Rational Approach to Predict Swelling Soil Behavior

T. S. NAGARAJ and B. R. SRINIVASA MURTHY

ABSTRACT

The development of swelling pressure, heave, or collapse on wetting due to the changes in the state of soil moisture are some of the most important engineering problems normally encountered with partly saturated soils. A phenomenological model has been formulated with truncated diffuse double layer theory as the scientific base to predict swelling pressure. It defines fully the swelling behavior of soil in terms of four parameters: (a) swollen void ratio, e_s; (b) swelling pressure, p_s; (c) preconsolidation pressure, p_c; and (d) the slope of the line joining the present state to preconsolidation pressure, ρ. It involves four equations in terms of initial void ratio, e_o, void ratio at liquid limit water content, e_L, and effective overburden pressure, p. For a range of values of e/e_L and p, the swelling pressure values have been computed by iteration, and the results are presented in graphic form. A good agreement between predicted and experimental published data emphasizes the applicability of this approach.

Most of natural soil systems above groundwater table, and invariably all the compacted soils in their initial conditions, are in a state of partial saturation. Further, the mechanics of partly saturated soils is of considerable interest in all the areas of the world where there is an annual excess of potential evaporation over precipitation. Even in areas where this condition does not apply, seasonal moisture deficits may occur. It is apparent that far more than one-half of the earth's surface is a moisture-deficit area. The majority of problems encountered in partly saturated soils are a result of the changes in the state of soil moisture and its associated changes in soil state, compatible with external stress conditions.

The mobilization of swelling pressure and heave or collapse of the soil due to the changes in the state of soil moisture is an important engineering problem. According to a general worldwide assessment, it has been estimated that damages due to expansive soils exceed those due to combined damages from floods, hurricanes, earthquakes, and tornados. The need for prevention or at least minimization of the vast damages caused by swelling soils in geotechnical engineering practice has prompted intensive research to understand, interpret, and predict the behavior of swelling soils. Two approaches are pursued: (a) empirical methods in which thorough statistical analysis of experimental data-simple working models are developed, and (b) rigorous scientific approaches based on the fundamentals of soil behavior to unravel the complexity for meaningful interpretation and generalization.

The intent of this investigation is to examine the possibility of providing simple links, both behavioral and parametric, between the preceding approaches for direct practical application by using only normally measured and observed parameters.

BACKGROUND INFORMATION

Attempts have been made to either link swelling behavior of soils directly with the Gouy-Chapman diffuse double layer theory (1-4) or to evolve statistical empirical relationships between involved and/or inferred soil state parameters (5-10).

There are several reports on the swelling properties of clays that are significant studies in soil engineering. Seed et al. and Saito presented experimental studies on the relationship between the swelling properties and consistency or activity of compacted clays (6,8).

Komornik and David, on the basis of the Gouy-Chapman diffuse double layer theory, logically involved the liquid limit of the soil as the needed parameter to define swelling behavior (7). On the basis of extensive test data they generated an empirical interrelationship in the form

$$\log p_s = \bar{2}.132 + 0.0208\ (w_L) + 0.000665\ (\gamma_d) - 0.0269\ w \quad (1)$$

where

p_s = swelling pressure in Kg/cm^2,
w_L = liquid limit in percentage,
γ_d = natural dry density in Kg/m^3, and
w = natural water content in percentage.

The foregoing interrelationship has a poor coefficient of multiple correlation of less than 0.6. Komornik and David have also indicated a relationship between percent swell and plasticity index.

Vijayavergiya and Ghazzaly, on the basis of 270 test results of undisturbed natural soils at shallow depths, proposed two correlations by which the swell pressure and percent swell under 0.1 ton/ft^2 can be predicted (9). The interrelationships have been indicated in the form

$$\log p_s = 1/12\ (0.4\ w_L - w - 0.4) \quad (2)$$

$$\log p_s = 1/19.5\ (\gamma_d + 0.65\ w_L - 139.5) \quad (3)$$

$$\log S = 1/12\ (0.4\ w_L - w + 5.5) \quad (4)$$

$$\log S = 1/19.5\ (\gamma_d + 0.65\ w_L - 130.5) \quad (5)$$

where S is the percent swell at 0.1 ton/ft^2 and γ_d is the dry density of the soil in lb/ft^3. These interrelationships have a correlation coefficient of about 0.7.

