Effects of CNS layer on the behaviour of underlying expansive soil media-An experimental study

by

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Introduction

Engineering behaviour of expansive soils is altogether different than conventional soils. Civil Engineering structures founded on these soils experience damages due to alternate swelling and shrinkage caused as a result of seasonal moisture changes in these soils. During last two to three decades continuous efforts are being made by research workers and field engineers, all over the world, to understand the behaviour of these soils with a view to suggest solutions for the problems arising from these deposits.

Studies by Katti et al (1969, 1973) have shown that cohesive forces of significant magnitude are developed with depth in an expansive soil system during saturation and are responsible for reducing swelling and counter-acting swelling pressures at depths. Katti et al (1967, 1969 a) indicated that cohesive nonswelling soil (*CNS*) layer may prove to be effective in resisting the swelling and swelling pressure of the underlying expansive soil.

With this in a view, an attempt is made to evaluate the effect of various thicknesses of *CNS* layer on the behaviour of underlying expansive soil media, in the laboratory by using large scale set-up. The analysis of the results is carried out in the light of micro-particle and microanchor approach suggested by Kulkarni and Katti (1973).

Experimental Programme

The soils studied, experimental set-up used and test procedure adopted are as follow.

Soils

The expansive soil used is a 'black cotton soil' from Poona and cohesive nonswelling soil is a locally available soil known as 'Powai murum'. The grain size distribution curves of these soils are shown in Figure 1 and their engineering properties are given in Table 1.

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This paper was received in December, 1979 and is open for discussion till the end of January, 1981.

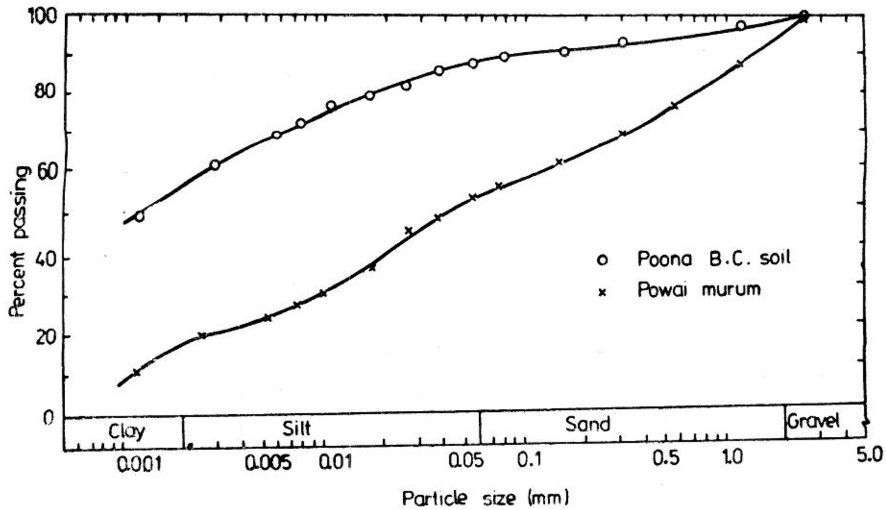

FIGURE 1 Grain size distribution curves of the soils

TABLE 1

Properties of Soils

Properties	Poona black cotton soil	Powai murum
1. Physical properties		
Liquid limit (%)	81.5	48.2
Plastic limit (%)	43.2	27.3
Plasticity index (%)	38.3	20.9
Shrinkage limit (%)	9.1	18.3
Specific gravity (at 25°c)	2.78	2.71
Standard Proctor density (g/cc)	1.35	1.47
Optimum moisture content (%)	28.9	26.1
2. Textural composition		
Gravel (>2. 00mm)	nil	4.0
Sand (2.00 to 0.06 mm)	12.0	43.0
Silt (0.06 to 0.002 mm)	32.0	35.0
2 micron clay	56.0	18.0
Classification (*USBPR*)	Clay	Clay loam
Base exchange capacity for 2 micron fraction (meq/100 gm)	123	49

Experimental set-up

A large stiffened steel tank of size 135 cm × 90 cm × 300 cm with provision to measure lateral pressures with depth was used. A sectional view of the experimental set-up is shown in Figure 2.

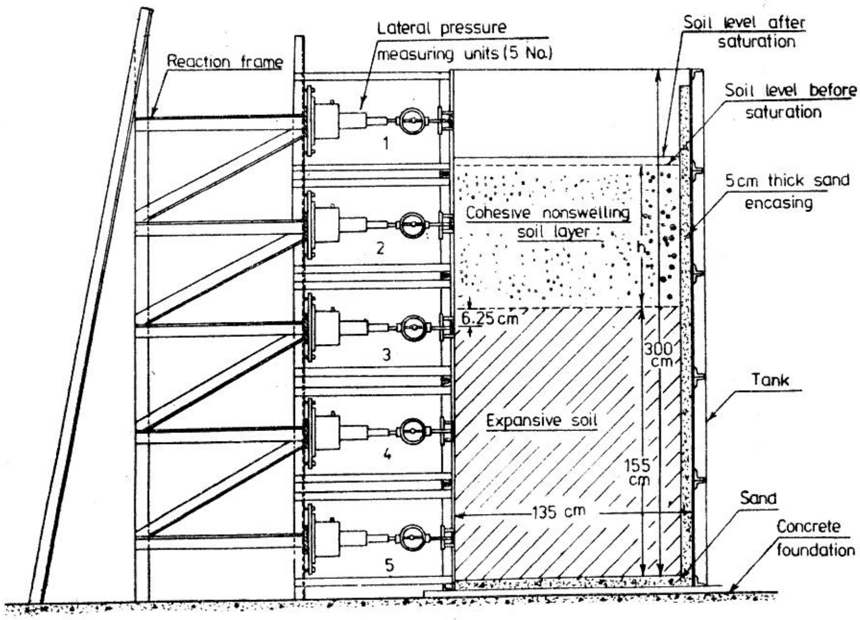

FIGURE 2 Sectional view of large scale experimental set-up

Test Procedure

Expansive soil was compacted in the steel tank at a dry density of 1.39 ± 0.01 g/cc at constant hygroscopic moisture content in layers of 2.5 cm thick upto a thickness of 155 cm. The probe plates used for recording vertical movement of soils at different depths were placed in the soil during compaction. In expansive soil, probe plates were placed at every 30 cm of compacted height and one was placed at the interface between expansive soil and *CNS* layer. Figure 3 shows the arrangement of probe plates in the experimental set-up.

Cohesive nonswelling soil layer was then compacted above the expansive soil at a dry density of 1.32 ± 0.01 g/cc and moisture content of 26 ± 1 percent. Cohesive nonswelling soil was compacted in layers of 2.5 cm up-to the selected thickness. A probe plate to record the total heave was placed on the surface of *CNS* layer. During compaction of both these soils utmost precaution was taken to control the density and moisture contents. Care was also taken to reduce the side wall friction. To expedite the saturation process, sand encasing all around the soils as shown in Figure 3 was provided. Soils were allowed to saturate. At the end of full saturation observations for vertical movements, density, moisture contents, lateral pressures, vane shear strengths at selected depths were taken. Lateral pressures were measured at depth intervals of

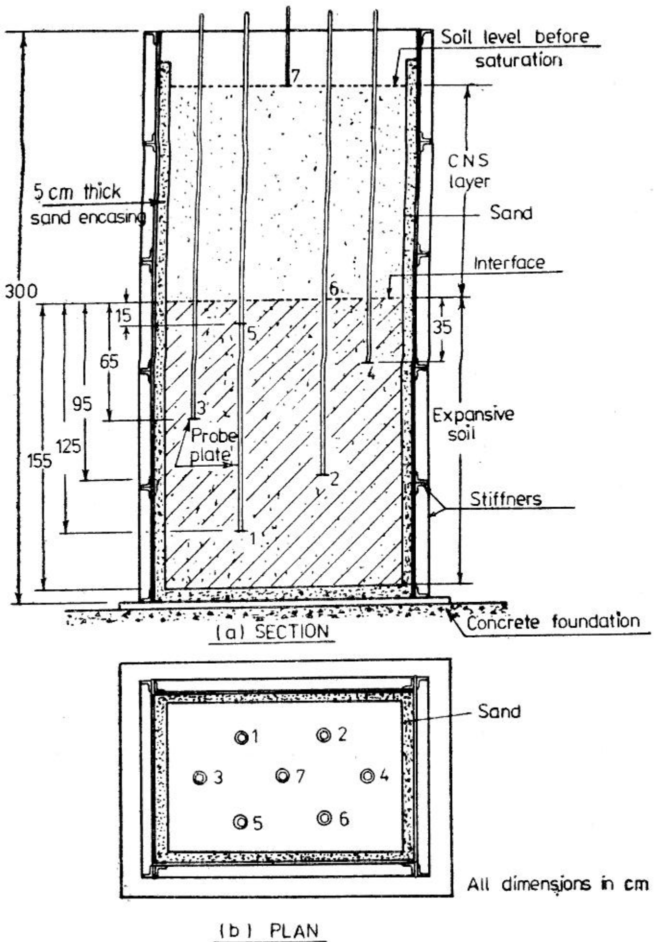

FIGURE 3 Arrangement of probe plates in large scale tank

60 cm. The thicknesses of cohesive nonswelling soil layers studied are 20, 40, 60, 80, 100 and 120 cm. For each thickness of *CNS* layer studied a separate experiment was conducted.

