seepage and nonlinear elasto-plastic behavior as well, as very powerful computers. The other concept is to model each phenomenon as separate model and use the output of one of them as an input to the other which is called Un-coupled model. Usually the output water pressure distribution from soil suction model is used as input for the effective stresses model. This concept uses standard seepage elements in soil suction model and standard nonlinear elasto-plastic element in effective stresses model, and since the two models are solved one by one, the common computers are sufficient enough.

II. PROPOSED FORMULA TO PREDICT HEAVE

Although finite element methods are more advanced and consider the non-linearity and variation in soil proprieties, and can handle distribution and flow of ground water but they need special field and laboratory test to measure certain parameters, besides, they need special software and are too complicated to be applied commercially.

The proposed formula belongs to the analytical methods that depend on constant volume odometer test. To calculate the heave of a homogenous and isotropic swelling clay layer infinity thick starts at ground surface without ground water table, first the critical depth (Hc) (or sometimes called active depth) have to be determined. At critical depth, the effective vertical stresses equals to the swelling pressure. Below the critical depth, the soil will not heave. Then critical depth is divided into 20 equal thick sub-layers, each layer is subjected to upward stress equals to the swelling pressure. The heave of each sub-layer is calculated based on the formula shown in Fig. (6).

$$\Delta h = \sum_{1}^{20} \Delta hi = \sum_{1}^{20} \frac{Cs\,.hi}{1+eo}\log\left(\frac{\gamma H}{\gamma Hc}\right)$$

$$= \frac{Cs\,.Hc}{1+eo}\sum_{1}^{20}\frac{1}{20}\log\left(\frac{H}{Hc}\right)$$

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$$= \frac{Cs\,.Hc}{1+eo}\frac{1}{20}\left[\log\frac{1}{20}+\log\frac{2}{20}+\log\frac{3}{20}+\cdots+\log\frac{20}{20}\right]$$

$$\text{Heave at ground surface }(\Delta h) = \frac{0.434\ Cs\,.Hc}{1+eo}$$

The ratio between total heave at any sub-layer (n) at depth (H) within the critical depth and total heave at ground surface could be calculated as the summation of the heave from sub-layer (n) and down to sub-layer (1) divided by the total heave at ground surface, which could be simplified by logarithmic regression to 0.25 Ln(Hc/H). Due to the simplification, H is limited between (0.02 Hc to 1.0 Hc). The total heave at any depth is the total heave at ground surface multiplied by this ratio.

$$\frac{\text{Total heave at sub-layer n}}{\text{Total heave at ground}} = \frac{1}{0.434 \times 20}\sum_{1}^{n}\log\frac{n}{20}$$

$$\approx \frac{1}{4}\,Ln\left(\frac{Hc}{H}\right)$$

At ground surface, Ln(Hc/H) equals 4.0, hence, H must not be less than 0.02 Hc.

$$\text{Total heave at depth H} \approx \frac{Cs\,.Hc}{9(1+eo)}\,Ln\left(\frac{Hc}{H}\right)$$

For limited thickness swelling clay with top surface at depth (Ht) and bottom surface at depth (Hb) from ground surface, the total heave of this layer (Δh)is the difference between the total heave at depths (Ht) and (Hb) as follows:

$$\Delta h \approx \frac{Cs\,.Hc}{9(1+eo)}\left[Ln\left(\frac{Hc}{Ht}\right) - Ln\left(\frac{Hc}{Hb}\right)\right]$$

$$\approx \frac{Cs\,.Hc}{9(1+eo)}\,Ln\left(\frac{Hb}{Ht}\right)$$

The formula could be simplified by substitute Hcwith(Ps-q)/γ, where Ps is the corrected swelling pressure, q is the surface surcharge and γ is the unit weight of soil.

$$\Delta h \approx \frac{Cs\,.(Ps-q)}{9\,\gamma\,(1+eo)}\,Ln\left(\frac{Hb}{Ht}\right) \qquad \ldots\ldots\ldots(1)$$

Where: $Ht \geq 0.02\ Hc$, $Hb \leq Hc$ & $Hc = (Ps-q)/\gamma$

If the odometer test results is not available, Cs equals to (1/6 to 1/10) Cc (compression index), and Cc is ranged between (0.007 - 0.009).(LL-10) according to over-consolidation ratio of the clay [7], where LL is the liquid limit percent (70% to 90% for most swelling clays). Hence, Cc is ranged between (0.06 to 0.13)LL , where LL is decimal fracture. Skempton (1953) suggested three classes of clay: inactive for activities less than 0.75; normal for activities between 0.75 and 1.25; and active for activities greater than 1.25. Typical values of activities for different clay minerals are as shown in Table (1)