Chen, after conducting an extensive study on expansive soils (11), concluded that the swelling pressure of a clay is independent of the surcharge pressure, initial moisture content, degree of saturation, and the thickness of the stratum. The swelling pressure increases with the increase in initial dry density of the soil. Snethan et al. conducted an investigation of the natural microscale mechanisms that cause volume change in expansive clays (10). They stated that "although some interesting trends may be inferred using this approach, it is still not successful to infer the microscale mechanisms from observed macroscopic volume change behaviour." It has been concluded that clay particle attraction and cation hydration microscale mechanisms play the greatest role in causing volume change. At higher moisture contents and higher cation concentration environments, the osmotic repulsion mechanism provides a secondary influence on volume change behavior.

Bolt and Warkentin et al., using the diffuse double layer model, confirmed experimentally the theory of particle repulsion for pure clays (1,2). The swelling pressure is attributed to the osmotic repulsive pressure mobilized due to the imbibition of water. This hypothesis differs from the mechanistic approach, which attributes the imbibition of water to a hydrostatic gradient including flow from the high-pressure zone of the solution to the low-pressure capillary zone between the particles. Komornik and David have indicated that both the mechanisms operate simultaneously in the soil system (7). They have also indicated that the specific surface of the soil being reflected by its liquid limit water content accounts for the osmotic repulsion component and the dry density or void ratio accounts for the capillary suction.

Lambe explained the swelling phenomena of compacted clays from a physicochemical point of view (12). Tsytovich estimated the amount of swelling of clays using a modified consolidation theory (13). Nishida (14) and Nishida et al. (4), on the basis of the Gouy-Chapman diffuse double layer theory, studied the compression index of clay. By extending the diffuse double layer theory, an equation for swelling pressure of clays of low activity has been presented. It has been revealed that the swelling of clays is proportional to the plasticity index and the amount of free swell is proportional to the specific surface of the clay.

Dennisov indicated that the swelling pressure of a soil depends on the overconsolidation and the composition of the soil (15). In an overconsolidated state, all soils are liable to swell after the natural cementation bonds, if any, are broken.

The preceding discussion on the subject, although incomplete, indicates that swelling behavior of soils is yet to be understood. The parameters that affect swelling behavior are quite varied.

Considering the physicochemical interaction forces, the compressibility of the fine-grained saturated soils has been modeled in the stress range of 25 to 800 kPa in the form

$$e/e_L = 1.122 - 0.2343 \log p \quad (6)$$

$$e/e_L = 1.122 - 0.188 \log p_c - 0.0463 \log p \quad (7)$$

for normally and overconsolidated states (16). ["Prediction of Compressibility of Overconsolidated Soils" (authors' communication to Geotechnical Division, ASCE).] The term e/e_L has been referred to as the generalized soil state parameter. The possibility of extending the foregoing considerations to define partly saturated soil behavior merits further consideration. As a first step, the physicochemical state of a partly saturated fine-grained soil is discussed. Further, the possibility of generating a phenomenological model to predict swelling soil behavior is examined.

PHYSICOCHEMICAL STATE OF PARTLY SATURATED SOIL

It is a well-known phenomena that in a clay water electrolyte system, the electronegativity of the surface results in the formation of an electric diffuse double layer. The concentration of ions and co-ions in the diffuse double layer varies from maxima and minima at the particle surface to a value equal to that of bulk solution at a far-off point, roughly in an exponential form. With the progressive removal of moisture from a clay electrolyte suspension, a stage will be reached when the thickness of the water film is less than the diffuse double layer. This may be achieved by moisture removal due to evaporation or by drainage under an applied stress. As the ions are forced to remain in the liquid phase, the extent of the double layer cannot exceed the thickness of the liquid layer in contact with the charged surface. In such an event all the ions present in the suspension are forced into the liquid phase causing a readjustment of the ionic concentration distribution. This condition has been schematically shown in Figure 1. Such a layer is termed by Bolt and Bruggenwert as the truncated diffuse double layer (17).

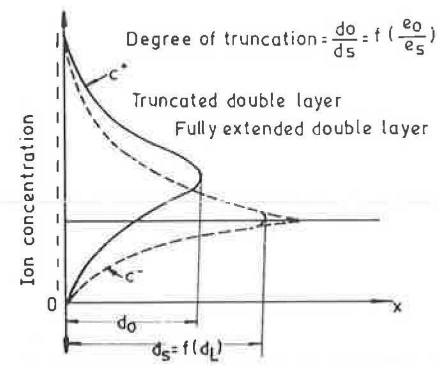

FIGURE 1 Schematic representation of redistribution of ions in a truncated double layer.

