Experimental Studies on the Combined Effects of CNS Layer and Surcharge Load on the Behaviour of Underlying Expansive Soil

By

J.M. Kate*

R.K. Katti**

Introduction

Civil engineers all over the world are well aware of the uncertainties associated with expansive soils both as materials of construction as well as foundation. Attempts are continuously being made by research workers and field engineers to understand the behaviour of these soils with a view to suggest solutions for the problems arising from these deposits.

Although much of the work relating to vertical swelling pressure of these soils have been done, very little attention has been given on the aspect of lateral swelling pressures. Kassiff and Zeitlen (1962) reported the lateral swelling pressures exerted on conduits in these soils. Zeitlen and Komornik (1965) dealt with an aspect of measurement of lateral swelling pressure. Katti et al (1969 b) in their studies on depth effects in expansive soils showed that the magnitude of lateral pressures developed in these soils are altogether different from those observed in case of ordinary cohesive soils. Certain studies have also been conducted (Joshi, 1978) to understand the effect of surcharge on the development of lateral pressures in black cotton soil samples.

Katti et al (1967, 1970, 1973) conducted both laboratory and field studies on these soils and indicated that cohesive nonswelling soil (*CNS*) layer may prove to be effective in resisting swelling and swelling pressure of these soils. In these studies the effectiveness of *CNS* layer was mainly studied with respect to the vertical swelling and swelling pressures.

In view of this, in the present investigation it has been attempted to evaluate the combined effects of *CNS* layer and surcharge loads on the behaviour of underlying expansive soils, particularly on the development of lateral pressures. The analysis of the results has been carried out in the light of micro-particle approach suggested by Kulkarni and Katti (1973). Conceptual microfootings model suggested by Kate and Katti (1980) and shear planes mobilized in *CNS* layer due to these footings have been extensively used for the qualitative explanation of the results.

Test Programme

The soils used, equipment designed and fabricated, test procedure

---

* Lecturer, Civil Engineering Deptt., I.I.T. Delhi, New Delhi-110016.

** Professor of Civil Engineering and Head, Centre of Studies in Resources Engineering, I.I.T. Bombay-400076.

This paper was received in December 1980, and is open for discussion till the end of September 1981.

adopted and the parameters studied in the present work are given below.

Soils

The expansive soil used is the 'black cotton soil' from Poona area and the soil used as cohesive nonswelling soil (*CNS*) is the locally available soil known as 'Powai murum'. Poona black cotton soil is derived from Basalt and the minerals present in its clay fraction are Montmorillonite and combination of Montmorillonite and Illite. Powai murum pedologically falls under the category of tropical red or yellow earth and the prominent mineral present in it is Kaolinite. Various properties of both these soils are given in Table 1.

Experimental set-up

The equipment used for the present investigation is shown in Figure 1. It consists of a mild steel container tank of inner dimensions 30 cm $\times$ 30 cm $\times$ 52 cm fabricated out of 2.5 cm thick m.s. plates. The container tank is rigidly fitted to the channel base anchored into the floor. A reaction support is provided at the other end of channel base. The set-up shows the lever arrangement to facilitate the application of dead load surcharge on the soil system. Loading jack and proving ring system is connected firmly to the reaction support.

The movement of 80 mm dia. piston is guided horizontally and smoothly by a cylindrical sleeve fitted to the container tank. The double collar arrangement controls the movement of the piston. While placing and compacting the soils in the container, the piston head could be brought in face with the vertical inner surface of the tank wall by adjusting the inner collar and it could be locked in position by adjusting the outer collar. The other end of the piston is connected by ball-point contact to proving ring loading jack system. Thus after releasing the lock, the lateral swelling pressure exerted by soils on the piston head is transmitted to the proving ring. The upward movements (vertical swelling) due to swelling of soils can be recorded by dial gauges attached to the loading frame. Schematic diagram of experimental set-up is shown in Figure 2.

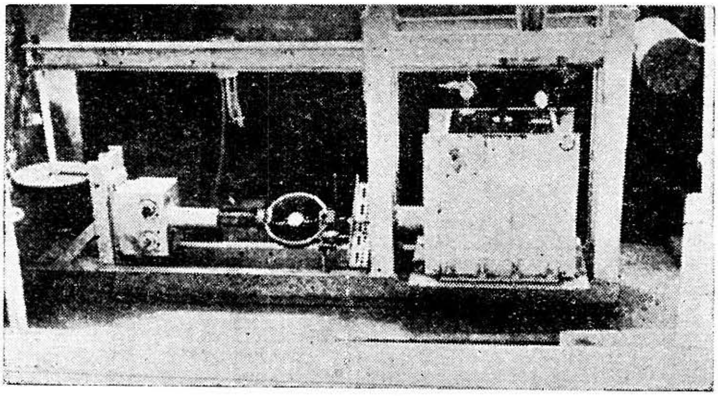


FIGURE 1 Overall view of the experimental set-up

TABLE 1

Soil Properties

Properties	Poona black cotton soil	Powai murum
1. Physical Properties		
Liquid limit (%)	81.5	48.2
Plastic limit (%)	43.2	27.2
Plasticity Indix (%)	38.3	20.9
Shrinkage limit (%)	9.1	18.3
Specific gravity at 25°C	2.78	2.71
2. Engineering Properties		
Standard Proctor density (g/cc)	1.35	1.47
Optimum moisture content (%)	28.9	26.1
3. Textural Composition		
Gravel (> 2.00 mm)	nil	4.0
Sand (2.00 mm to 0.06 mm)	12.0	43.0
Silt (0.06 mm to 0.002 mm)	32.0	35.0
5 micron clay	68.0	23.0
2 micron clay	56.0	18.0
1 micron clay	47.0	6.0
Textural classification (USBPR system)	Clay	Clay loam
Engineering classification (AASHO system)	A-7-5 (20)	A-7-6 (7)
4. Chemical Analysis		
pH	8.50	7.00
Organic matter content (%)	0.96	0.25
Carbonate content (%)	4.35	Nil
SiO_2 (%)	52.13	51.90
Al_2O_3 (%)	18.11	15.63
MgO (%)	3.86	2.40
Fe_2O_3 (%)	1.77	16.92
Loss on ignition (%)	9.62	10.51
Base Exchange Capacity (meq/100 g)		
(*i*) # No 200 Sieve fraction	71	32
(*ii*) 5 micron fraction	105	43
(*iii*) 2 micron fraction	123	49

Experimental procedure

All the inside faces of container tank were coated with a thin layer of silicon grease and then were covered with polythene paper to minimize side friction. Air dry black cotton soil passing through 2.36 mm aperture IS Sieve and hygroscopic moisture content of 9 ± 1 per cent was compacted in container tank in layers of 2.5 cm thickness at a dry density of 1.39 ± 0.01 g/cc. This dry density corresponds to the void ratio of 1.0. The black cotton soil was compacted upto a total thickness of 17.5 cm.