Results and Discussions

The results of these studies are given below.

Vertical movement at different depths

Without any *CNS* layer the vertical movement (heave) at the surface is 12.81 cm; with increase in thickness of *CNS* layer the vertical movement goes on decreasing rapidly as shown in Figure 4. Thus, by the time the thickness of *CNS* layer reaches to around 125 cm, the swelling action of underlying expansive soil media seems to be completely counteracted.

The vertical movements at various depths in expansive soil as recorded with the help of probe plates are given in Figure 5. It may be seen that

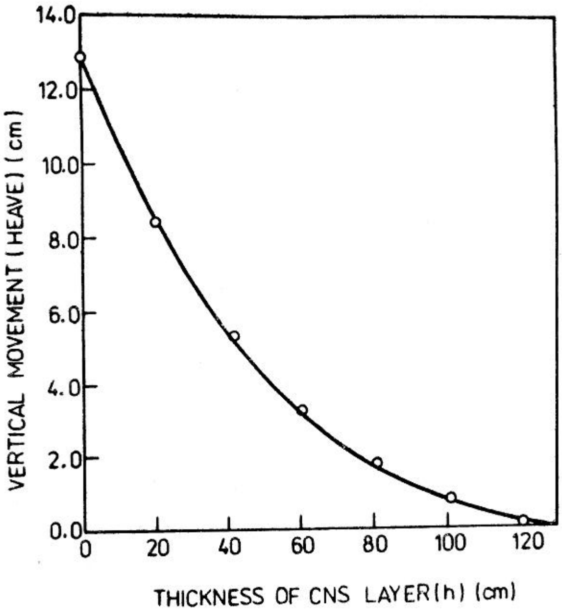

FIGURE 4 Vertical movement at the surface of soil system

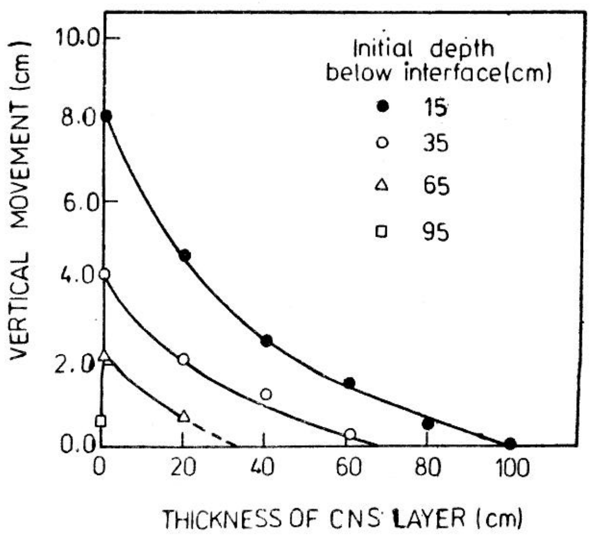

FIGURE 5 Vertical movement at different depths in expansive soil

the vertical movement decreases as the depth increases. Further, the vertical movement at certain specified depth below interface, are observed to decrease with increase in thickness of *CNS* layer. For example, at an initial depth of 35 cm, the upward movements observed are 4.25, 2.15, 1.29 and 0.36 cm, under the thickness of 20, 40 and 60 cm of *CNS* layer respectively. This clearly shows that cohesive nonswelling soil layer is effective in counteracting the swelling action of expansive soil not only at the surface but also within the soil media.

Density variation with depth

The variation of density with depth in expansive soil media, under various thicknesses of *CNS* layer is shown in Figure 6 (*a* to *g*), subsequently the values of density of expansive soil at the interface as evaluated from Figure 6 (*a* to *g*) are plotted against the corresponding thickness of *CNS* layer as shown in Figure 7. It is interesting to note from Figure 7 that the increase in thickness of *CNS* layer has a tendency to decrease the change in density of expansive soil at the interface. For example, without *CNS* layer, the density of expansive soil at the interface is of the order of 0.93 g/cc for 20, 40, 60, 80, 100 and 120 cm thicknesses the corresponding densities of expansive soil at the interface are 1.17, 1.255, 1.30, 1.335, 1.365 and 1.38 g/cc respectively. Thus, the changes in density are 0.24, 0.085, 0.045, 0.035, 0.03 and 0.015 g/cc. for every 20 cm increment in thickness of *CNS* layer.

Even for the case wherein no *CNS* layer is placed, the density increases from surface downwards and attains nearly a constant value at a depth of about 115 cm. This is in agreement with the earlier work by Katti et al (1969, c) and Kulkarni and Katti (1973).

The effects of varying thicknesses of *CNS* layer on the depth beyond which density remains almost constant (referred here as 'no volume change' depth) are shown in Figure 7. The 'no volume change' depths observed are 115, 80, 60, 45, 30, 15 and 5 cm under the *CNS* layer of thicknesses 0, 20, 40, 60, 80, 100 and 120 cm respectively. These observations indicate that the cohesive nonswelling soil layer has considerable effect on reducing the volume changes within the swelling soil media with depth. The density variation in *CNS* layer is insignificant.

Figure 8 (*a to g*) illustrates the variation of moisture content with depth in soil system. It may be noted that the degree of saturation at various depths in expansive soil, computed on the basis of density and moisture content existing at that depth level, show more or less 100 percent saturation. The moisture contents at different depths in *CNS* layer also corresponds to full saturation moisture.

Variation of lateral pressure with depth

The variation between lateral pressure with depth in the soil system is shown in Figure 9 (*a* to *g*). The lateral pressures recorded in the *CNS* layer are of very small order. The magnitudes of lateral swelling pressures generated in expansive soil at the interface, goes on increasing with increase in thickness of *CNS* layer as shown in Figure 7. The lateral swelling pressures observed at the interface are 0, 0.5, 1.25, 1.80, 2.40, 2.85 and 3.15 kg/cm^2 under *CNS* layer of thicknesses 0, 20, 40, 60, 80, 100 and 120 cm respectively.

The studies by Kulkarni (1972) showed that, the vertical and lateral swelling pressures at any depth in expansive soil media are nearly same in magnitude. This is in confirmity with micro-particle theory. Thus at the interface, the vertical swelling pressures expected to develope are of the order of 0, 0.50, 1.25, 1.80, 2.40, 2.85 and 3.15 kg/cm^2 under the thicknesses of *CNS* layer of 20, 40, 60, 80, 100 and 120 cm. respectively. The downward stresses, acting due to self weight of *CNS* layer, corresponding to these thicknesses are 0, 0.03, 0.06, 0.09, 0.12, 0.15 and 0.18 kg/cm^2