The special property of the truncated double layer is to reabsorb moisture forcefully and to develop to an extent commensurate with the concentration of the equilibrium bulk solution. In this process, the double layer reaches a stage at which all the voids are saturated with exertion of swelling pressure for no change in void ratio. If the volume change is permitted, the double layer grows to its full extent. Figure 2 schematically shows the truncated double layer due to partial saturation, the interacting double layer on saturation in equilibrium under swelling pressure, and the fully extended double layer under no load.

PHENOMENOLOGICAL MODELING

From the considerations of the truncated diffuse double layer theory, the compressibility behavior of partly saturated, fine-grained soils has been defined in terms of the external applied stress p and the generalized soil state parameter for the partly

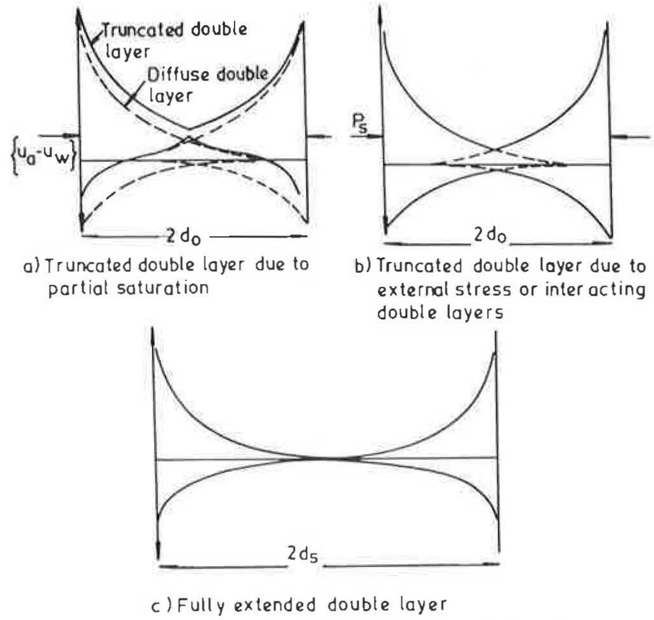

FIGURE 2 Schematic representation of ion distribution in truncated and fully extended diffuse double layer.

saturated state $e/e_L \sqrt{S_r}$ (18). ["Generalisation of Compressibility Behavior of Partly Saturated Soil" (authors' communication to Geotechnical Division, ASCE.)] According to Bolt and Bruggenwert, the swelling pressure is always a function of the degree of truncation of the diffuse double layer (17). In the present context, the degree of truncation can be explained as follows. In a constant volume swell pressure test the initial void ratio represents the degree of truncation. The system is in equilibrium under the influence of capillary stresses due to partial saturation and the physicochemical interaction stresses. On imbibition of water, the capillary stresses tend toward zero and the interacting stresses increase, resulting in the development of swelling pressure. At this state, the system can be in equilibrium at the same initial void ratio only under an external applied pressure equal to the swelling pressure. In the process, the truncation due to partial saturation transforms into the one due to saturation and under external pressure. On release of the pressure, the double layer grows to its full extent, compatible with the concentration of the bulk solution. This state at a nominal pressure of 10 kPa is represented as e_s.

Now $e_o = G\gamma_w S d_o \times 10^{-4}$ and $e_s = G\gamma_w S d_s \times 10^{-4}$

$$e_o/e_s = d_o/d_s$$

where

S = specific surface of the soil,
d_o = initial half-space distance,
d_s = final half-space distance in swollen state,
G = specific gravity of soil particles, and
γ_w = density of water.

Then $e_o/e_s = d_o/d_s$ represents the degree of truncation. As such, phenomenologically the equation for swelling pressure can be written as

$$p_s = f\,(d_o/d_s) = f\,(e_o/e_s) \tag{8}$$

When the soil system is in equilibrium in a partly saturated state under an external load, the volume change behavior of the soil system on saturation depends on the relative magnitudes of the existing pressure and the developed swelling pressure. If the swelling pressure is less than the existing external pressure, the system undergoes consolidation or collapse to reach a compatible state with the external pressure. If the swelling pressure is greater than the existing pressure, the soil swells to reach another equilibrium state (15).