Cohesive nonswelling soil (*CNS*) with a constant moisture content of 26 ± 1 per cent and dry density of 1.32 ± 0.01 g/cc (corresponds to void ratio of 1.05) was compacted in layers of 2.5 cm thickness upto desired thickness. To expedite saturation of the soils, a thin layer of standard sand

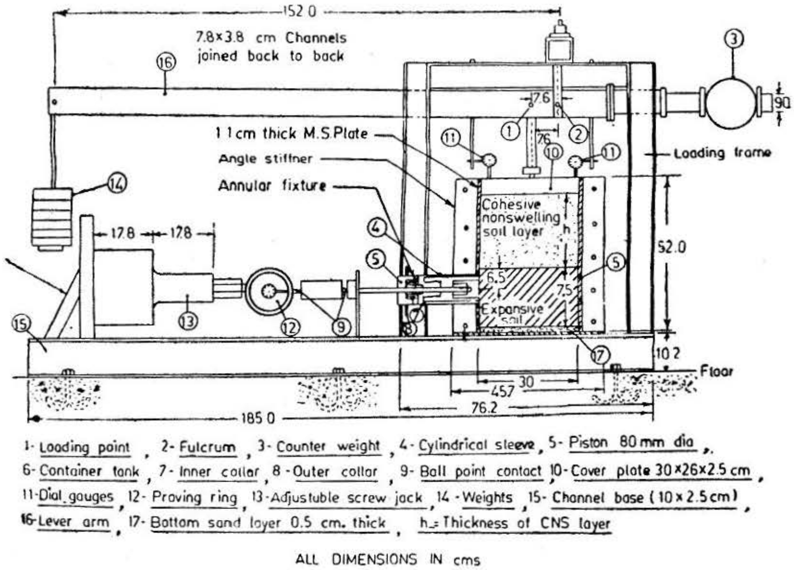


FIGURE 2 Schematic arrangement of the set-up

was placed all around and also at bottom of soil mass in the container during compaction process. The soil surface was properly levelled.

The levelled soil surface was covered with filter paper and a mild steel cover plate was placed on it. At each corner of cover plate a dial gauge with suitable extension was placed for recording the vertical movements. Proving ring was brought in contact with the piston rod by adjusting the play of the jack. Initial readings of all the dial gauges and proving ring were recorded. The piston was released from the locking system to have the movement to allow transfer of pressure from soil to proving ring. The lever arrangement was set in position and the desired surcharge load was transferred to the soil system through cover plate.

Soils were allowed to saturate by circulating the water continuously through water inlet. The observations for lateral pressures and vertical movements were recorded simultaneously and continuously during saturation. Whenever dial gauges and proving ring exhibits no change in readings when observed continuously for several days is an indication of full saturation of the soils. At the end of saturation period which ranged from 5 weeks to 10 weeks (depending upon the magnitude of dead load surcharge) final readings of proving ring and dial gauges were taken.

During removal of soils in layers from container tank, representative soil samples required for determination of density and moisture content were collected at a depth interval of 2.5 cm. Special stainless steel cutters were used for this purpose. Recorded values of lateral pressures were corrected for the error due to compression in proving ring by adopting the procedure similar to that employed by Katti et al (1967, 1969 a) and the corrected values are reported here.

Initial densitites and moisture contents during compaction in both the soils were achieved under utmost controlled conditions.

Parameters varied

In all the experiments conducted, the black cotton soil was compacted at initial dry density and moisture contents of 1.39 ±0.01 g/cc and 9 ± 1 per cent respectively. Similarly, *CNS* layers were compacted at a dry density of 1.32 ± 0.01 g/cc and moisture contents of 26 ± 1 per cent. The thickness of black cotton soil was kept contant at 17.5 cm for all the experiments.

The thicknesses of *CNS* layer varied are 0, 5, 10, 15, 20, 25 and 30 cm. For each thickness of *CNS* layer studied, the dead load surcharges varied are 0, 0.01, 05, 0.1, .0.5, 1.0, 2.0, 3.0 and 4.0 kg/cm². The ranges of dead load surcharges selected for study are such that these falls within and beyond the magnitude of swelling pressure at 'no volume change condition' of Poona black cotton soil. Every time a separate experiment was conducted for each of the magnitude of parameters varied.

It may be mentioned here that the present experiments conducted were intended mainly for comparative study as the results were likely to be affected by boundary effect owing to the small dimensions of the experimental set-up. However, utmost precautions were taken to minimize side wall friction between the soils and container.

Supplementary tests

To understand the changes in shear strength of *CNS* layer brought out by the dead load surcharges, a separate study was conducted. In these experiements *CNS* layer was compacted at a dry density of 1.32± 0.01 g/cc and moisture content of 26 ± 1 per cent in layers upto a thickness of 15 cm. The soil was allowed to saturate under various surcharges mentioned above. At the end of saturation, in-situ vane shear strength of soil was determined by laboratory vane. Samples were collected for conducting unconfined compression tests and direct shear tests. For each dead load surcharge separate experiment was conducted. Similar experiments were also conducted on black cotton soil compacted at a dry density of 1.39 ± 0.01 g/cc and initial moisture content of 9 ± 1 per cent. Direct shear tests were conducted at a deformation rate of 0.0125 mm/min. and shear parameters were obtained.

Results and Discussions

The results of various experiments conducted are given below.

Supplementary studies

The results of the supplementary studies are presented in Table 2 for both the soils

Cohesive nonswelling soils

The vane shear strength increases from 0.30 kg/cm² to 0.42 kg/cm² as the surcharge load under which cohesive nonswelling soil is allowed to saturate ranged from 0 to 4.0 kg/cm². It may be mentioned here that the vane shear strengths are obtained immediately after the removal of dead load surcharge from soil surface. Thus these values reflect the alterations

TABLE 2

Results of Shear Tests on Soils

Type of shear test	Soil	Dead load surcharge (kg/cm²)								
		0	0.01	0.05	0.1	0.5	1.0	2.0	3.0	4.0
Vane shear strength (Kg/cm²)	Cohesive nonswelling	0.30	0.30	0.31	0.33	0.35	0.365	0.39	0.43	0.49
	Expansive	0.18	0.24	0.32	0.37	0.43	0.48	0.59	0.66	0.78
Direct shear										
c' (kg/cm²)	Cohesive nonswelling	0.20	0.20	0.20	0.205	0.205	0.21	0.22	0.24	0.26
ϕ'		18°-30′	18°-30′	18°-30′	18°-45′	19°-15′	20°-00′	21°-00′	22°-00′	23°-30′
c' (kg/cm²)	Expansive	0.22	0.22	0.22	0.23	0.23	0.24	0.26	0.29	0.51
ϕ'		19°	19°	19°	19°	18°-30′	18°	17°-30′	16°-30′	11°-30′
$q_u = \frac{1}{2}\,UCS$	Cohesive nonswelling	0.25	0.26	0.28	0.30	0.33	0.37	0.38	0.41	0.45
(kg/cm²)	Expansive	0.24	0.26	0.31	0.39	0.48	0.51	0.64	0.71	0.83

in vane shear strength due to the compression of soil caused by surcharge loads.

The results of the direct shear tests indicate that the soil undergoes increase in both cohesion (c') and angle of internal friction (ϕ'). The values of c' ranged from 0.20 kg/cm² to 0.26 kg/cm² and those of θ' from 18°-30′ to 23°-30′ respectively when the surcharge loads increased from 0 to 4.0 kg/cm². Further, the shear strengths evaluated from unconfined compression strength tests ranged from 0.25 kg/cm² to 0.45 kg/cm² for the dead load surcharges ranging from 0 to 4 kg/cm².

Expansive soil

The vane shear strength increased from 0.18 kg/cm² to as high as 0.78 kg/cm² when the surcharge load ranged from 0 to 4 kg/cm². The results of the direct shear test are interesting indicating different magnitudes of shear parameters for surcharge loads within and beyond swelling pressure range. It can be seen from Table 2 that the changes in c' and ϕ' are not significant upto the surcharge load of 3 kg/cm². However, the value of c' and θ' indicates a significant changes for a surcharge load of 4.0 kg/cm². It may be mentioned here that the vertical swelling pressure at 'no volume change' condition for this soil is around 3.25 kg/cm². With increase in surcharge load the values of c' increases but ϕ' decreases. The shear strength (q_u) obtained from *UCS* test increases from 0.24 kg/cm² when the surcharge load increases from 0 to 4 kg/cm².