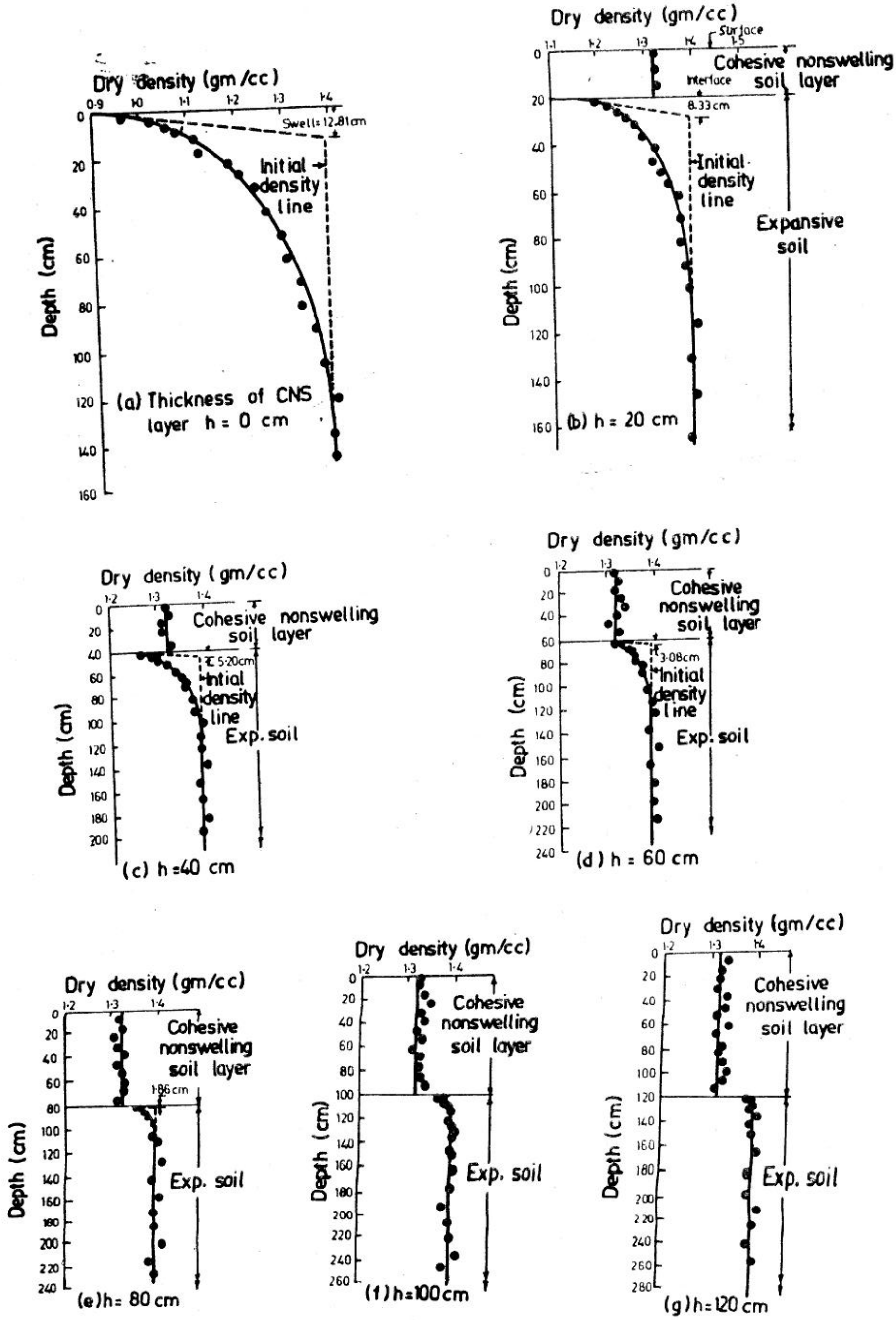

FIGURE 6 **Dry density variation with depth in soil system for different thicknesses of *CNS* layer.**

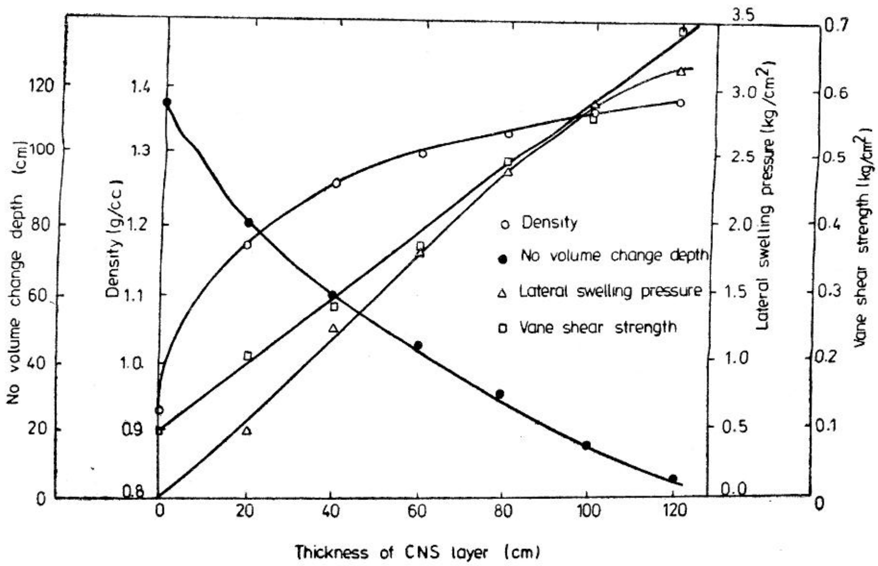

FIGURE 7 **Variation of dry density, lateral swelling pressure and vane shear strength at the interface and 'no volume change' depth with thickness of *CNS* layer.**

respectively. Which means, the amount of upward swelling pressure balanced by *CNS* layer is of much higher magnitude than its weight. This shows that the internal characteristics of *CNS* layer and the interaction of various swelling clay particles with *CNS* layer and amongst themselves at the interface, seems to be responsible for counteracting the swelling pressure over and above the self weight of *CNS* layer.

It is seen that the *CNS* layer of thicknesses 20, 40, 80, 100 and 120 cm are able to resist the vertical swelling pressure more or less of the order of 0.50, 1.25, 1.80, 2.40, 2.85 and 3.15 kg/cm^2 respectively. With this consideration, the resistances offered to the vertical swelling pressures by each 20 cm increment in thickness of *CNS* layer from 20 to 40, 40 to 60, 60 to 80, 80 to 100 and 100 to 120 cm are 0.75, 0.55, 0.60, 0.45 and 0.30 kg/cm^2 respectively.

Shear strength versus depth

Figure 10 (*a* to *g*) shows the variation between vane shear strength and depth in the soil system. The vane shear strengths of *CNS* layer at different depths do not show any significant variation. It remains within the range of 0.30 ± 0.02 kg/cm^2.

The trend in variation of vane shear strength with depth in expansive soil below interface is similar to that observed in case of density versus depth. The vane shear strength of expansive soil at the interface increases with increase in thickness of *CNS* layer as illustrated in Figure 7.

Based on the values of dry density obtained at the interface under different thicknesses of *CNS* layer and corresponding lateral swelling pressure and vane shear, a relationship as shown in Figure 11 has been obtained. It can be seen from this relationship that both the lateral