Table (1): Typical Values of Activity for Clay Minerals [2]

Mineral	Activity
Halloysite (4H_2O)	0.10
Halloysite (2H_2O)	0.50
Kaolinite	0.33 – 0.50
Ilitte	0.50 – 1.00
Attapulgite	0.50 – 1.20
Altophane	0.50 – 1.20
Montmorillonite (Ca)	1.50
Smectities	1.00 – 7.00
Montmorillonite (Na)	7.20

Based on Skempton (1953), activity (A) for most minerals is ranged between (0.75 - 1.50). Hence Cs = (LL. A)/ 11

$$\Delta h \approx \frac{A\,.LL\,.(Ps-q)}{100\,\gamma\,(1+eo)}\,Ln\left(\frac{Hb}{Ht}\right) \qquad \ldots\ldots\ldots(2)$$

Where: $Ht \geq 0.02\ Hc$, $Hb \leq Hc$ & $Hc = (Ps-q)/\gamma$

Also, swelling pressure could be measured experimentally or estimated using any empirical formula listed in Fig. (3).

III. VERIFICATIONS OF PROPOSED HEAVE FORMULA

A. Case History of a Slab on Grade Floor on Regina Clay [5],[6]

The heave of a floor slab of a light industrial building in north central Regina, Saskatchewan is monitoring and analyzed by Yoshida et al., (1983) using analytical method based on constant volume odometer test, and reported and analyzed by Fredlund and Hung, (2004) using un-coupled finite element model. Construction of the building and instrumentation took place during August 1961. Instrumentation installed at the site included a deep benchmark, vertical movement gauges, and a neutron moisture meter access tube. Vertical ground movement was monitored at depths of 0.58, 0.85, and 2.39 m below the original ground surface

The owner of the building noticed heave and cracking of the floor slab in early August 1962, about a year after construction. An unexpected increase in water consumption of approximately 35000L was recorded. The line of hot water was cracked under the floor slab. Laboratory analysis for samples at the site was performed.

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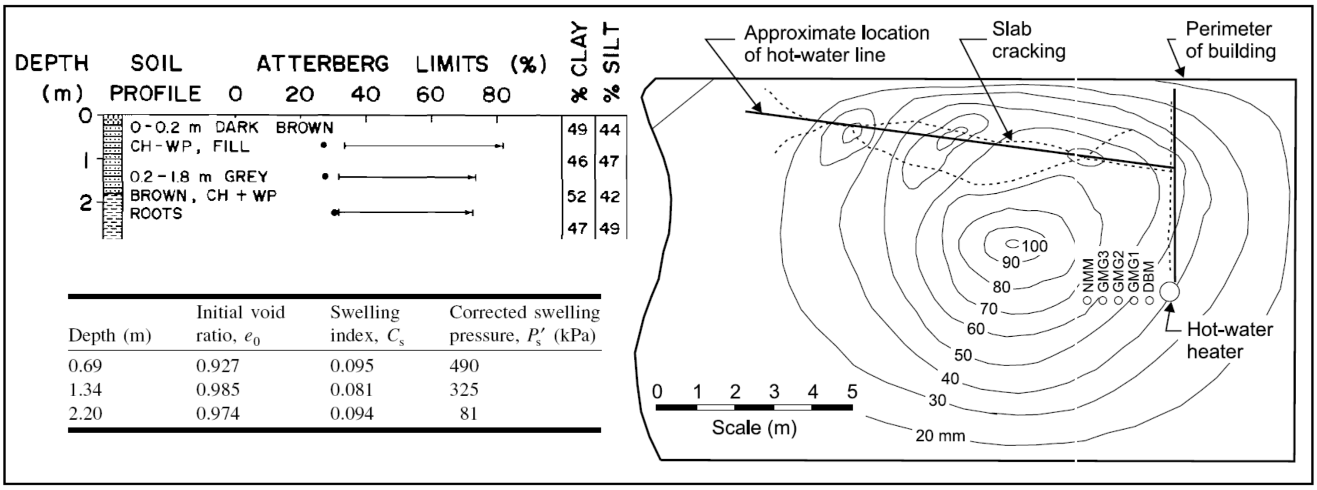

Depth (m)	Initial void ratio, e_0	Swelling index, C_s	Corrected swelling pressure, P_s' (kPa)
0.69	0.927	0.095	490
1.34	0.985	0.081	325
2.20	0.974	0.094	81

Fig.(9): Data summary for case history of a slab on grade floor on Regina clay [5]

Atterberg limits, in-situ water content, grain size distribution and swelling pressure of samples were evaluated. Swelling pressure and swelling index were obtained by constant volume odometer test for three samples. Location of the cracks, contours of the heave, summary of Atterberg limits and odometer tests results are shown in Fig. (9).