Development of lateral pressure

The data for the lateral pressures obtained under various surcharge loads and thicknesses of *CNS* layer is presented in Table 3. The variation of lateral pressure with dead load surcharge for various thicknessses of *CNS* layer is shown in Figure 3. It can be clearly seen that the surcharge load has considerable effect on the development of lateral pressure. For any particular thickness of *CNS* layer lateral pressure increases with

TABLE 3

Lateral Pressure Results

Thickness of *CNS* layer (cm)	Lateral pressure (kg/cm²)								
	Dead load surcharge (kg/cm²)								
	0	0.01	0.05	0.1	0.5	1.0	2.0	3.0	4.0
0	0.73	0.83	0.98	1.091	1.81	2.26	2.87	3.22	3.44
5	1.21	1.31	1.45	1.48	2.20	2.53	3.04	3.31	3.56
10	1.60	1.68	1.76	1.83	2.48	2.77	3.22	3.43	3.69
15	1.95	2.02	2.08	2.17	2.71	3.01	3.26	3.50	3.73
20	2.20	2.27	2.34	2.42	2.90	3.12	3.34	3.58	3.77
25	2.41	2.49	2.55	2.65	2.98	3.25	3.42	3.65	3.85
30	2.64	2.68	2.73	2.81	3.13	3.27	3.49	3.73	3.97

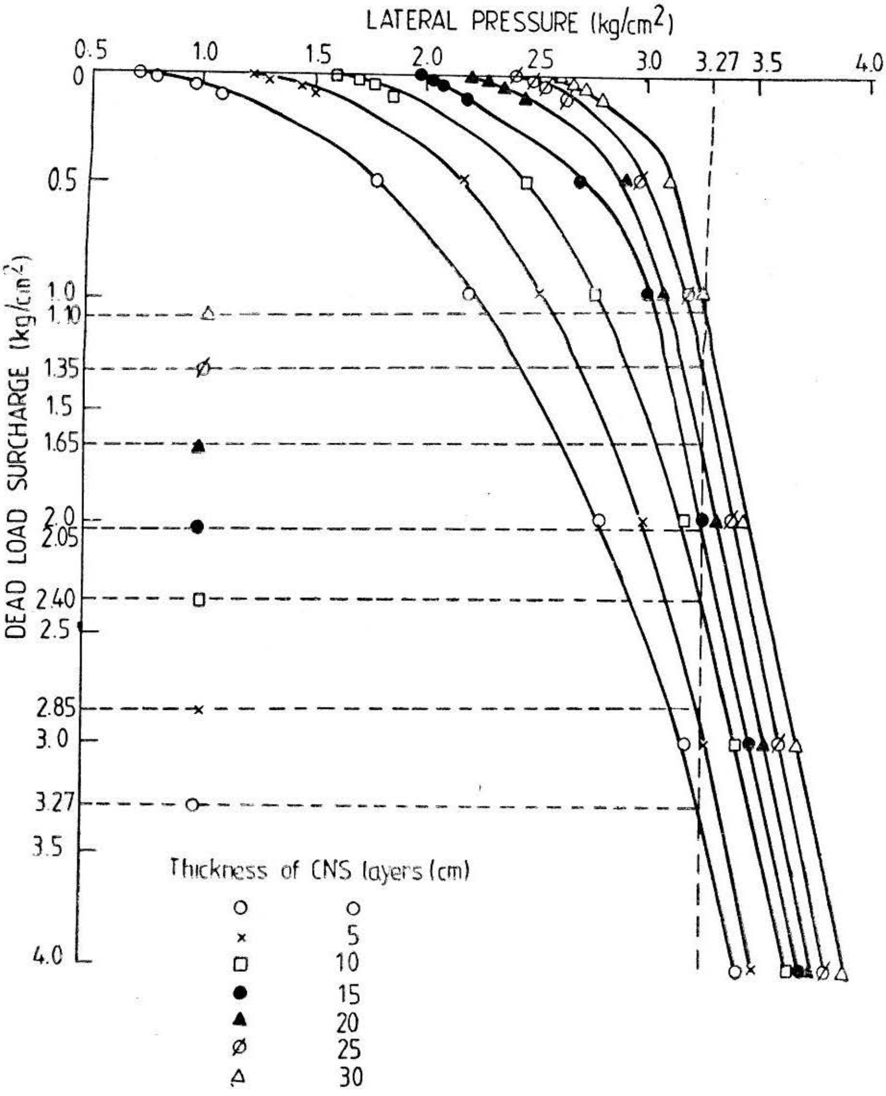


FIGURE 3 Variation of lateral pressure with surcharge loads

increase in surcharge loads. Initially the increase is curvilinear which becomes linear after certain magnitude of dead load surcharge. This trend has been observed for all thicknesses of *CNS* layer studied. The magnitude of dead load surcharge at which this variation becomes linear depends upon the thickness of *CNS* layer. Higher the thickness of *CNS* layer less is the surchage load beyond which the variation becomes linear.

An attempt has been made from Figure 3 to obtain the contribution of dead load surcharge and *CNS* layer in the development of lateral pressures. The 'no volume change' swelling pressure of the soil, studied in the present work, under the same condition of initial compaction density and moisture content; is around 3.27 kg/cm². The dotted line corresponding to zero thickness of *CNS* layer represents 'no volume change' swelling pressure line. Similarly, the 'equivalent no volume change' swelling pressure lines are indicated for various thicknesses of *CNS* layer in the same figure. The 'equivalent no volume change' swelling pressure, for a particular thickness of *CNS* layer, indicates the magnitude of dead load surcharge (applied at the top of *CNS* layer), required to allow no change in volume of expansive soil (underlying *CNS* layer), compacted initially at certain density and moisture content; in a saturated condition. The magnitudes of 'equivalent no volume change' swelling pressure, evaluated from Figure 3 are 2.85, 2.40, 2.05, 1.65 1.05 and 1.10 kg/cm² for the thicknesses of *CNS* layer of 5, 10, 15, 20, 25 and 30 cm respectively. The lateral pressure development, due to increase in dead load surcharge, over and above the swelling pressure at 'equivalent no volume change' condition, shows a linear triangular distribution as illustrated, in Figure 3. The results, thus clearly indicate that, the application of dead load surcharges within swelling pressure range and dead load surcharges beyond swelling pressure range, have altogether different effects, on the development of lateral pressure.

Further an attempt is made, to understand the effect of different thicknesses of *CNS* layer in generating the lateral pressure, by examining the curves in the following way. As such, the equivalent dead weights of *CNS* layer of thicknesses 5, 10, 15, 20, 25 and 30 cm are 0.0075, 0.015 0.0225, 0.03, 0.0375 and 0.045 kg/cm² respectively. Under a dead load surcharge of 0.5 kg/cm², if dead weight is the criteria, the loads acting on the surface of expansive soil would be 0.5075, 0.515, 0.5225, 0.53, 0.5375 and 0.545 kg/cm² for the thicknesses of 5, 10, 15, 20, 25 and 30 cm respectively. If, the presence of *CNS* layer is similar to the dead load, then the lateral pressures as obtained from curves in Figure 3 are around 1.83, 1.85, 1.86, 1.87, 1.77 and 1.89 kg/cm² for the dead loads of 0.5075, 0.515, 0.5225, 0.53, 0.5375 and 0.545 kg/cm² respectively. However, under 0.5 kg/cm² dead load surcharge, the actual lateral pressures observed are 2.20, 2.48, 2.71, 2.90, 2.98 and 3.13 kg/cm² for the thicknesses of 5, 10 15, 20, 25 and 30 cm respectively. In other words, the actual lateral pressures developed are much higher than those evaluated for equivalent dead load condition.