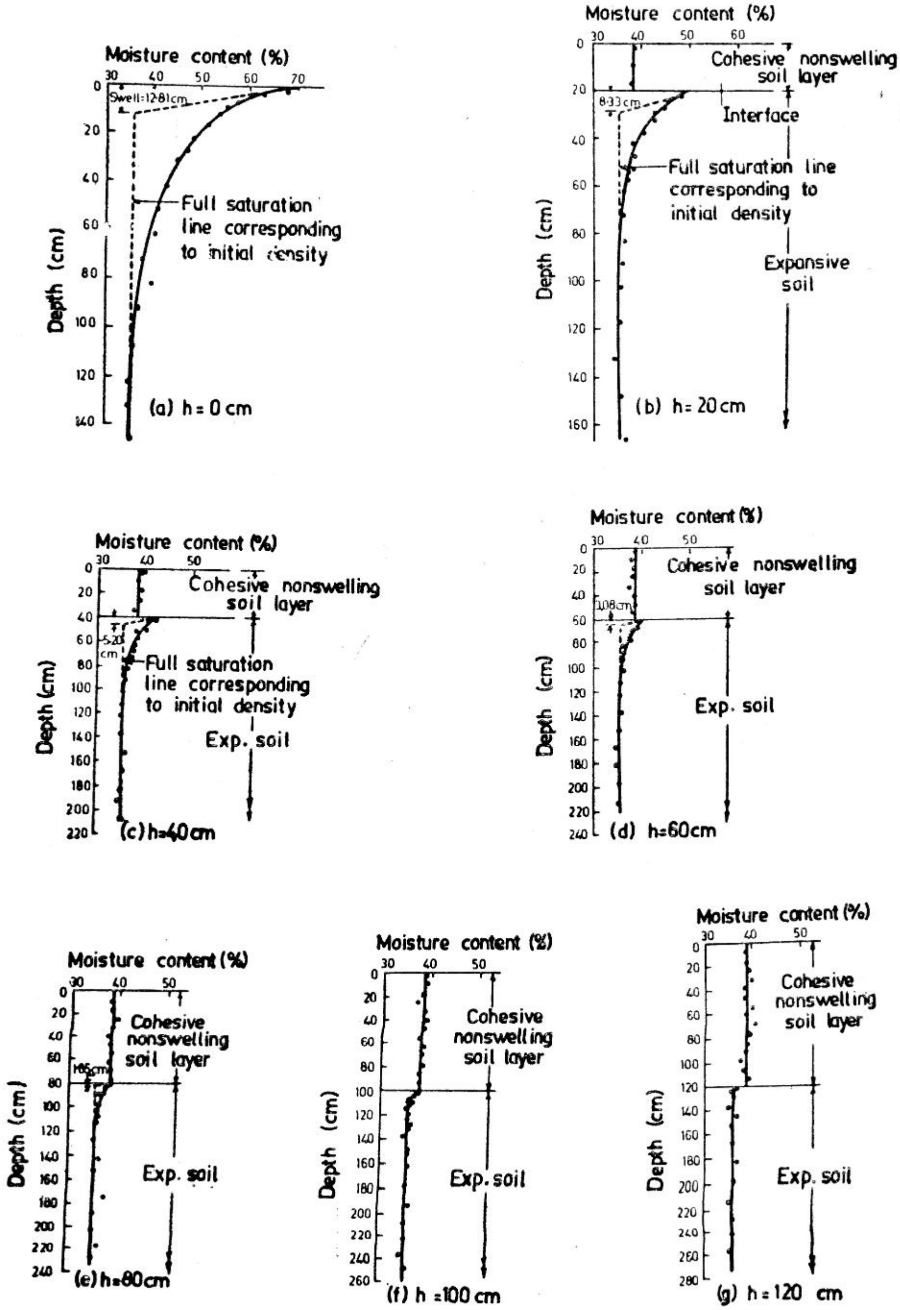

FIGURE 8 Variation of moisture contents with depth in soil system for various thicknesses of *CNS* layer.

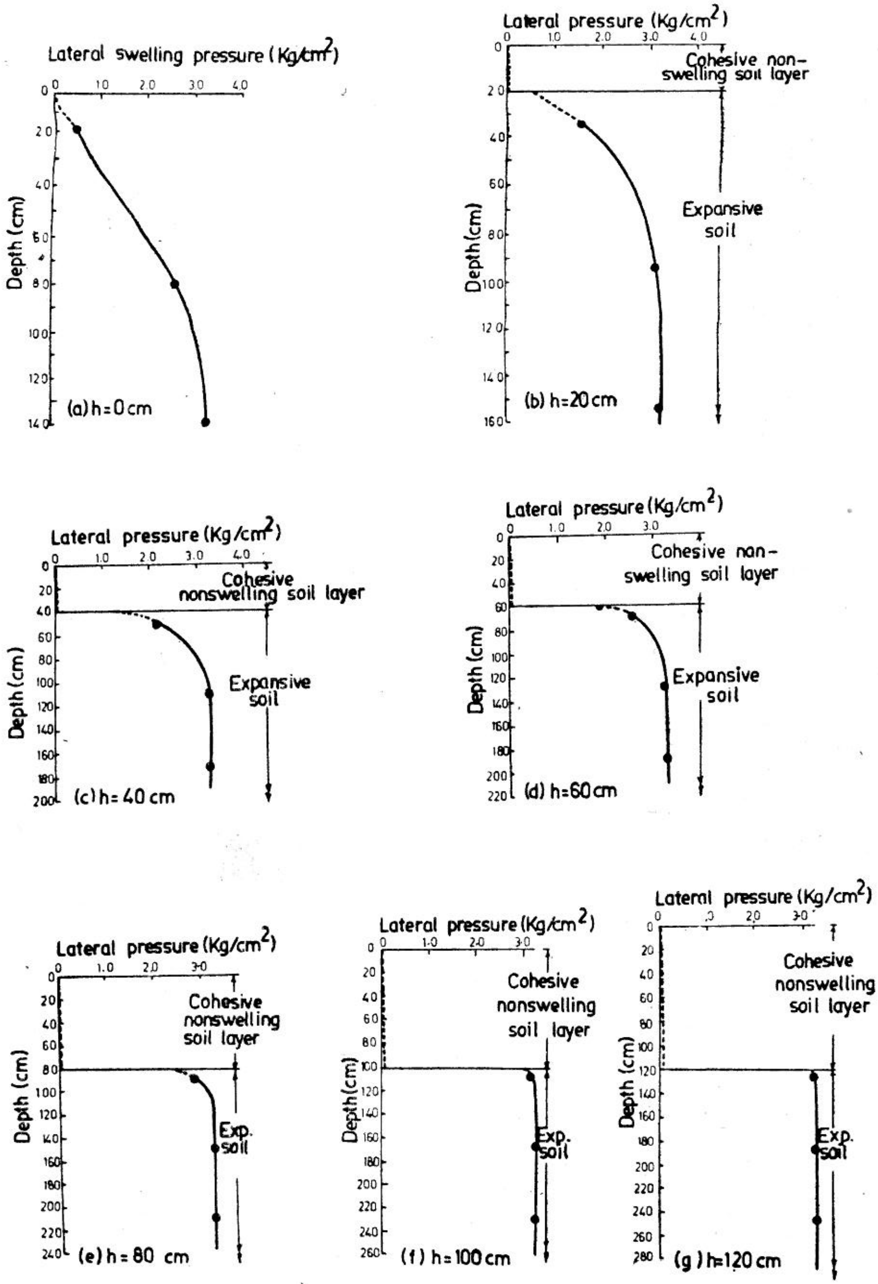

FIGURE 9 Plots of lateral pressure with depth in soil system for different thicknesess of *CNS* layer.

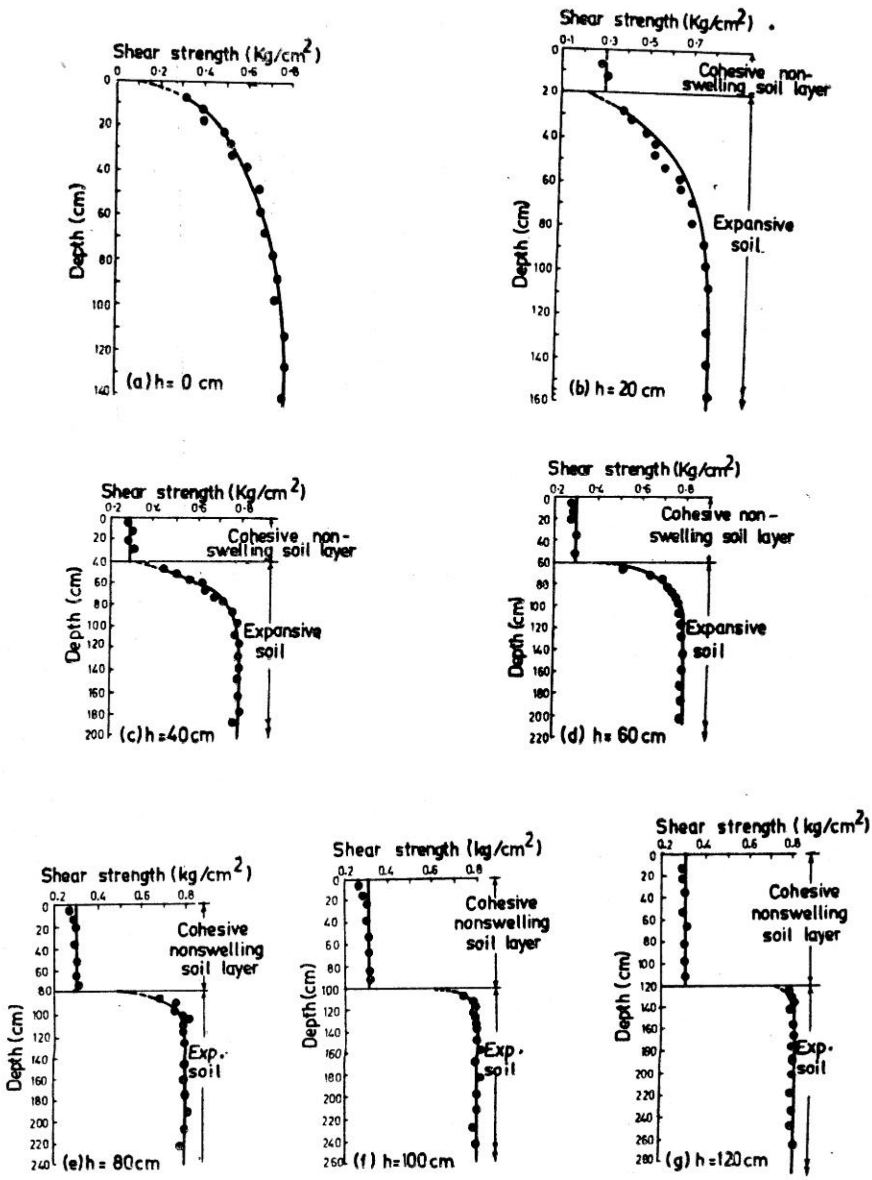

FIGURE 10 **Vane shear strength variation with depth in soil system for various thicknesses of *CNS* layer.**

swelling pressure and vane shear strength increases curvilinearly with increase in dry density.