Surcharge is the weight of 100mm thick concrete slab on grade and 180mm thick sand layer beneath it. Average values of soil parameters are:

Unit weight =	1.89 t/m^3
Initial void ratio =	0.962
Liquid limit =	0.77
Plastic limit =	0.33
Clay content =	0.50
Soil activity =	1.14
Swelling pressure =	32.5 t/m^2
Surcharge =	0.57 t/m^2
Swelling index =	0.09
Top level of clay =	-0.35 m
Bottom level of clay =	-2.30 m
Critical depth =	17.0 m

The calculated heaves at ground surface using equations (1) & (2) are 160mm and 140mm respectively, where the measured value is 105mm.

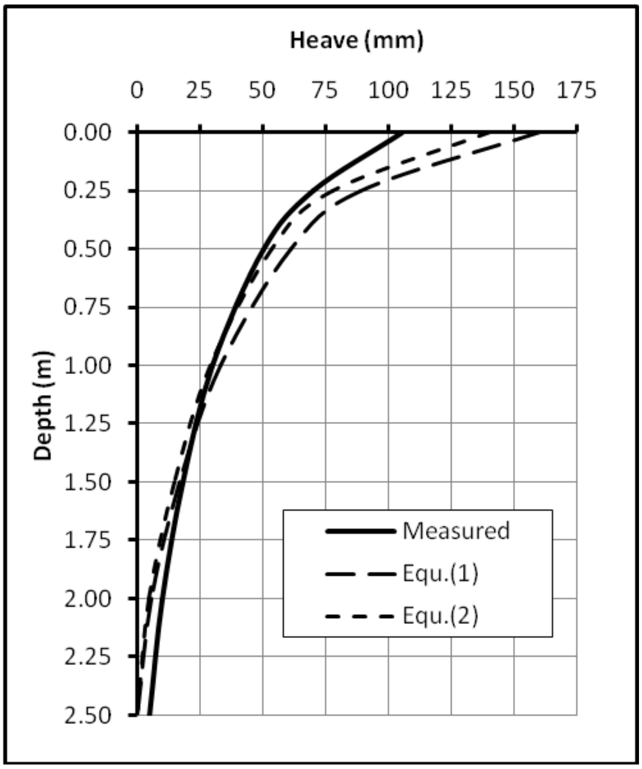

Fig.(10): Predicted and measured heave values with depth

B. Simple Heave Problem (Fredlund and Rahardjo, 1993) [2]

Fredlund and Rahardjo (1993) conducted an analysis to evaluate the total heave of expansive soil layer due to change in soil suction. In this analysis, a 2.0 m thick layer of expansive clay was subjected to change in soil suction due to covering with an impermeable layer of asphalt as shown in Fig.(11). The initial void ratio, eo, of the soil is 1.0, the total unit weight is 1.80 t/m3, and the swelling index, Cs is 0.10. Only one odometer test was performed on a sample taken from a depth of 0.75 m.

The test data showed a corrected swelling pressure, Ps, is 20.0 t/m2. It was assumed that the corrected swelling pressure is constant throughout the 2.00 m layer.

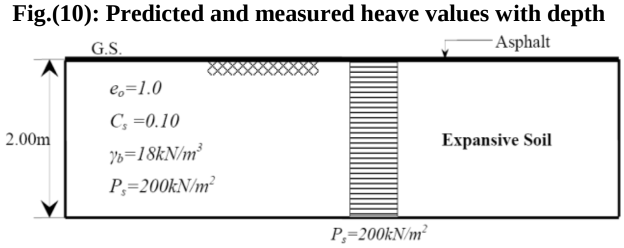

Fig.(11): Data summary for Simple Heave Problem (Fredlund and Rahardjo, 1993)

It is anticipated that with time, the negative pore-water pressure in the soil below the asphalt will increase as a result of the discontinuance of evaporation and evapotranspiration. For analysis purposes, it was assumed that the final pore-water pressure will increase to zero throughout the entire depth. Calculations performed by Fredlund showed a

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total heave of 114 mm, approximately 36% of the total heave occurs in the upper quarter of the clay strata.

Used values of soil parameters are:

Unit weight =	1.8 t/m^3
Initial void ratio =	1.0
Top level of clay =	-0.25 m
Bottom level of clay =	-2.00 m
Swelling pressure =	20 t/m^2
Surcharge =	0.0 t/m^2
Swelling index =	0.10
Critical depth =	11.0 m

The calculated heave at ground surface using equation (1) is 128 mm, 33% of the heave occurs in the upper quarter of the clay strata.