The ratios, between the additional lateral pressure (over and above that developed for no dead load surcharge condition) generated under each dead load surcharge and the corresponding dead load surcharge, as computed for various thicknesses of *CNS* layer are given in Table 4. It can be seen that, the ratio decreases with increase in dead load surcharge as well as increase in thickness of *CNS* layer.

Table 5 reports the ratios between lateral pressure developed over and above 'no volume change' swelling pressure and corresponding dead load surcharge over and above 'equivalent no volume change' swelling pressure. These ratios on an average are 0.233, 0.259, 0.264, 0.239, 0.214, 0.227 and 0.242 for the thicknesses of 5, 10, 15, 20, 25 and 30 cm respectively. It may be noted that, these ratios show more or less the same value.

Nature of Vertical Movement

The varitation of vertical movement with dead load surcharges for various thicknesses of *CNS* layer is given in Figure 4. For any particular thickness of *CNS* layer, the increase in dead load surcharge decreases the vertical movement as shown in Figure 4 e.g. under 10 cm thickness of *CNS*

TABLE 4

The Ratio between the additional lateral pressure developed and corresponding dead load surcharge

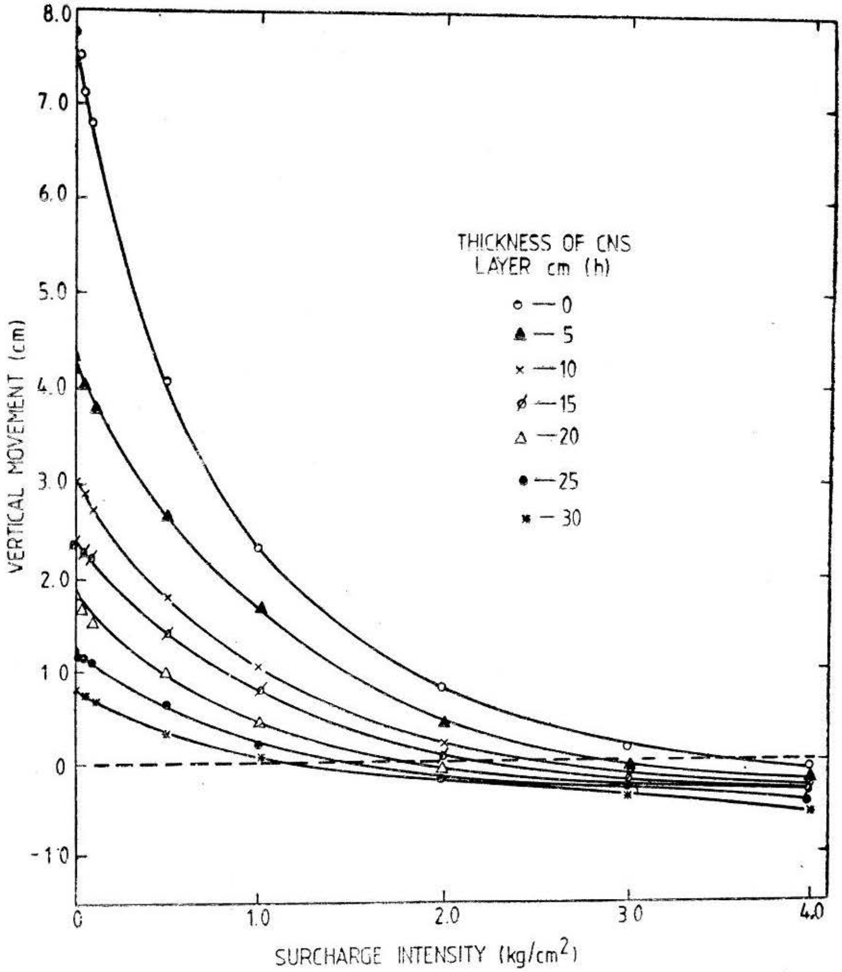
Thickness of *CNS* layer (cm)	Dead load surcharge (kg/cm²)							
	0.01	0.05	0.1	0.5	1.0	2.0	3.0	4.0
0	10	5.0	*3.6*	*2.16*	*1.53*	*1.07*	*0.83*	*0.68*
5	10	4.8	2.7	1.98	1.32	0.915	0.70	0.585
10	8	3.2	2.3	1.76	1.17	0.81	0.61	0.52
15	7	2.6	2.2	1.52	1.06	0.655	0.515	0.445
20	7	2.8	2.2	1.40	0.92	0.57	0.46	0.39
25	8	2.8	2.4	1.14	1.11	0.505	0.41	0.36
30	4	1.8	1.7	0.98	0.63	0.425	0.365	0.33

TABLE 5

The ratio between the lateral pressure generated over and above 'No Volume Change' swelling pressure and the dead load surcharge over and above 'Equivalent no volume change' swelling pressure

Thickness of *CNS* layer (cm)	'Equivalent no volume change' swelling pressure (kg/cm²)	Ratio			
		Dead load surcharge (kg/cm²)			Average ratio
		2.0	3.0	4.0	
0	3.27	—	—	0.233	0.233
5	2.84	—	0.267	0.252	0.259
10	2.40	—	0.266	0.262	0.264
15	2.05	—	0.242	0.236	0.239
20	1.65	0.20	0.23	0.212	0.214
25	1.35	0.231	0.23	0 219	0.227
30	1.10	0.245	0.24	0.242	0.242

The 'no volume change' swelling pressure af the expansive soil, compacted at initial dry density of 1.39 ± 0.01 g/cc and hygroscopic maisture content of 9 ± 1 per cent is 3.27 kg/cm².



FICURE 4 Vertical movement Versus dead load surcharge

layer, the vertical movements observed for the dead load surcharges, of 0, 0.01, 0.05, 0.1, 0.5, 1.0, 2.0, 3.0 and 4.0 kg/cm² are 3.06, 3.01, 2.93, 2.75, 1.80, 1.02, 0.18, -0.11 and -0.26 cm respectively. Similarly, increase in thickness of *CNS* layer under any particular dead load surcharge decreases the vertical movement.

Density variation with depth in expansive soil

The observations for density and moisture contents in expansive soil were taken at depth interval of 2.5 cm after saturation. The moisture contents observed at various depth under different experimental conditions shows roundabout 100 per cent saturation.

The variation of dry density with depth under various dead load surcharges has been shown in Figure 5 for typical thicknesses of *CNS* layer. Figure 5 (*a*), 5 (*b*) and 5 (*c*) shows these variation for 0,15 and 30 cm thickness of *CNS* layer ressectively.

It may be seen, in case of dead load surcharges having magnitudes lower than 'no volume change' swelling pressure that, the density of expansive soil is lowest at the interface, and it increases with depth. For the dead load surcharge of 1 kg/cm^2, the densities of expansive soil observed under 15 cm thick *CNS* layer are 1.32, 1.365, 1.38, and 1.395 g/cc at the depths of 0, 6.25, 11.25 and 16.25 cm below interface respectively. Whereas, for the surcharge higher than 'no volume change' swelling pressure, the density remains more or less constant with depth, and the magnitude of density at any depth is always higher than the initial compaction density. Thus, indicating the settlement of soil system instead of swelling.