Analysis

The experimental investigation with cohesive nonswelling soil (*CNS*) layer clearly indicated that, not only the weight of *CNS* layer, but to a large extent its internal characteristics appear to be responsible for resist-

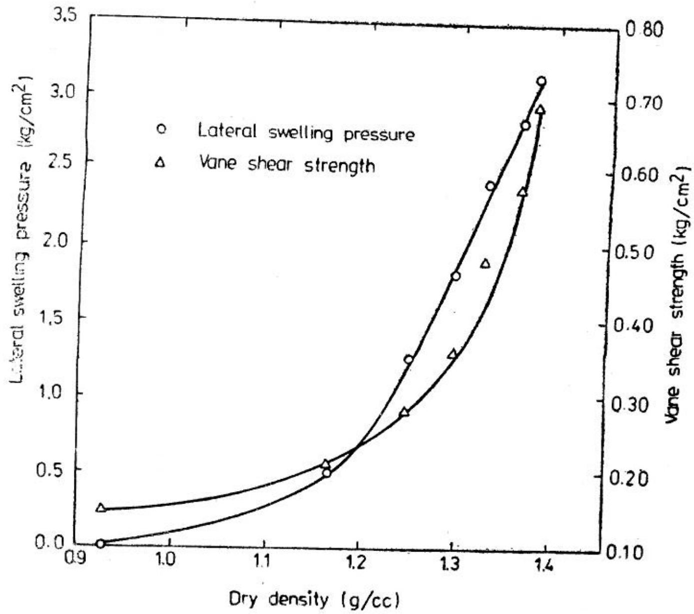

FIGURE 11 Relationship between lateral swelling pressure and vane shear strength with dry density

ing swelling and swelling pressure action of underlying expansive soil mass to varying degrees, depending upon the thickness.

To understand the probable effect of *CNS* layer on swelling and swelling pressure aspect of expansive soil media below, an attempt is herein made to visualize the possibility of expansive forces at the interface being resisted by the shear stresses mobilised in the *CNS* medium.

Configuration of particles in an idealised system

With this in a view, the configuration as adopted in micro-particle and micro-anchor approach (Kulkarni and Katti, 1973, Kate and Katti, 1975) to evaluate the number of particles and their spacings in an idealized system, is used in the present analysis. Accordingly, the following expression gives the number of clay particles in unit volume of soil mass

$$\overline{N} = \frac{p}{l^3(1+e)} \qquad \text{...(1)}$$

Wherein,

$\overline{N}$ = Number of clay particles in unit volume of soil mass in an idealized system

l = Dimension of an idealized individual clay particle considered to be cubical in shape

p = Percentage of clay fraction of size l in the soil

e = Void ratio of the soil mass

Number of idealized clay particles in unit volume of soil mass in one row or one column is given by

$$\overline{N}_r = \frac{1}{l} \times \left[\frac{p}{1+e}\right]^{1/3} \qquad \ldots(2)$$

where, $\overline{N}_r$ is the number of clay particles in one row or one column in unit volume of soil mass.

Similarly, with the help of following expression, the number of idealized clay particles with their 'c' axes oriented in vertical direction in unit area, can be evaluated.

$$\overline{N}_v = \frac{1}{3}\left[\overline{N}_r\right]^2 = \frac{1}{3l^2}\left[\frac{p}{1+e}\right]^{2/3} \qquad \ldots(3a)$$

in which, $\overline{N_V}$ is the number of vertically oriented clay particles in unit area. Similarly if $\overline{N_L}$ denotes the number of idealized clay particles in unit area with their 'c' axes oriented in any of the lateral direction then, by symmetry :

$$\overline{N}_L = \frac{1}{3\,l^2}\left[\frac{p}{1+e}\right]^{2/3} \qquad \ldots(3b)$$

The distance between two adjacent particles is given by

$$d = \frac{1}{\overline{N}_r} = l\left[\frac{1+e}{p}\right]^{1/3} \qquad \ldots(4)$$

where, d is the centre to centre spacing between adjacent particles.

The spacing between two vertically oriented (or like oriented) particles in a row or column is

$$D = 3d = 3\,l\left[\frac{1+e}{p}\right]^{1/3} \qquad \ldots(5)$$

wherein, D is the centre to centre distance between two vertically oriented (or like oriented) particles in a row or column.

The Table 2 shows the values of spacings between two vertically oriented particles (D), and the ratio of these spacings with the size of clay particles (D/l), as computed for various void ratios (e) of the soil mass by considering $l = 2\ \mu$ (micron) and $p = 56.0$ percent of $2\ \mu$ clay.

TABLE 2

***D/l* ratios for the particles in an idealized system**

e	0 8	0.9	1.0	1.1	1.2	1.3
D (micron)	8.82	8.99	9.14	9.30	9.42	9.57
D/l	4.41	4.495	4.57	4.65	4.71	4.785

From these values, it can be seen that, these particles are spaced more than 4 times their width.

Conceptual footings and shear planes

At the interface between *CNS* layer and expansive soil, vertically oriented swelling soil particles of 2 μ size, may be visualized as though footings of 2 μ x 2 μ, resting on *CNS* layer in an inverted fashion; as indicated in Figure 12 (*a*). These conceptual footings are acted upon by the forces, developed due to the swelling pressure of an idealized individual particle (q_{wsi}). Figure 12 (*b*) shows the conceptual shear planes in a soil mass, developed due to the footings. The spacings and configuration of these particles are such that, during development of their swelling and swelling pressure, they can produce shear surfaces in the cohesive nonswelling soil layer. When the expansive soil having semi-infinite extent is covered with *CNS* layer also of semi-infinite extent; these particles at the interface would produce shear lines as conceived in Figure 13. The shear lines generated due to individual particle would be interfered mutually, as several particles are involved in the phenomenon. The angle of these shear lines with the horizontal may vary, depending upon the frictional (ϕ) conditions of *CNS* layer.

The probable area of shear surfaces mobilised in *CNS* layer of particular developed cohesion (c_u) value, would depend upon the swelling pressure characteristics of individual particles of underlying expansive soil; and also on the configuration of other adjoining swelling particles. The swelling pressure of an idealized individual clay particles, can be deter-

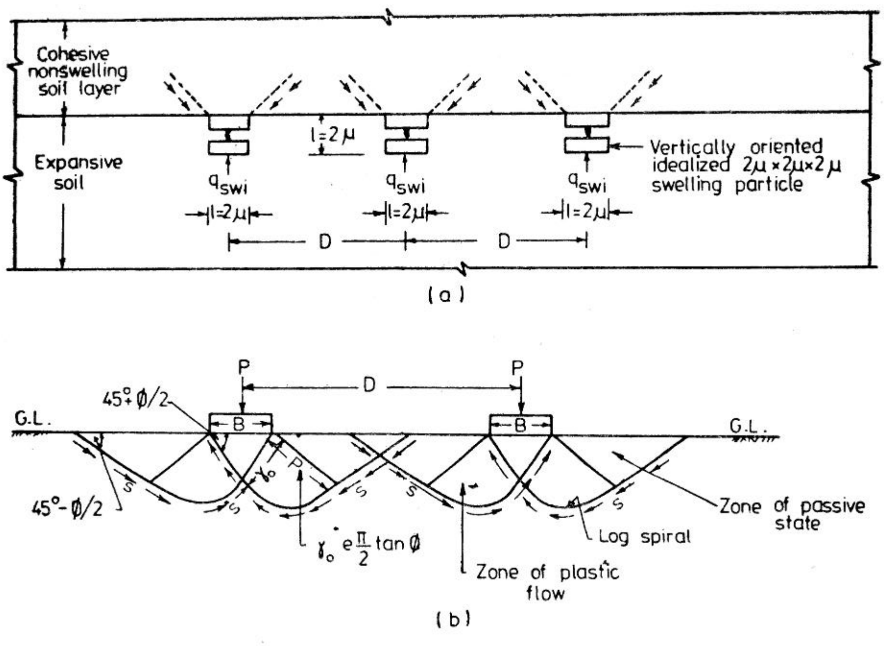

FIGURE 12 (a) Conceptual idealized footings of 2μ x 2μ resting on *CNS* layer in an inverted fashion