IV. ESTIMATING REPLACEMENT DEPTH TO CONTROL HEAVE

Soil replacement is very common technique to control the swelling behavior of shallow expansive clay layers. It is based on replacing the surface layer of expansive clay with non-swelling, well compacted granular mixture. Granular layer (may called sand cushion) reduces heave effect by two ways, first, it reduces the total amount of heave by reducing the depth of the swelling layer, second, it acts as porous filter and distribute any leaked water on a large area to reduce the differential heave. Although it is a very common technique, there is no reliable formula to estimate the required replacement depth. Most design codes leave this issue to the engineering judgment or give some general guide lines.

Since excessive settlement and excessive heave have the same effect on the structures above, allowable values for total and differential movements could be used for both of them. Unless otherwise specified, most design codes allows for 50mm to 100mm total settlement (or heave) and slope of (1/150 to 1/750) for differential settlement (or heave) according to the sensitivity of the structure.

The proposed method to estimate the replacement depth required to maintain the heave within the acceptable range is based on using the developed equations (1),(2) to determine the depth of the top surface of the swelling soil (Ht) that develops the allowable heave value (Δh).

Depth of the top surface of the swelling soil (Ht) could be calculate by rearranging equations (1), (2) to get equations (3), (4)

$$Ht = \frac{Hb}{e^{\left(\frac{9\ \Delta h\ \gamma\ (1+eo)}{Cs\ .(Ps-q)}\right)}} \quad \text{..........} \ (3)$$

$$Ht = \frac{Hb}{e^{\left(\frac{100\ \Delta h\ \gamma\ (1+eo)}{A\ .\ LL\ .(Ps-q)}\right)}} \quad \text{..........} \ (4)$$

Where:

Hc, Maximum active depth = (Ps-q)/γ

Ht, Depth of the top surface of the swelling soil ≥ 0.02 Hc

Hb, Depth of the bottom surface of the swelling soil ≤ Hc

V. VERIFICATIONS OF PROPOSED REPLACEMENT DEPTH FORMULA

A. Parametric study by Youssef (2010) [2]

Youssef carried out a parametric study for the effect of sand cushion on soil settlement and soil heave was performed using uncoupled model. The study was performed under the effect of climate conditions with 3.00 m seasonal moisture fluctuation zone depth and 1.50pF soil suction change at ground surface. Parametric study included the effect of different sand cushion parameters such as; depth, lateral extension, and relative density of sand cushion. Regina clay properties are shown in Fig.(12).

The loading are applied in two stages. The footing pressure was applied in first stage and soil suction change was applied in the second stage. In second stage, final soil suction is assumed to be hydrostatic with soil suction value of 3.2 pF (150kPa) at ground surface which simulates wet conditions in winter. The reported results are predicted at the midpoint of footing width.

Comparison between predicted heave using equation (2) and after Youssef for different replacement depths and different surcharge are summarized in table (2).The results shows good matching at light surcharge and the difference increases with increasing the surcharge because the original study carried out by Youssef considered 2.0m width strip foot, while equation (2) considered infinity width surcharge.
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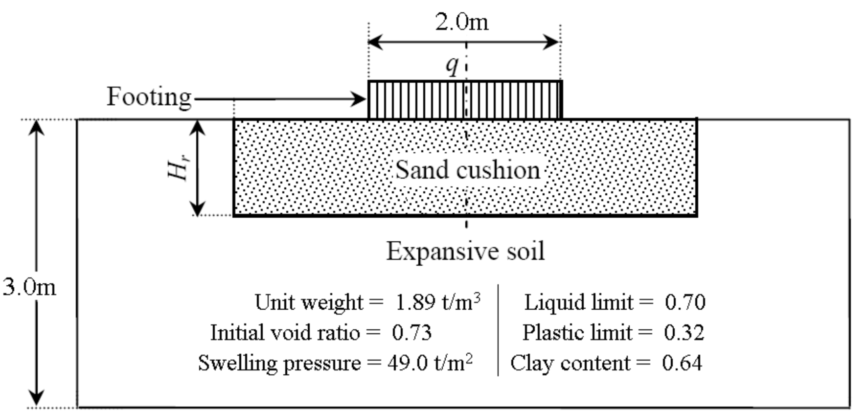

Fig.(12): Data summary for Youssef (2010) parametric study

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Table (2): Comparison between predicted heave using Eq. (2) and after Youssef for different replacement depths and different surcharge