Density of expansive soil at the interface

The values of density of expansive soil at the interface are evaluated from the density versus depth curve shown in Figure 5. These values of interface densities are plotted against dead load surcharge for different thicknesses of *CNS* layer as indicated in Figure 6. It can be seen from the figure that the density at the interface increases with increase in dead load surcharge as well as increase in thickness of *CNS* layer until, it attains to initial compaction value. It can be further seen that for dead load surcharge higher than 'no volume change' swelling pressure, the density at the interface is more than the initial compaction density.

In summary, the results of various tests show that, cohesive non-swelling soil layer underlying dead load surcharge, brings the alterations in the development of swelling pressure of expansive soil, due to internal characteristics. For dead load surcharges beyond swelling pressure range, studies do indicate that, the expansive soil system starts behaving as an ordinary soil system.

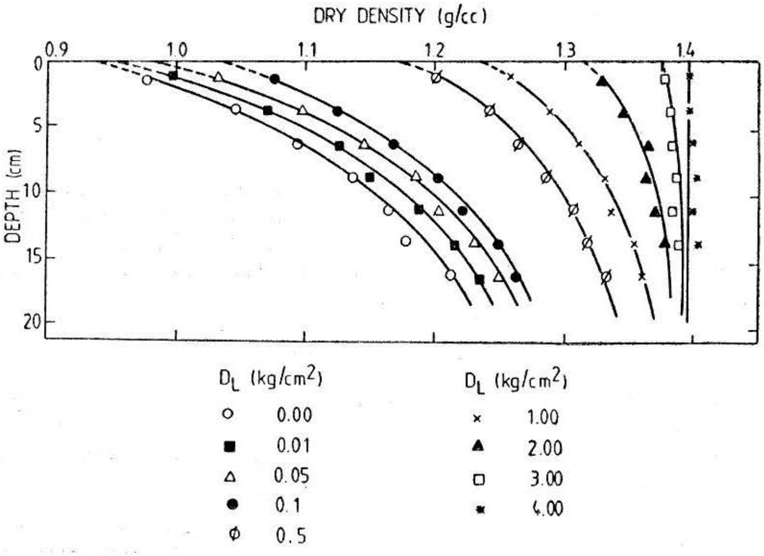


FIGURE 5 (*a*) Density variation with depth without *CNS* layer

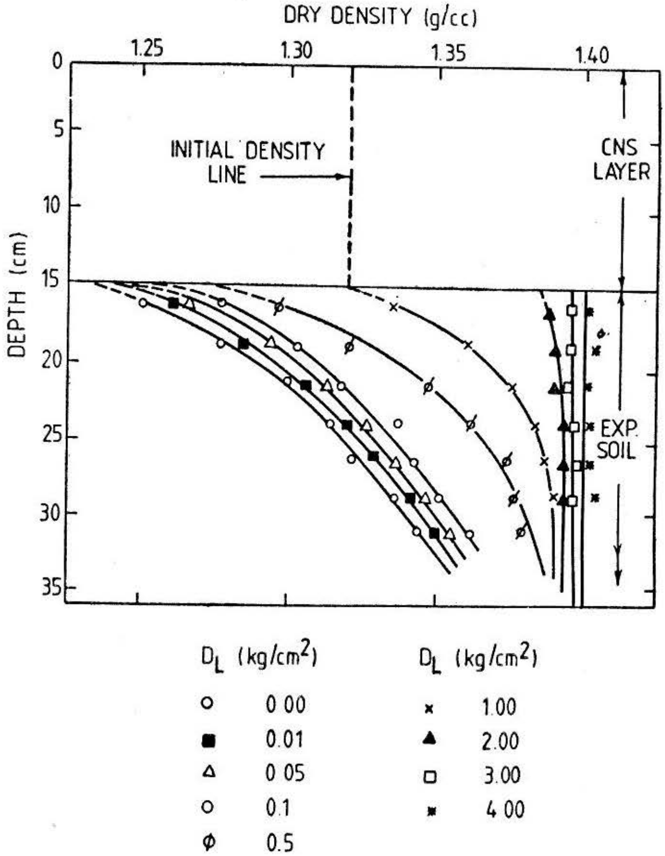
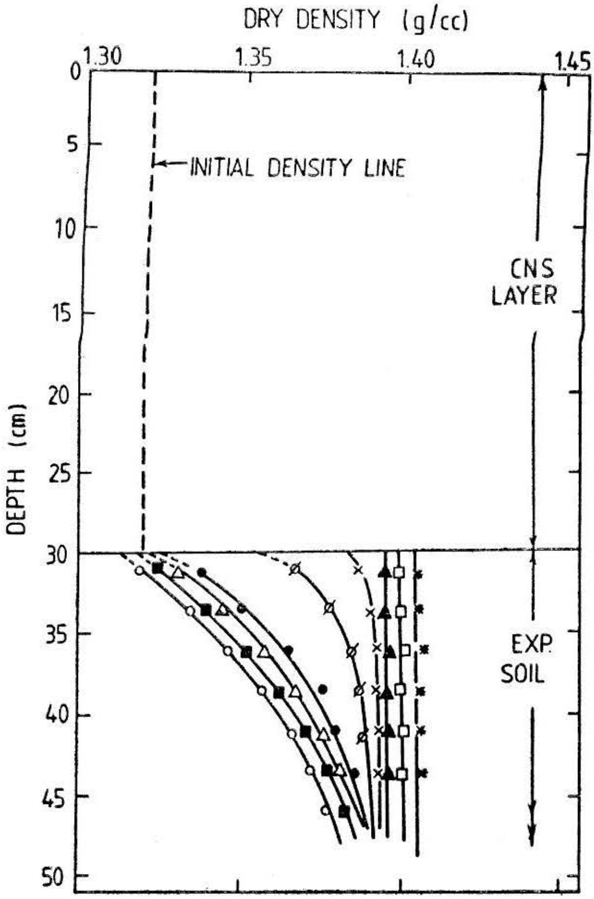


FIGURE 5 (*b*) Density variation with depth for 15 cm thick *CNS* layer

Analysis

The experimental studies clearly indicated that surcharge loads and *CNS* layer both have a significant contributions in the development of lateral pressures in an underlying expansive soil mass. From the results it also appeared that not only the weight of *CNS* layer, but to a large extent its internal characteristics is responsible for the development of lateral pressures. Herein an attempt has been made to analyse the results obtained in this study in the light of micro particle approach (Katti et al 1969 *b*, Kulkarni and Katti, 1973, Kate and Katti, 1975, 1980).