(b) Conceptual shear planes developed in a soil mass due to footings

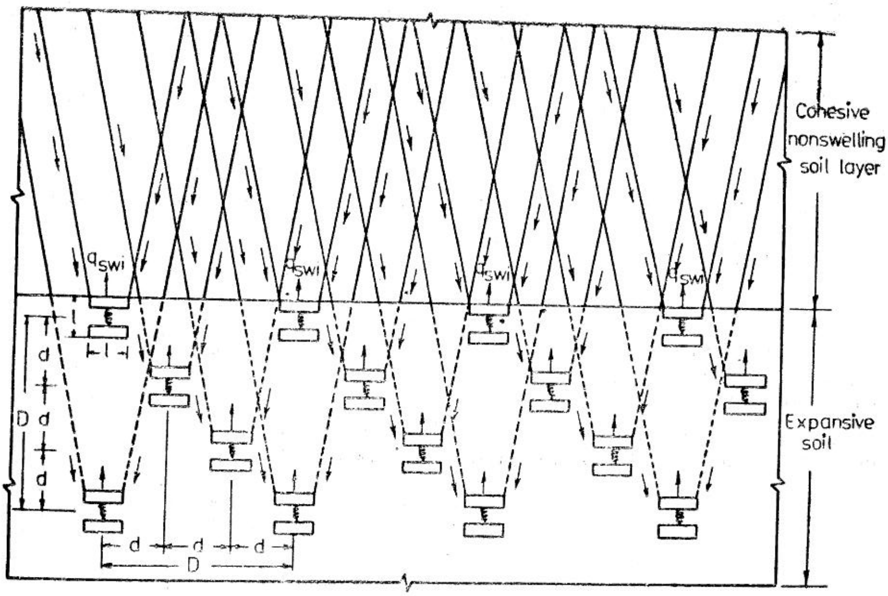

FIGURE 13 Conceptual shear lines generated in semi-infinite soil mass by idealized footings

mined from the known value of swelling pressure measured at the surface of soil mass, by the following equation (Kulkarni and Katti, 1973).

$$q_{swi} = 3\ q_{sw} \left[\frac{1+e}{p}\right]^{2/3} \qquad \ldots(6)$$

where,

q_{swi} = Swelling pressure of an idealized individual clay particle, and

q_{sw} = Swelling pressure of the soil mass measured at its surface.

Relation between swelling and swelling pressure of an idealized individual particle

The swelling pressure of an idealized individual particle considered in the '*c*' direction alone, will have a maximum value when its swelling is prevented totally along '*c*' direction. The relationship between swelling pressure of an idealized individual particle and its expansion characteristics, may be expressed approximately with the following generalised expression.

$$\left[\frac{q_{swi}}{q_{swi_{max}}}\right]^{n_1} + \left[\frac{\epsilon - l}{\epsilon_{max} - l}\right]^{n_2} = 1 \qquad \ldots(7)$$

in which,

$q_{swi_{max}}$ = The maximum possible value of swelling pressure of an idealized individual particle *i.e.* when its swelling is prevented totally

ϵ = The swelling of an idealized individual clay particle

ϵ_{max} = The maximum possible value of swelling of an idealized individual particle i.e. when its swelling pressure is zero

n_1, n_2 = The exponential coefficients. The magnitudes of n_1 and n_2 depend on the electrical imbalance of the particle and the soil characteristics.

In the soil system having an idealized particle orientation, the swelling and swelling pressure would be same in vertical and horizontal directions.

Equilibrium of Vertically Oriented Particle

The expansive soil compacted at initial void ratio of e_1 is covered with *CNS* layer of thickness *h*. After saturation, the expansive soil attains a void ratio e_2 at the interface. Considering the equilibrium of an idealized individual vertically oriented swelling particle at the interface, the driving force would be its swelling force acting vertically upward, and the resisting force would be shearing force mobilised on certain area in *CNS* layer acting downward as indicated in Figure 14 (a).

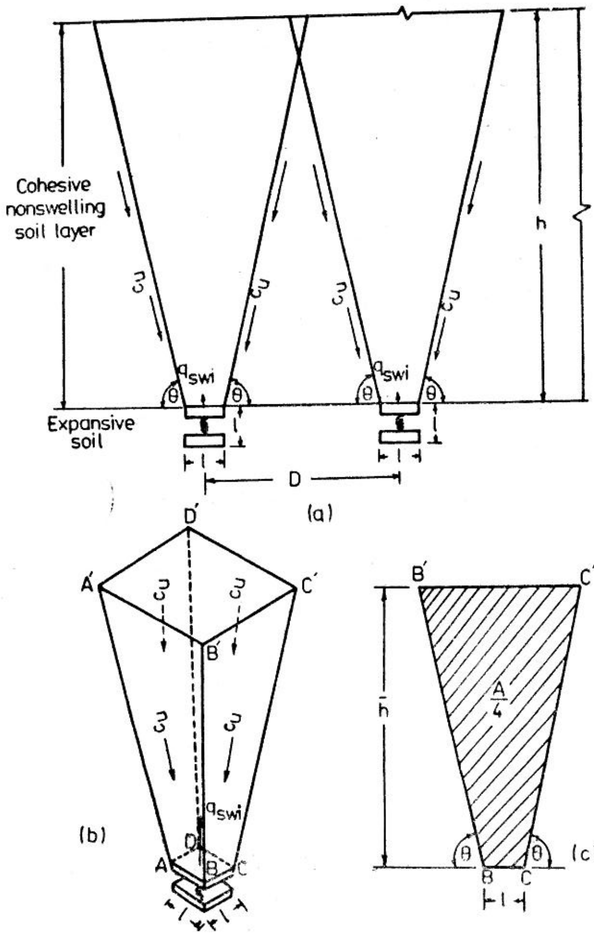

FIGURE 14 Conceptual pattern of shear surface mobilized in *CNS* layer due to vertically oriented swelling particles

It is assumed that, the shear surfaces generated in *CNS* layer forms a frustum of an inverted regular pyramid as illustrated in Figure 14 (b). Thus, the area A of shear surfaces on which developed cohesion c_u of *CNS* layer is mobilised, would consists of the area of four identical trapezoids forming the frustum. If $\bar{h}$ denotes the height upto which shearing planes are generated in *CNS* layer and θ is the angle of these planes with the horizontal as shown in Figure 14 (c) then, the magnitude of A is obtained by,

$A = 4 \times$ Area of each trapezoid

$$= 4\left[\frac{\bar{h}}{\sin\theta}\,(l+\bar{h}\cot\theta)\right] \qquad \text{... (8)}$$

Neglecting the self weight of *CNS* layer, the following expression can be formulated by equating driving forces with resisting forces.

[Swelling force on individual particle in vertical direction] = [Component of shearing force mobilised in *CNS* layer in vertical direction]

which gives,

$$q_{swi}\,.\,l^2 = A.c_u.\sin\theta \qquad \text{... (9a)}$$

Substituting the value of A as evaluated in Equation (8), the Equation (9*a*) takes the form as given below.

$$q_{swi}\,.\,l^2 = 4\left[\frac{\bar{h}}{\sin\theta}\,(l+\bar{h}\cot\theta)\right] c_u.\sin\theta \qquad \text{... (9b)}$$

or,

$$\cot\theta.\,\bar{h}^2 + l\bar{h} - \frac{q_{swi}\,.\,l^2}{4\,c_u} = 0 \qquad \text{... (9c)}$$

from which $\bar{h}$ is determined by

$$\bar{h} = -\frac{l}{2\cot\theta}\left[1 \pm \sqrt{1+\cot\theta.\,\frac{q_{swi}}{c_u}}\right] \qquad \text{... (10a)}$$

Introducing the value of q_{swi} as given in Equation (6) the magnitude of $\bar{h}$ is obtained in the final form by the following expression

$$\bar{h} = -\frac{l}{2\cot\theta}\left[1 \pm \sqrt{1+3\cot\theta.\,\frac{q_{sw}}{c_u}\left(\frac{1+e}{p}\right)^{2/3}}\right] \qquad \text{... (10b)}$$