Figure 3 schematically shows the initial partly saturated state in equilibrium under the external pressure p by point A, and on inundation, the swelling pressure p_s required to keep the system in equilibrium at the same initial void ratio by point B. Also shown in Figure 3 are the overconsolidated saturated line BC, the normally consolidated saturated line CE, and the swollen void ratio e_s along the rebound path CBD. It is evident geometrically from Figure 3 that $p_s = f\,(e_o/e_s)$.

Effect of Preconsolidation Pressure

The rebound of a saturated soil on unloading is proportional to the interacting repulsive pressure mobilized at the time of unloading, which is otherwise balanced by the external pressure in its equilibrium state. The repulsive pressure mobilization is a function of the interacting specific surface at any equilibrium state. The effective inter-

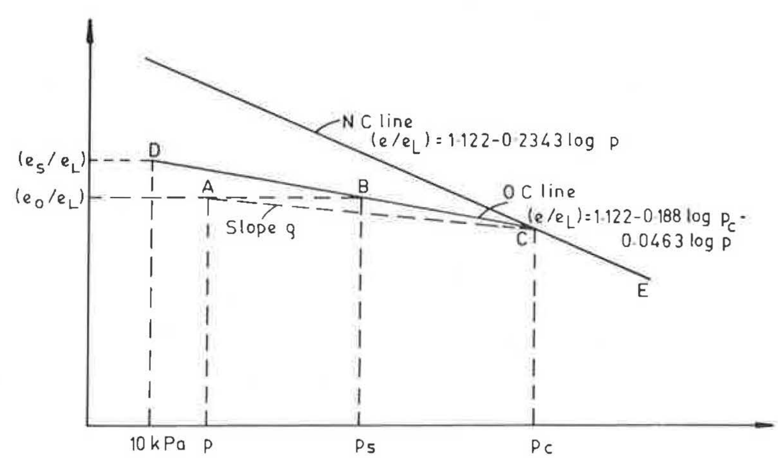

FIGURE 3 Schematic representation of states before and after saturation.

acting specific surface reduces with an increase in stress due to the formation of clusters, packets, and domains (16). Hence, the mobilized repulsive pressure at any equilibrium state depends on the stress level of rebound, that is, preconsolidation pressure p_c. It has been shown (authors' communication to Geotechnical Division, ASCE) that the saturated average rebound recompression lines of all fine-grained soils bear a unique slope value of $\rho = 0.0463$ in the (e/e_L) - log p plot (Equation 7). It can logically be expected that the swollen void ratios of all the soils bear a unique relationship with e_L and the preconsolidation pressure p_c.

Now in Figure 3, the point B represents the saturated equilibrium state under the external pressure equal to the swelling pressure along the average rebound recompression line. Thus the swollen void ratio at a pressure of 10 kPa can be obtained by substituting p = 10 kPa in Equation 7.

$$(e_s/e_L) \text{ @ } 10 \text{ kPa} = 1.08 - 0.188 \log p_c \quad (9)$$

or

$$(e_s/e_L) \text{ @ kPa} = f (\log p_c) \quad (10)$$

Now a combination of Equations 6 and 10 results in

$$p_s = f (e_o/e_L, \log p_c) \quad (11)$$

This is further substantiated by putting $p = p_s$ in Equation 7 at the same (e_o/e_L). It is apparent because at p_s, (e_o/e_L) is the same before and after saturation.

$$\text{That is, } (e_o/e_L) = 1.122 = 0.188 \log p_c - 0.0463 \log p_s \quad (12)$$

This implies that, for known values of p_c and (e_o/e_L), the swelling pressure of the soil can be predicted.

In any of the earlier approaches, either empirical or rational, the involvement of p_c, accounting for the change in the effective specific surface, is not indicated. The other two factors e_o and e_L represent similar terms d and w_L, used for prediction by various investigators.

Dennisov has indicated the need for the parameter p_c in the prediction of the swelling behavior of soils (15). He has stated that only overconsolidated soils can exhibit swelling. For a partly saturated soil at field conditions, existence of preconsolidation pressure is not apparent. It may be termed as an equivalent preconsolidation pressure. The present state of the soil is due to several cycles of alternate wetting and drying and the corresponding alterations in the capillary stresses. Parry and Bjerrum have stated that such a type of overconsolidation is identical to conventionally known stress-release type of overconsolidation (19-21).