The configuration as adopted in micro-particle and micro-anchor approach (Kulkarni and Katti, 1973) to evaluate the number of particles and their spacings in an idealized system is used in the present analysis. Figure 7 shows such idealized system in which each clay particle is assumed of a cubical shape with each side equal to l and the spacing between the adjacent particles with 'c' axes oriented in mutually perpendicular directions along the row or column is d. So that the spacings between like oriented particles in a row or column is D ($= 3d$). The nonswelling particles which occupies the positions in between these clay particles have not been shown in the figure. With this configuration the number of



D_L (kg/cm²)		D_L (kg/cm²)	
○	0.00	×	1.00
■	0.01	▲	2.00
△	0.05	□	3.00
●	0.10	*	4.00
ϕ	0.5		

FIGURE 5(c) Density variation with depth for 30 cm thick *CNS* layer

idealized clay particles with their '*c*' axes oriented in vertical direction ($\bar{N}_v$) and in lateral direction ($\bar{N}_L$) in unit area can be obtained by

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

in which,

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

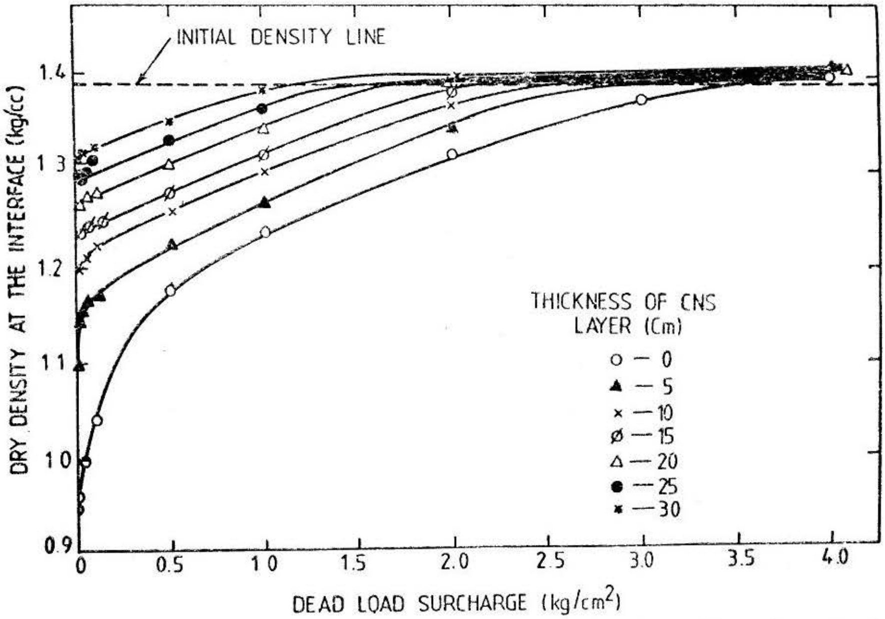


FIGURE 6 Variation of density of expansive soil at the interface with surcharge load

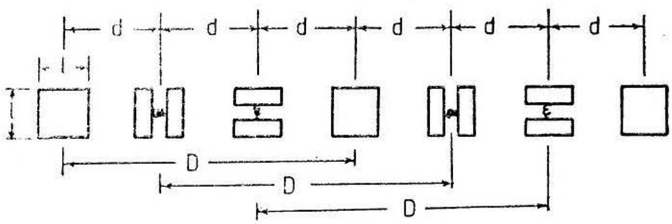


FIGURE 7 Arrangement of idealized clay particles in an idealized system

e = void ratio of the soil mass.

The swelling pressure of an idealized individual clay particle, can be determined from the known value of swelling pressure measured at the surface of soil mass (Kulkarni and Katti, 1973)

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

where

q_{sw_i} = Swelling pressure of an idealized individual clay particle.

q_{sw} = Swelling pressure of soil mass.

Equilibrium of vertically oriented particle

At the interface between *CNS* layer and expansive soil, vertically oriented swelling soil particles of 2 microm size were visualized by Kate & Katti (1980) as though footings of 2 micron size resting on *CNS* layer in an inverted fashion. With the aid of these conceptual footings and assuming the shear surfaces generated in *CNS* layer in the form of frustum of an inverted regular pyramid Kate and Katti (1980) obtained the following equilibrium equation

$$q_{swi} \times l^2 = A \times c_u \times \sin\theta \qquad \ldots(3)$$

A it the area of shear surface on which developed cohension c_u of *CNS* layer is mobilized and it would consist 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 horizontal, then

$$A = 4\left[\frac{\bar{h}}{\sin\theta}(l+\bar{h}\cot\theta)\right] \qquad \ldots(4)$$

From these equations $\bar{h}$ is determined by

$$\bar{h} = -\frac{l}{2\cot\theta}\left[1 \pm \sqrt{1+3\cot\theta\frac{q_{sw}}{c_u}\left(\frac{1+e}{p}\right)^{2/3}}\right] \qquad \ldots(5)$$

Further, they also gave an expression for interference number (N_I)

$$N_I = \frac{h}{\bar{h}} \qquad \ldots(6)$$

wherein,

N_I is 'Interference Number' or number of particles with their shear planes interferring mutually and h is the actual thickness of *CNS* layer encountered in the process of balacing the swelling pressure of soil mass.

Vertically and Horizontally oriented particles co-existing

In an idealized expansive soil mass as visualized here, the vertically and horizontally oriented particles are lying side by side as illustrated in Figure 7. When the soil mass comes in contact with moisture, these particles develop swelling and try to expand along their respective '*c*' axes. Thus horizontally oriented particles may try to push against the vertically oriented particles and vertically oriented particles try to move in the upward direction at right angles to horizontally oriented particles. As a result of which these particles experience differential movement which in turn creates shear drag between these particles as indicated in Figure 8.

Thus, the movement of vertically oriented swelling particle can be visualized being resisted by

(*i*) Body weight of overlying *CNS* layer

(*ii*) Shearing stresses caused due to the drag created by horizontally oriented swelling particles on the four vertical faces and

(*iii*) Shearing stresses generated in *CNS* layer.

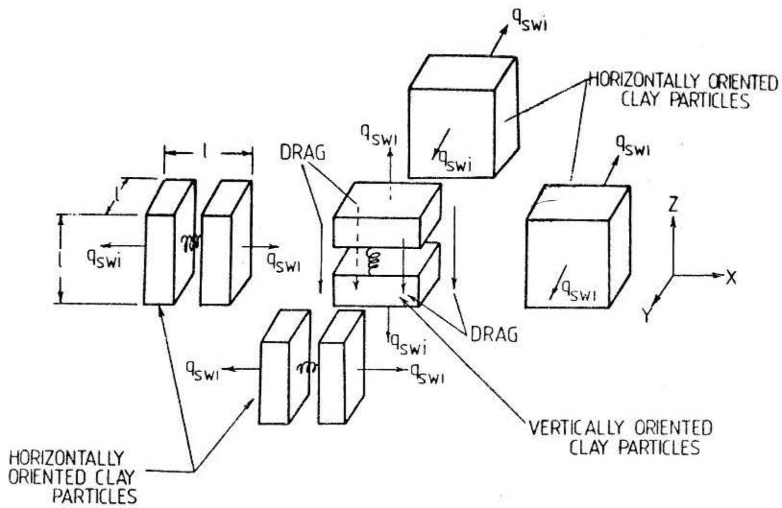


FIGURE 8 Forces acting on vertically oriented clay particles in an idealized system

Hence, the equilibrium of vertically oriented particle under these forces can be expressed by an equation

$$q_{swi}\times l^2 = \gamma_{sub}\times h\times l^2+4l^2\times q_{sw_i}\times \tan\phi_s+A\times c_u\times \sin\theta \qquad ...(7)$$

in which,

γ_{sub} is the submerged unit weight of *CNS* layer and $\tan\phi_s$ is the coefficient of skeleton friction.

Studies with dead load surcharges

The application of dead load surcharge (D_L) which acts vertically downwards, results into preventing the swelling of particles with their '*c*' axes oriented in vertical direction. Depending upon the magnitude of dead load surcharge, the swelling of these particles would be prevented partially, fully or even the compression of the soil system may take place as illustrated in Figure 9.