It may be mentioned here that, the Equation (10*b*) is not applicable for $\theta = 90°$. Hence, for the case when $\theta = 90°$, $\bar{h}$ can be derived from Equation (9*c*), which gives

$$\bar{h} = \frac{q_{swi}\,.\,l}{4\,c_u} = \frac{3}{4}\,.\,l.\,\frac{q_{sw}}{c_u}\left(\frac{1+e}{p}\right)^{2/3} \qquad \text{... (10c)}$$

For example, the magnitude of q_{sw} for 'no volume change' condition for an initial void ratio of 1.0 and $p = 56.0$ per cent of 2 μ clay, is observed to be 3.27 kg/cm². Considering c_u of the order of 0.2 kg/cm², the $\bar{h}$ needed to balance the swelling pressure of an idealized individual 2 μ

particle for θ equal to say 45° is

$$\bar{h} = -\frac{l}{2 \cot \theta}\left[1 \pm \sqrt{1+3 \cot \theta . \frac{q_{sw}}{c_u}\left(\frac{1+e}{p}\right)^{2/3}}\right]$$

$$= -\frac{2\times10^{-4}}{2\times1}\left[1 \pm \sqrt{1+3\times1\times \frac{3.27}{0.20}\left(\frac{1+1.0}{0.56}\right)^{2/3}}\right]$$

$= +9.80$ micron and -11.80 micron. Considering positive value only, $\bar{h}=$ 9.80 micron. The magnitudes of $\bar{h}$, thus computed for various values of θ are as given below.

TABLE 3

Values of $\bar{h}$ for different θ

θ (Degree)	35	40	45	50	55	50	65	70
$\bar{h}$ (micron)	8.27	9.08	9.80	10.65	11.52	12.60	13.51	15.32

The computations show that, the magnitude of $\bar{h}$ increases with increase in θ. For the extreme case when $\theta = 90°$, $\bar{h}$ is maximum and the value of which in the above example would be 57.6 micron. Thus, $\bar{h}$ required to balance the swelling force of an idealized discrete particle would be of very small order. However, the observations in the laboratory indicates that, the actual thickness of *CNS* layer needed to balance the swelling pressure could be of much higher magnitude. The higher amount of actual thickness required in the process, may be attributed to the interference effect of other swelling particles dispered in the swelling soil media.

The interference action may be a function of swelling pressure of an individual particle and configuration of other swelling particles. Thus, indirectly, the probable number of particles with their shear planes interfering with each other may be determined approximately, by the expression

$$N_I = \frac{h_1}{\bar{h}} \qquad \dots (11)$$

wherein,

N_I is 'Interference number' or the number of particles with their shear planes interfering mutually and

h_1 is the actual thickness of *CNS* layer encountered in the process of balancing the swelling pressure of soils mass.

In the preceding example, the values of $\bar{h}$ are ranging from 8.27 to 57.6 micron, depending upon θ. Whereas, for the same void ratio condition of expansive soil, the actual value of thickness of *CNS* layer required to bring 'no volume change' condition at the interface, is about 125 cm as observed in large scale tests. On this basis, taking $h_1 = 125$ cm, the Interference number (N_I) as calculated for different magnitudes of θ are shown in Table 4.

TABLE 4

N_I values for various θ

θ (Degree)	35	40	45	50	55	60	65	70	90
N_I 10^4	15.1	13.78	12.76	11.74	10.76	9.92	9.25	8.15	2.17

These values clearly indicate that, when shearing planes are at smaller angles the magnitude of $\bar{h}$ would reduce; which in turn may increase the number of particles interfering with each other. Or, the Interference number assuming constant for a particular type of soil, the area of shear surface available to resist the swelling force would depend on the thickness of *CNS* layer. Higher the thickness of *CNS* layer more will be the possibility of generating higher area of shear surfaces and vice versa.

In large scale experiments, it is observed that the swelling pressure of the order of 3.15 kg/cm² has been balanced by *CNS* layer of thickness 120 cm; whereas, the downward stress due to the self weight of 120 cm thick *CNS* layer is only 0.18 kg/cm². In the light of analysis, the remaining swelling pressure over and above the self weight seems to have been balanced by shearing surfaces developed within the *CNS* layer.

When the thickness of *CNS* layer is less than 125 cm, the soil-system shows heaving; thus indicating indirectly, that the number of particles shearing the same shear surfaces remains approximately same. Assuming for the simplicity of computations the magnitude of $\theta = 90°$; the values of $\bar{h}$ and N_I as computed for various thicknesses of *CNS* layer studied are reported in Table 5. The relationship between $\bar{h}$ and h, as illustrated in Figure 15 shows an increase in $\bar{h}$ with increase in h within the ranges of h studied. The magnitudes of N_I are in the range of 1.70×10^4 and 2.16×10^4.

TABLE 5

Magnitudes of q_{swi}, ϵ, $\bar{h}$ and N_I at the interface for different thicknesses of *CNS* layer

$e_1 = 1\,0$, $p = 56.0$ per cent, $c_u = 0.20$ kg/cm² and $\theta = 90°$

h (cm)	γ_d (g/cc)	e_2	q_{sw} (kg/cm²)	q_{swi} ((kg/cm²)	ϵ (micron)	$\bar{h}$ (micron)	N_I
20	1.17	1.37	0.50	3.93	5.05	9.83	2.04×10^4
40	1.255	1.215	1.25	9.43	4.00	23.6	1.70×10^4
60	1.30	1.14	1.80	13.3	3.65	33.2	1.81×10^4
80	1.335	1.08	2.40	17.35	3.45	43.4	1.85×10^4
100	1.365	1.037	2.85	20.6	3.00	51.5	1.94×10^4
120	1.38	1.01	3.15	22.2	2.90	55.5	2.16×10^4

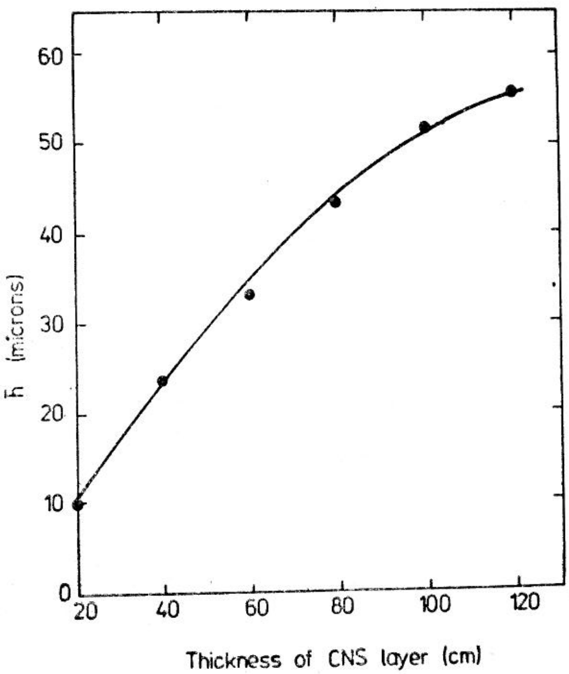

FIGURE 15 Variation of $\bar{h}$ with thickness of *CNS* layer

In case, the thickness of *CNS* layer is adequate to provide the area of shear surfaces, required to balance the entire swelling force then, the swelling will not be encountered in the soil mass, and the 'no volume change' condition will exist at the interface itself. However, when the thickness is inadequate to provide the required area of shear surfaces, then certain amount of swelling may take place until the total shear stresses mobilised in the *CNS* layer, come to an equilibrium with the swelling pressure of the particle under swollen state.

When the particles at the surface of expansive soil are prevented from swelling, automatically other particles below will be resisted, since the lower particles get the higher opportunity to adjust themselves as a result of the resistance offered by the overlying expansive soil media. Thus, the 'no volume change' depth which is also a function of swelling of individual particle, would approach near to the interface depending upon the degree of swelling of individual particle at the interface. In other words, 'no volume change' depth will go on reducing with increase in thickness of *CNS* layer.

Total swelling (heave) observed would naturally be a function of swelling of individual particles, taking place upto 'no volume change' depth. The experimental curve showing thickness of *CNS* layer versus total swelling (vertical movement at the surface) and the curve indicating relationship between thickness of *CNS* layer and ϵ as given in Table 5 are shown in Figure 16, which indicates that both the curves have similar trend.