Determination of Preconsolidation Pressure for a Partly Saturated Soil

For a given partly saturated soil, the preconsolidation pressure cannot be determined with any of the known methods. The following method is tentatively proposed. It is shown schematically in Figure 3 that the slopes of the line joining the present state to the preconsolidation pressure can define the preconsolidation pressure. The equation of this line can be written in the form

$$(e/e_L) = 1.122 - (0.2343-\rho) \log p_c - \rho \log p' \quad (13)$$

where (e/e_L) is the generalized soil state parameter at stress p' the value p' is between p and p_c or p' = p, corresponding to (e_o/e_L), the initial state,

$$(e_o/e_L) = 1.122 - (0.2343-\rho) \log p_c - \rho \log p \quad (14)$$

From the schematic diagram shown in Figure 3, it is clear that the slope ρ can be related to (e_o/e_L), p and p_s, which eliminates p_c.

$$\text{Thus, } \rho = f (e_o/e_L, p, p_s) \quad (15)$$

Now, phenomenologically developed equations can be listed as

$$p_s = f (e_o/e_s) \quad (16)$$

as well as Equations 10, 14, and 15

$$(e_s/e_L) = f (\log p_c)$$

$$(e_o/e_L) = 1.122 - (0.2343 - \rho) \log p_c - \rho \log p$$

$$\rho = f (e_o/e_L, p, p_s)$$

In the preceding four interrelationships, for known values of (e_o/e_L) and p, the other four quantities ρ, p_c, e_s, and p_s can be evaluated. Because with the present state of the art these forms of equations cannot be generated from the fundamental principles, recourse to a semiempirical method is taken. On the basis of a set of experimental data, generation of the type of equations cited earlier is attempted.

SEMIEMPIRICAL EQUATIONS

To generate the possible type of equations cited earlier, the extensive experimental data of Snethan et al. (10) are selected. Of the available 20 constant volume swell pressure test data, results of 13 tests, conforming to a high degree of final saturation, are selected. The data in Table 1 indicate the analyzed test data of Snethan et al. (10). The swollen void ratio e_s has been selected at a pressure of 10 kPa. The additional computed terms presented in Table 1 are p_c and ρ.

The term p_c is calculated from the equilibrium condition at p_s, corresponding to the same initial state (e_o/e_L) as per Equation 7. Then the (e/e_L) at p_c is calculated by using Equation 6 for the normally consolidated saturated state and, thus, the slope is computed using the equation

$$\rho = [(e_o/e_L) - (e/e_L \text{ @ } p_c)]/(\log p_c - \log p) \quad (17)$$

The tabulated results are statistically interrelated to obtain the functional relationships of Equations 8, 10, 14, and 15 in the form

$$(e_o/e_s) = 1.00571 - 0.0004036\, p_s \quad (18)$$

$$(e_s/e_L) = 1.068 - 0.1934 \log p_c \quad (19)$$

$$\rho = 0.0601 - 0.0297 [(e_o/e_L) + \log (p_s/p)] \quad (20)$$

with correlation coefficients of 0.97, 0.994, and 0.996, respectively. In the preceding equations p_s, p_c, and p are all in kPa.

Equations 18-20 together with Equation 14 will define completely the swelling behavior of soils. It is interesting to note that Equation 19 is similar to Equation 9 for p = 10 kPa.

$$\text{That is, } (e_s/e_L) = 1.076 - 0.188 \log p_c \quad (21)$$

TABLE 1 Collected and Collated Data *(10)*

Sl/No	Sl No in Ref	e_o	e_L	e_o/e_L	e_s	e_o/e_s	e_s/e_L	p_s in kPa	p_c in kPa	ρ	$(e_o/e_L)+\log(p_s/p)$
1		2	3	4	5	6	7	8	9	10	11
1	1	1.155	2.798	0.413	1.370	0.843	0.490	382	1364	0.0152	1.548
2	2	0.716	1.635	0.438	0.749	0.956	0.458	96	1411	0.0317	0.973
3	3	1.265	2.611	0.485	1.279	0.989	0.490	43	973	0.0407	0.671
4	4	0.686	1.523	0.450	0.690	0.995	0.453	35	1554	0.0437	0.547
5	6	0.421	0.962	0.438	0.424	0.993	0.441	66	1548	0.0364	0.810
6	7	0.544	1.320	0.412	0.553	0.984	0.419	65	2136	0.0373	0.778
7	10	0.727	1.517	0.479	0.755	0.963	0.498	93	861	0.0301	1.000
8	12	0.736	2.063	0.357	0.769	0.957	0.373	95	3825	0.0348	0.887
9	14	0.765	1.568	0.488	0.791	0.967	0.505	80	800	0.0318	0.944
10	15	1.125	1.739	0.647	1.133	0.993	0.652	38	760	0.0275	1.069
12	19	0.589	1.904	0.309	0.597	0.986	0.313	59	7686	0.0402	0.633
13	20	0.99	2.176	0.455	1.069	0.926	0.492	254	902	0.0169	1.413