Sur-charge (t/m^2)	Total heave at ground surface (mm)							
	Youssef (2010) Replac. depth (m)				Proposed Equ. (2) Replac. depth (m)			
	0.5	1.0	1.5	2.0	0.5	1.0	1.5	2.0
0	104	77	52	30	111	69	44	25
2	100	76	51	30	109	67	42	24
4	97	74	50	29	104	64	40	23
6	95	72	49	29	100	61	38	22
10	90	69	48	28	90	55	35	20
14	85	66	46	28	81	49	31	18

B. Pump station at Wadi-Elsaida, Edfo, Egypt

A water pumping station was constructed in Wadi-Elsaida site, Upper Egypt in 1990. Project location is an urban desert area, soil profile was stiff dry clay starting from ground surface and down to the end of borings, and there was no ground water table. Laboratory tests on clay samples showed a high swelling potential. For such water structure it was very hard to prevent surface leakage to the swelling soil. The allowable total and differentials heaves (or settlement) value was limited by 80mm due to piping, mechanical and equipment tolerances. In order to maintain the heave within that limit, the author depended on three precautions, first, using a rigid concrete raft foundation to resist the effect of differential heave on the structure, second, replacing the upper 6.0m of the swelling clay with non-swelling, well compacted granular mixture, and finally using 300 mm thick reinforced concrete mat at the middle of the replacement layer to act as cut off for any leaked water and to increase the heave (or settlement) uniformity. A Plaxis finite element model was used to verify the validity of the precautions. 25 years after construction, the station didn't suffer any ground movements. The soil parameters used in the analysis are:

Unit weight =	2.20 t/m^3
Initial void ratio =	0.75
Liquid limit =	0.85
Plastic limit =	0.35
Clay content =	1.0
Soil activity =	2.0
Swelling pressure =	40.0 t/m^2
Surcharge =	12.5 t/m^2
Allowable heave =	0.08 m
Critical depth =	12.5 m
Top level of clay =	-0.25 m
Bottom level of clay =	-12.5 m

Using equation (4), the required replacement depth to maintain the total heave below 80mm is 6.50m which matches the original design.

VI. CONCLUSIONS

The results of this research could be concluded as follows:

1- Heave of swelling soil is complex action includes two phenomena, volume change due to soil suction and volume change due to effective stresses.

2- Heave value could be predicted analytically based on constant volume odometer test or suction test, and numerically used coupled or un-couples finite element models.

3- The research proposed two equations to predict heave value analytically based on the available data as follows:

- If odometer test results is available:

$$\Delta h \approx \frac{Cs\,.(Ps-q)}{9\,\gamma\,(1+eo)}\,\mathrm{Ln}\left(\frac{Hb}{Ht}\right)$$

- If odometer test is not available

$$\Delta h \approx \frac{A\,.LL\,.(Ps-q)}{100\,\gamma\,(1+eo)}\,\mathrm{Ln}\left(\frac{Hb}{Ht}\right)$$

Where: $Ht \geq 0.02\ Hc$, $Hb \leq Hc$ & $Hc = (Ps-q)/\gamma$

4- Those two proposed equations could be rearranged to calculate the required replacement depth to maintain heave value within accepted range as follows:

- If odometer test results is available:

$$Ht = \frac{Hb}{e^{\left(\frac{9\,\Delta h\,\gamma\,(1+eo)}{Cs\,.(Ps-q)}\right)}}$$

- If odometer test results is available:

$$Ht = \frac{Hb}{e^{\left(\frac{100\,\Delta h\,\gamma\,(1+eo)}{A\,.\,LL\,.(Ps-q)}\right)}}$$

Where: $Ht \geq 0.02\ Hc$, $Hb \leq Hc$ & $Hc = (Ps-q)/\gamma$

5- Verifications using case studies shows good matches with proposed equations.

6- Further studies should be carried out to determine the minimum lateral distance that replacement layer should be extended beyond foundation edge to control the heave at foundation perimeter.

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AUTHOR'S PROFILE

Hisham Arafat. He received his Ph.D. in soil mechanics from Ain-Shams Uni.-Egypt in cooperation with Braunschweig Uni.-Germany in 1998. He is high ways & bridges consultant since 2000, He published 12 researches is geotechnical engineering, soil mechanics, bridges maintenance and repair and construction management. Presently, he is the head of structural department, faculty of engineering, Future Uni.-Egypt.

Ahmed M. Ebid. He received his Ph.D. in soil mechanics from Ain-Shams Uni.-Egypt 2004.He is a consultant in designing concrete structures since 2012, He published 5researches is geotechnical engineering, soil mechanics, repairing using FRP and optimizing the design of concrete elements. Presently, he isa lecturein structural department, faculty of engineering, Future Uni.-Egypt.