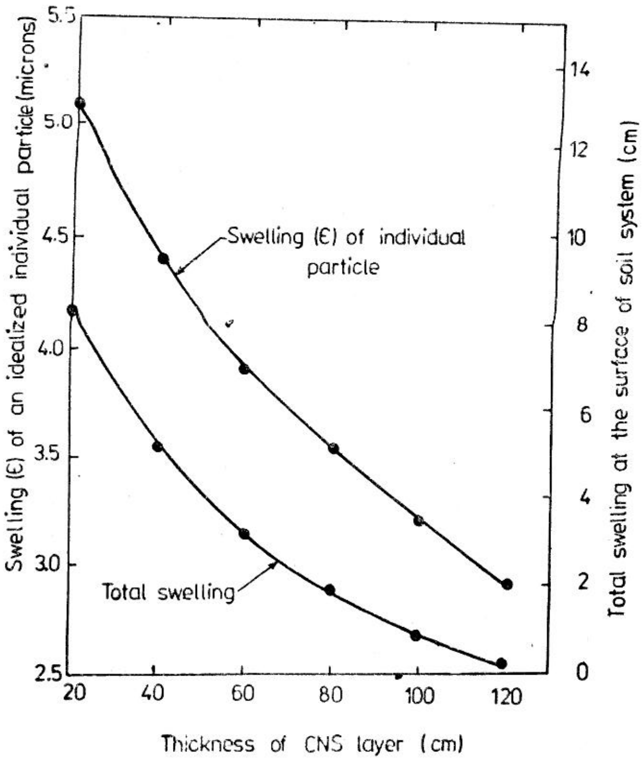

FIGURE 16 **Variation of swelling of an idealized individual particle and total swelling with thickness of *CNS* layer.**

The following expression may be formulated to relate, the total swelling with the swelling of an idealized individual particle.

$$S = \Sigma\ N_{v1}\ (\epsilon_1 - l - \epsilon_1') + N_{v2}\,(\epsilon_2 - l - \epsilon_2') \ldots N_{vn}\ (\epsilon_n - l - \epsilon_n') \quad \ldots(12)$$

in which,

S = Total swelling (heave) of soil mass at the surface,

$N_{v1}, N_{v2}, \ldots N_{vn}$ = Number of vertically oriented swelling soil particles in each unit height of column below interface upto 'no volume change' depth,

$\epsilon_1, \epsilon_2, \ldots \epsilon_n$ = The average swelling of an idealized individual clay particle in each unit height of column upto 'no volume change' depth, and

$\epsilon_1', \epsilon_2', \ldots \epsilon_n'$ = The average swelling taking place inside the voids in each unit height of column upto 'no volume change' depth below interface.

When a clay particle swells in the soil media, it may swell partly in the voids and partly over the voids. The swelling of clay particles over and above the voids may start bringing out alterations in the volume of soil mass. The pressure condition is affected both due to swelling taking place inside the voids as well as the overall swelling. It is possible to determine approximately the swelling of individual particle taking place inside the voids by knowing the alteration in the void ratio with respect to initial condition, and the relation between the swelling pressure and void ratio.

Initially, the void ratio of soil mass is e_1 which changes to e_2 due to moisture intake. Thus, the volumetric change $\triangle_v$ taking place per unit volume of soil mass would be

$$\triangle_v = \left[\frac{e_1 - e_2}{1+e_1}\right] \times 1 \quad \ldots(13)$$

If $\bar{N}$ is the number of particles in unit volume of soil mass, the volumetric change per particle $\triangle_{v_i}$ will be given by

$$\triangle_{v_i} = \left[\frac{e_1 - e_2}{1+e_1}\right] \frac{1}{\bar{N}} \quad \ldots(14)$$

The swelling of each individual particle is taking place along its 'c' axis. The swelling of each particle along 'c' axis could be obtained by dividing its volumetric change with its constant area along 'a' and 'b' axes. Denoting the swelling of an idealized individual particle over and above the voids by $\bar{\epsilon}$, then $\bar{\epsilon}$ is evaluated by

$$\bar{\epsilon} = \frac{\triangle_{v_i}}{l \times l} = \left[\frac{e_2 - e_1}{1+e_1}\right] \cdot \frac{1}{\bar{N}} \cdot \frac{1}{l^2} \quad \ldots(15)$$

Considering ϵ as the swelling of an idealized individual particle of dimension l, its swelling in the voids ϵ' would be

$$\bar{\epsilon}' = (\epsilon - l) - \bar{\epsilon} = \epsilon - l - \left[\frac{e_2 - e_1}{1+e_1}\right] \frac{1}{\bar{N}} \cdot \frac{1}{l^2} \quad \ldots(16)$$

Although it is realised that, swelling and swelling pressure relationship is responsible for the heave, but it may not be possible to evaluate the heave directly, from ϵ versus q_{swi} relationship, owing to the various degrees of swellings taking place in the voids which itself is a function of configuration of particles in relation to each other. Thus, if the internal amount of swelling is known it is possible to know the heave.

Similarly, the equilibrium of laterally oriented clay particle can be explained by micro-anchor approach suggested by Kulkarni and Katti (1973).

Conclusions

In the light of the studies conducted, the following conclusions have been arrived at

1. The experiments conducted in large scale set-up indicate that the shear strength of cohesive nonswelling soil layer is highly effective in counteracting the swelling and swelling pressure of the underlying expansive soil media. The weight of the cohesive nonswelling soil layer is far less than the swelling force generated due to expansive soil.

2. Cohesive nonswelling soil layer has considerable effect on density, lateral swelling pressure and shear strength of underlying expansive soil with depth, and also on the 'no volume change' depth.

   Within the ranges of thicknesses of cohesive nonswelling soil layer studied here for a saturated condition, increase in thickness of cohesive nonswelling soil layer increases the magnitudes of density, lateral swelling pressure and shear strength at the interface and decreases 'no volume change' depth in the expansive soil media.

3. The analysis, based on micro-particle and micro-anchor approach seems to explain the alterations taking place in the expansive soil surface in contact with cohesive nonswelling soil layer.

The effects observed may have wide applications in relation to construction of civil engineering structures in expansive soil deposits.

Acknowledgements

Authors are thankful to the laboratory staff of Soil Engineering and Highway Engineering Sections of Civil Engineering Department of I.I.T. Bombay for their considerable help during various stages of experimentation.

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Notations

A	Area of shear surface
CNS	Cohesive nonswelling soil
c_u	Developed cohesion
D	Spacing between two like oriented particles in a row or column
d	Centre to centre distance between adjacent particles
e	Void ratio
e_1	Initial compaction void ratio
e_2	Final equilibrium void ratio attained on saturation
h	Thickness of cohesive nonswelling soil layer
h_1, h_2	Actual thickness of cohesive nonswelling soil layer encountered in the process of balancing swelling and swelling pressure of underlying expansive soil mass
$\bar{h}$	Vertical distance in *CNS* layer upto which shear planes are mobilised due to the effect of swelling force on an idealised individual particle
l	Dimension of an idealized individual clay particle, considered to be cubical in shape, (=2 micron)
$\bar{N}$	Number of idealized clay particles in unit volume of soil mass
N_I	Interference number
$\bar{N}_L$	Number of idealized clay particles oriented in lateral direction, in unit area
$\bar{N}_r$	Number of idealized clay particles in unit volume of soil mass in one row or one column
$\bar{N}_v$	Number of vertically oriented idealized clay particles in unit area
$N_{v1}, N_{v2} \ldots N_{v_n}$	Number of vertically oriented swelling soil particles in each unit height of column below interface, upto 'no volume change' depth
n_1, n_2	Exponential coefficients
p	Percentage of 2 micron clay fraction in the soil
q_{sw}	Swelling pressure of soil mass
q_{swi}	Swelling pressure of an idealized individual clay particle

$q_{swi_{max}}$	Maximum swelling pressure exerted by an idealized individual clay particle
S	Total swelling (heave) of soil mass at the surface
s	Shear strength
γ_d	Dry unit weight of soil
Δ_v	Volumetric change per unit volume of soil mass
Δ_{vi}	Volumetric change per particle
ϵ	Swelling of an idealized individual clay particle
$\epsilon_1, \epsilon_2, \epsilon_n$	Average swelling of an idealized individual clay particle in each unit height of column upto 'no volume change' depth
ϵ'	Swelling of an idealized individual particle in the voids
$\epsilon_1', \epsilon'_2, \ldots \epsilon'_n$	Average swelling of an idealized individual clay particle taking place in the voids in each unit height of column upto 'no volume change' depth below interface
$\bar{\epsilon}$	Swelling of an idealized individual particle over and above the voids
ϵ_{max}	Maximum possible value of swelling of an idealized individual clay particle
$\theta, \theta_1, \theta_2$	Angle of shear planes with the horizontal
μ	Micron
ϕ	Angle of internal friction of soil