e_o - Initial void ratio, e_L - void ratio at liquid limit, e_s - swollen void ratio upon saturation at 10 kPa, ρ - slope. The overburden pressure is constant at 28 kPa for all the tests.

The difference in the values of intercept and slope are negligible. The minor difference is due to the backward extrapolation of Equation 7 to a low stress level of 10 kPa. Because the computations of other equations are based on the test results of Snethan et al., Equation 19 is preferred.

For computing the swelling pressure, e_s is eliminated between Equations 18 and 19 resulting in

$$p_s\,(\text{kPa}) = 2492 - 12811.3\ (e_o/e_L)/(5.522 - \log p_c) \quad (22)$$

The other two equations are Equations 20 and 14:

$$\rho = 0.0601 - 0.0297\ [(e_o/e_L) + \log\ (p_s/p)]$$

$$(e_o/e_L) = 1.122 - (0.2343-\rho)\ \log p_c - \rho \log p$$

Equations 14, 20, and 22 have three unknowns in p_s, p_c, and ρ. It is not possible to simplify further to directly solve for p_s. In addition, Equations 14 and 22 are nonlinear. As such, solutions to these equations are obtained by the following iteration process. For known field values of (e_o/e_L) and the overburden effective pressure p, a value of swelling pressure p_s is assumed. For the assumed value of p_s and the known value of p, the slope of the line joining the partly saturated state point to the p_c point (Figure 3) is computed using Equation 20. With the computed value of ρ and known values of (e_o/e_L) and p, the preconsolidation pressure p_c is calculated using Equation 14. Using Equation 22 for a known value of (e_o/e_L) and a computed value of p_c, p_s is calculated. This procedure is repeated assuming a new value of p_s until this assumed value agrees with the finally computed value of p_s. The entire operation has been programmed to the DEC-10 system, and p_s values have been generated for the entire range of (e_o/e_L) and p values. The results are plotted in a semilog plot as (e_o/e_L) versus $\log p_s$ for various p values (Figure 4). For known values of (e_o/e_L) and p, the swelling pressure can be directly read from the graph. In Figure 4 a small region is indicated in which the soil is susceptible to collapse under an overburden pressure of 200 kPa. Similarly, for higher p values, zones of collapse may be marked.

VALIDITY OF THE APPROACH

To examine the validity of the approach, published experimental swell pressure data of Komornik and David are probed (7). Of the available data on 125 soils, only those for which all the relevant information, such as initial void ratio, overburden pressure, initial moisture content, final void ratio, and final degree of saturation, are available have been selected. Further, probing is restricted to soils conforming to identical initial and final void ratios, a high degree of final saturation, and a swelling pressure greater than the overburden pressure. For all such soils, the experimental and the predicted swelling pressure values have been indicated in Table 2. The comparison indicates a fair agreement.

There are a few cases in which the predicted value is higher than the experimental value. The reason for this behavior may be due to the presence of natural cementation or desiccated bonds that reflect lower experimentally determined swelling pressure. Similarly, a few cases for which the predicted values are lower than their respective experimental values are also indicated. The reason for such a discrepancy may be the result of deviations in the assessed values of overburden pressure.

In general, it may be stated that the swelling pressure of a soil may be predicted within the limits of accuracy at engineering level. Once the swelling pressure is predicted, all other swelling behaviors, such as percent swell, heave, or collapse, may be predicted.