Assuming an idealized arrangement of particle orientation, the dead load surcharge would be shared up, only by vertically oriented particles as long as they are in the state of swelling. With this consideration, the following equation is worked out by equating, the dead load surcharge acting on unit area (D_L) with the product of dead load surcharge acting on an individual particle (D_{L_i}) and number of vertically oriented particles in unit area ($\overline{N}_v$). The equation is

$$D_L\times 1\times 1 = D_{L_i}\,(l\times l)\times \overline{N}_v \qquad ...(8)$$

Substituting the value of $\overline{N}_v$ in the above expression, it gives,

$$D_{L_i} = \frac{D_L}{\overline{N}_v\, l^2} = \frac{D_L}{\frac{1}{3\,l^2}\left[\frac{p}{1+e}\right]^{2/3}.\,l^2} = 3\,D_L\left[\frac{1+e}{p}\right]^{2/3} \qquad ...(9)$$

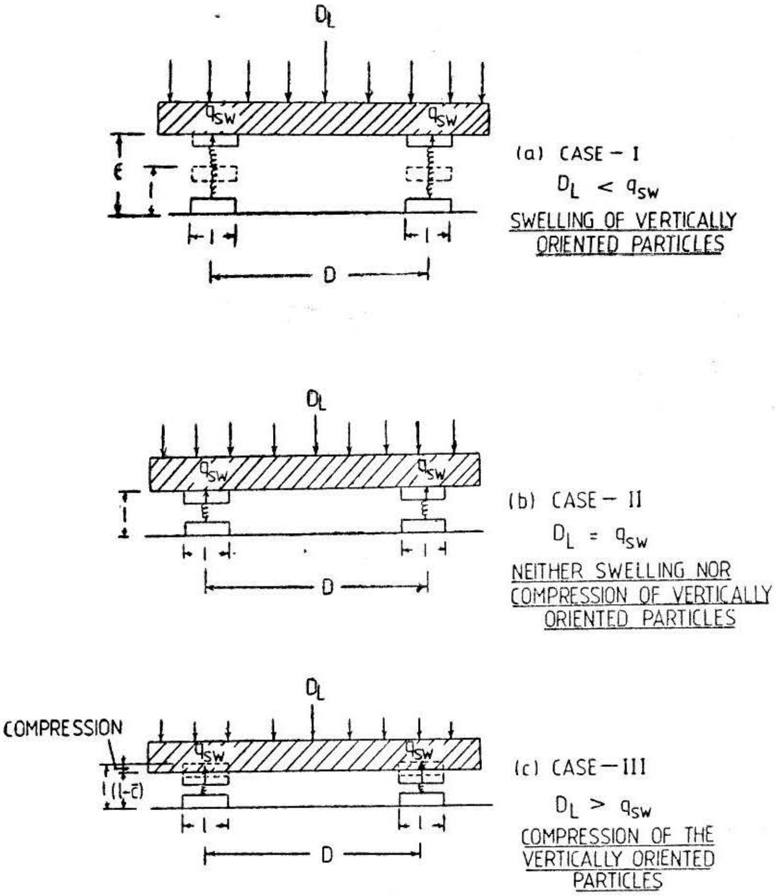


FIGURE 9 Effect of dead load surcharges of different magnitudes on vertically oriented swelling particles

Dead Load Surcharge in Combination with CNS Layer

The expansive soil is covered with *CNS* layer of thickness h_1 and the dead load surcharge is applied at the top of *CNS* layer. The initial compaction would ratio e_1 of expansive soil changes to e_2 at the interface, after saturation.

The Equation 7 which is a generalized equation for the equilibrium of an idealized vertically oriented particles, can be modified to take in to account the effect of dead load surcharge, as expressed in the following form.

$$q_{sw_i} . l^2 = \gamma_{sub}\, h_1\, l^2 + D_{Li} . l^2 + 4 l^2\, q_{swi} \tan\phi_s + A_1 . c'_u \,.\, \sin\theta_1 \qquad \ldots(10a)$$

or

$$q_{swi} . l^2 \,(1 - 4 \tan s\, \phi_s) = \gamma_{sub}\, h_1\, l^2 + D_{Li} . l^2 + A_1 . c'_u \,.\, \sin\theta_1 \qquad \ldots(10b)$$

wherein,

c'_u = Developed cohesion of *CNS* layer under the influence of dead load surcharge,

A_1 = The area of shear surfaces generated in *CNS* layer. On the basis of an assumption that, the shear surfaces generated in *CNS* layer forms a frustum of an inverted regular pyramid as shown in Figure 10, A_1 is given by

$$A_1 = 4\left[\frac{\bar{h}'}{\sin\theta_1}\left(1+\bar{h}'\cot\theta_1\right)\right]$$

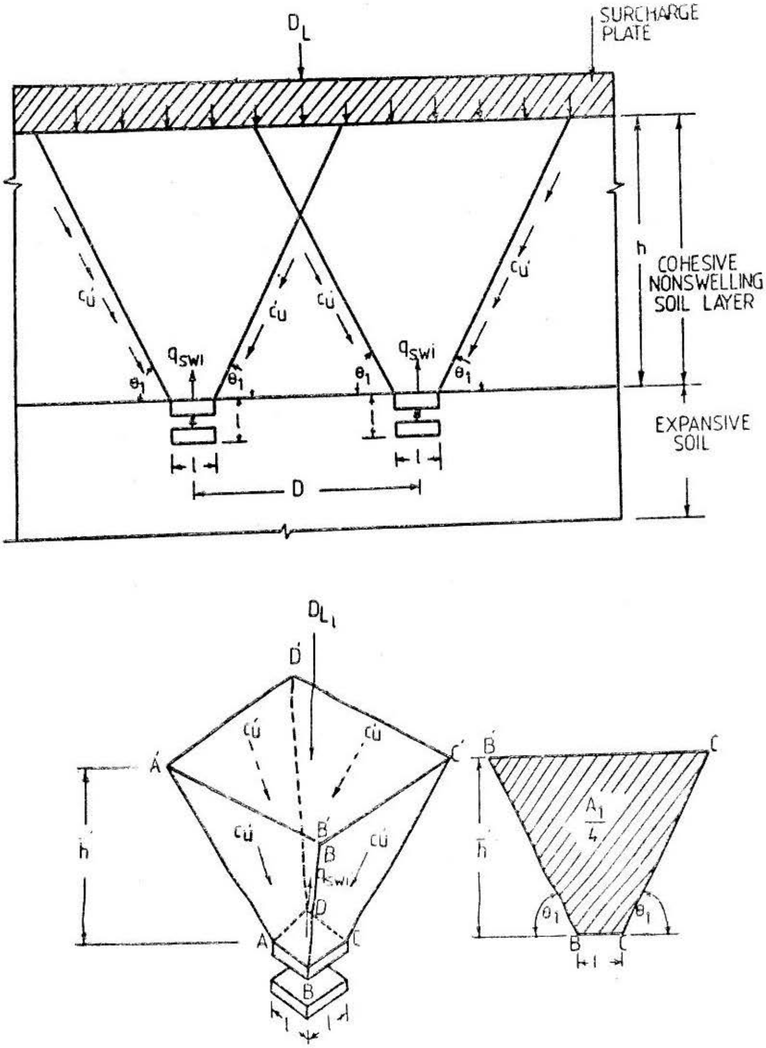


FIGURE 10 Conceptual pattern of shear surface mobilised in *CNS* layer underlying surcharge load, due to vertically oriented swelling particle

in which, $\bar{h}'$ is the vertical distance in *CNS* layer upto which shearing surfaces are mobilised, and

θ_1 = The angle made by shear surfaces with the horizontal. Introducing the value of A_1 in Expression 10*b* and solving for $\bar{h}'$; the magnitude of $\bar{h}'$ is given by.