CONCLUDING REMARKS

On the basis of the truncated double layer theory applicable to a partly saturated soil system, a

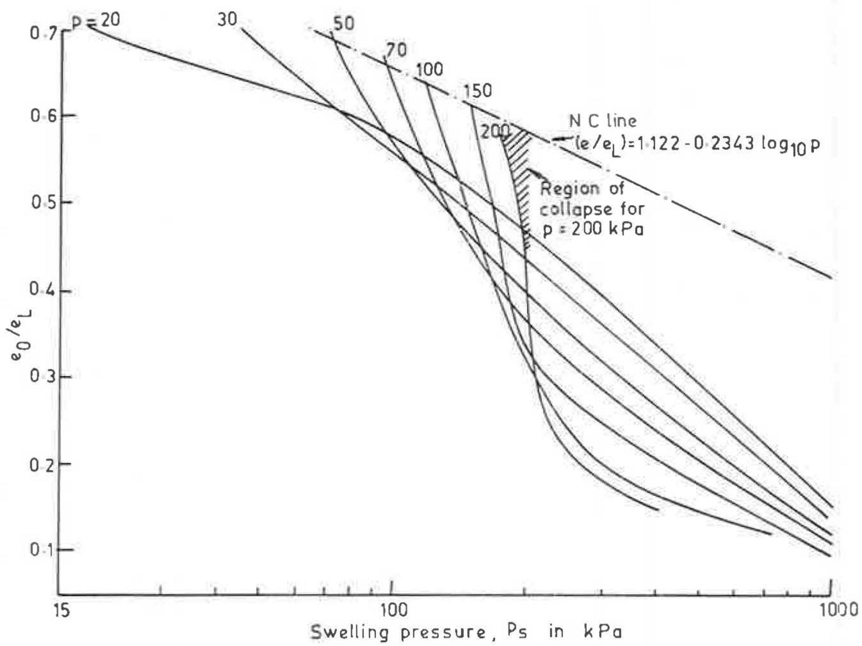

FIGURE 4 Prediction of swelling pressure of soils.

TABLE 2 Analysis of Data for Predicting Swelling Pressure *(7)*

Sl No / Sl No (Ref)	e_o	w_L	e_L	e_o/e_L	p in kPa	p_s (Expt) in kPa	p_s (Computed) in kPa
1	2	3	4	5	6	7	8
1/10	0.442	35	0.928	0.476	62	127	140
2/11	0.510	45	1.224	0.417	47	190	190
3/12	0.876	76	2.098	0.418	61	191	180
4/13	0.680	61	1.665	0.408	63	223	185
5/14	0.651	57	1.562	0.417	50	222	185
6/23	0.868	78	2.114	0.411	75	155	165
7/29	0.785	73	2.029	0.387	66	222	192
8/37	0.900	79	2.244	0.401	48	175	195
9/39	0.618	52	1.440	0.429	25	151	220
10/40	0.548	42	1.147	0.478	30	103	155
11/42	0.911	70	1.981	0.460	38	128	160
12/43	0.568	41	1.111	0.511	32	48	127
13/45	0.535	55	1.485	0.360	42	248	255
14/46	0.468	33	0.884	0.529	61	128	120
15/48	1.236	62	1.693	0.730	17	24	18
16/51	0.760	69	1.898	0.400	59	225	190
17/52	0.854	61	1.696	0.504	92	175	142
18/53	0.702	53	1.463	0.480	138	200	170
19/56	0.766	71	1.945	0.394	58	325	205
20/58	0.657	67	1.882	0.349	137	200	195
21/59	0.687	71	1.938	0.354	176	200	200
22/63	1.320	85	2.295	0.575	25	54	85
23/64	1.180	85	2.295	0.515	25	70	128
24/66	1.250	96	2.717	0.460	21	196	203
25/102	0.570	116	3.190	0.179	133	455	420
26/103	0.500	103	2.936	0.170	146	805	430
27/104	0.520	122	3.477	0.150	145	805	550
28/105	0.61	97	2.668	0.229	141	417	280
29/106	0.53	110	3.08	0.172	143	570	440

e_o - initial void ratio, w_L - liquid limit,
p, - effective over burden pressure in kPa,
p_s - swelling pressure in kPa

phenomenological model has been generated to define the swelling behavior of soils. Using experimental test data, a semiempirical model analogous to the phenomenological model has been developed. The semiempirical model involving (e_o/e_L), p, p_s, and p_c, has been solved for the possible range of values of (e_o/e_L) and p. The results have been represented in the form of (e_o/e_L) versus log p_s plot for various values of p. The p_s value can be predicted using the foregoing plot for known values of (e_o/e_L) and p. The validity of the approach has been substantiated in relation to the published data.

REFERENCES

1. G.H. Bolt. Physico-chemical Analysis of the Compressibility of Pure Clays. Geotechnique, No. 2, Vol. 6, 1956, pp. 86-93.
2. B.P. Warkentin, G.H. Bolt, and R.D. Miller. Swelling Pressure of Montmorillonite. Soil Science Society of America Proceedings, Vol. 21, 1957, pp. 495-497.
3. R.N. Yong and B.P. Warkentin. Physico-chemical Analysis of High Swelling Clays. Proc., 1st Pan-American Conference on Soil Mechanics and Foundation Engineering, Mexico, Vol. 2, 1960, pp. 865-888.
4. Y. Nishida. A Brief Note on Compression Index of Soil. Journal of Soil Mechanics and Foundations Division, ASCE, Vol. 82, No. SM3, 1956, pp. 1027-1 to 14.
5. J.H. Gibbs and W.G. Holtz. Engineering Properties of Expansive Clays. Transactions, ASCE, Vol. 121, Paper 2814, 1956, pp. 641-663.
6. H.B. Seed, R.J. Woodward, and R. Lundgren. Prediction of Swelling Potential for Compacted Clays. Soil Mechanics and Foundation Engineering, ASCE, Vol. 88, No. SM3, New York, 1962, pp. 53-87.
7. A. Komornik and D. David. Prediction of Swelling Pressure of Clays. Soil Mechanics and Foundation Engineering, ASCE, Vol. 95, No. SM1, 1969, pp. 209-225.
8. T. Saito and K. Yanai. Some Swelling Characteristics of Compacted Soils. Transactions, Japan Society of Civil Engineering, No. 172, 1969, pp. 11-18.
9. V.N. Vijayavergiya and O.I. Ghazzaly. Prediction of Swelling Potential for Natural Clays. Proc., 3rd International Conference on Expansive Soils, Haifa, Israel, Vol. 1, 1973, pp. 227-236.
10. D.R. Snethan, L.D. Johnson, and D.M. Patrick. An Investigation of the Natural Microscale Mechanisms that Cause Volume Change in Expansive Clays. Interim Report FHWA-RD-77-75, FHWA, U.S. Department of Transportation, 1977.
11. F.H. Chen. Foundations on Expansive Soils. Elsevier Scientific Publishing Co., Amsterdam, The Netherlands, 1975.
12. T.W. Lambe. The Structure of Compacted Clay and The Engineering Behavior of Compacted Clay. Soil Mechanics and Foundation Engineering, ASCE, Vol. 84, No. 5, New York, 1958.
13. N.A. Tsytovich, N.K. Zaretsky, and Z.G. Ter-Martirosyan. Problems of Soil Swelling on Wetting. Proc., 3rd Asian Regional Conference on Soil Mechanics and Foundation Engineering, Haifa, Israel, Vol. 1, 1967, pp. 120-123.
14. Y. Nishida, S. Nakagawa, and H. Koike. On Swelling Pressure of Clays. Proc., 4th Asian Regional Conference on Soil Mechanics and Foundation Engineering, Bangkok, Thailand, 1971, pp. 391-395.
15. N.Ja. Dennisov. Discussion on the Heaving of Buildings and the Associated Economic Consequences with Particular Reference to the Orange Free State Goldfields by J.E. Jennings and J.W. Kerrich. The Civil Engineer in South Africa, 1963.
16. T.S. Nagaraj and B.R. Srinivasa Murthy. Rationalisation of Skemptons Compressibility Equation. Geotechnique. Vol. 33, No. 4, 1983, pp. 433-443.
17. G.H. Bolt and M.G.M. Bruggenwert. Soil Chemistry--A, Basic Elements. Elsevier Science Publishing Co., New York, 1976.
18. T.S. Nagaraj and B.R. Srinivasa Murthy. Prediction of Compressibility of Partly Saturated Soil. Indian Geotechnical Conference, Madras, India, Vol. 1, Dec. 1983, pp. I-27-32.
19. R.H.G. Parry. Field and Laboratory Behavior of Lightly Overconsolidated Clay. Geotechnique, Vol. 18, No. 2, 1968, pp. 151-171.
20. R.H.G. Parry. Some Properties of Heavily Overconsolidated Oxford Clay at a Site near Bedford. Geotechnique, Vol. 22, No. 3, 1972, pp. 485-507.
21. L. Bjerrum. Problem of Soil Mechanics and Construction of Soft Clay and Structurally Unstable Soils. General Report, Session 4, Proc., 8th International Conference Soil Mechanics and Foundation Engineering, Moscow, USSR, Vol. 3, 1973, pp. 111-159.

Publication of this paper sponsored by Committee on Engineering Geology.