$$\bar{h}' = -\frac{l}{2\cot\theta_1}\left[1 \pm \sqrt{1+\frac{\cot\theta_1}{c'_u}\{q_{swi}(1-4\tan\phi_s)-D_{Li}-\gamma_{sub}.h\}}\right] \quad ...(11)$$

When $\theta_1=90°$, $\bar{h}'$ is determined from Equation 10*b* as.

$$\bar{h}' = \frac{l}{4c'_u}\left[q_{swi}(1-4\tan\phi_s)-D_{Li}-\gamma_{sub}.h\right] \quad ...(12)$$

As an example, the magnitude of q_{swi} for 2 μ clay fraction comes out to be 23.0 kg/cm², as obtained by considering swelling pressure at constant volume of the order of 3.27 kg/cm² for the initial compaction void ratio of 1.0; and p=56.0 percent. The magnitude of $\bar{h}'$ thus computed for various dead load surcharges, by assuming $\theta_1=90°$, $\tan\phi_s=0.05$, c'_u for the simplicity of calculation purpose assumed to be constant at a value of 0.22 kg/cm², and neglecting the term $\gamma_{sub}.\bar{h}$ are presented in Table 6.

TABLE 6

$\bar{h}'$ values for different surcharge loads

D_L (kg/cm²)	0	0.5	1.0	1.5	2.0	2.5
D_{L_i} kg/cm²)	0	0.53	7.05	10.58	14.10	17.63
$\bar{h}'$ (micron)	41.8	33.8	25.8	17.8	9.7	1.8

The computations indicate reduction in $\bar{h}'$ with increase in D_L. The actual thickness of *CNS* layer h_1, required to balance complete swelling and swelling pressure of underlying expansive soil mass under a given dead load surcharge, would be in proportion with $\bar{h}'$. This clearly shows that, higher the dead load surcharge applied less would be the actual thickness of *CNS* layer required to achieve constant volume condition of expansive soil media.

The interference number $\left(N'_I\right)$ in this case would be

$$N'_I = \frac{h_1}{\bar{h}'} \quad ...(13)$$

Lateral Pressure at the Interface

The lateral pressure P_L, generated in a saturated expansive soil mass in the presence of *CNS* layer; would be equal to its swelling pressure q_{sw} as long as the thickness h of *CNS* layer placed on expansive soil does not exceed the thickness h_1, actually required to produce 'no volume change' in expansive soil. Thus for the case, when $h \leqslant h_1$; the magnitude of P_L in terms of q_{swi} and $\bar{N}_L$ (the number of particles, in unit area, oriented in lateral direction along which the pressure is measured) can be determined by the following expression.

$$P_L = \bar{N}_L \times q_{swi} \times l^2 \quad ...(14)$$

In case when $h > h_1$; P_L would be given by.

$$P_L = \bar{N}_L \times q_{swi_c} \times l^2 + k\gamma_{sub}\,(h - h_1) \quad ...(15)$$

wherein,

q_{swi_c} = The swelling pressure of an idealized individual particle evaluated for a constant volume condition at a given void ratio.

k = A coefficient, depending upon $c - \phi$ condition and body weight of expansive soil particles.

In a similar fashion, the magnitude of P_L developed under the combination of *CNS* layer and dead load surcharge can be obtained by Equation (14) until the value of dead load surcharge, applied at the top of *CNS* layer, does not exceed the 'equivalent no volume change' swelling pressure q_{sw_e}. In this case i.e. when $D_L \leqslant q_{sw_e}$, as the swelling force in upward direction is greater than the doward force; the dead load surcharge may not exhibit the independent effect on lateral pressure development.

When the value of D_L exceeds q_{sw_e}; the compression of the complete soil system takes place under the effect of dead load surcharge of the order of $(D_L - q_{sw_e})$, which results in generating lateral thrust in addition to the swelling pressure at constant volume condition of the soil mass. Thus, P_L in this case would be obtained in the following form,

$$P_L = \bar{N}_L \times q_{swi_c} \times l^2 + k(D_L - q_{sw_e}). \quad ...(16)$$

Conclusions

The following conclusions have been summerized from the experimental *investigation conducted to understed* the behaviour of expansive soil subjected to combined effect of overlying *CNS* layer and surcharge load.

(*i*) The cohesive nonswelling soil layer underlying dead load surcharge has considerable influence in generating the lateral pressures in the soil system. Further, it is seen that, until the lateral pressure reaches the value, more or less equal to swelling pressure at 'no volume change' condition, the lateral pressure development is influenced by the swelling pressure characteristic of the expansive soil media. However, for the loads over and above 'equivalent no volume change' swelling pressure, the soil-system seem to behave as a simple $c - \phi$ system, wherein only the shear parameters of the soil mass contribute towards the development of lateral pressure.

(*ii*) The results of vertical movements at the surface of soil system and the densities at the interface are all in agreement with the results of lateral pressures.

(*iii*) The micro-particle and micro-footings approach seems to explain qualitatively the behaviour of expansive soil mass subjected to the combined effects of *CNS* layer and dead load surcharge.

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.

References

JOSHI, R.P., (1978), "The studies on the effect of surcharge on the development of lateral pressure in Black Cotton Soil Sample", unpublished *Ph. D. thesis*, I.I.T. Bombay.

KASSIFF, G. and ZEITLEN, J.G., (1962), "Lateral swelling pressure on conduit from expansive clay black fill", *Highway Research Board Bulletin*, No. 313.

KATE, J.M., (1974), "Studies on the effect of cohesive nonswelling soil layer on certain characteristics of underlying expansive soil media" *Ph. D. thesis*, Civil Engg., Deptt., I.I.T. Bombay.

KATE, J.M. and KATTI, R.K., (1975), "Role of micro-particles in interation between *CNS* layer and underlying expansive soil media", *Proc, of the 5th Asian Regional Conf. on SM and FE*, Bangalore, Vol. 1, pp. 15-18.

KATE, J.M. and KATTI. R.K., (1980), "Effect of *CNS* layer on the behaviour of underlying expansive soil media—An experimental Study" *Indian Geotechnical Journal*, Vol. 10, No. 4, pp. 281-305.

KATTI, R.K. (1979), "Research for solutions to problems in black cotton soils", First IGS Annual lecture, published in *Indian Geotechnical Journal*, Vol. 9, No. 1, January, pp. 1-80.

KATTI, R.K., DESAI, I.D. and KULKARNI, S.K., (1973), "Field observations on variations of engineering characteristics with depth in expansive soils", *Proc. of 3rd Inter. Conf. on Expansive Soils*, Haifa, Israel, Vol 1, pp. 115-120.

KATTI, R.K., KATE, J.M. and KULKARNI, S.K., (1970), "An approach to problem of shallow foundations in swelling soils—A laboratory investigation", *Symposium on Shallow Foundations*, Bombay, pp. 211-219.

KATTI, R.K. and KULKARNI, S.K., (1967), "Studies on nonswelling soil layer as a means to resist swelling pressure of expansive soil system", *Proc. of South East Asian Reg. Conf. on Soil Engineering*, Bangkok, pp. 147-157.

KATTI, R.K., KULKARNI, S K. and KATE, J.M., (1969, *a*) "Experimental Investigation on Cohesive nonswelling soil layer as intercepting media for footing on expansive soil". *Proc. 2nd Int. Conf. on Expansive Clay Soils*, College Station, Texas, pp. 327-333.

KATTI, R.K., LAAD, H.B., KULKARNI, S.K., and LAL, R.K., (1969 *b*), "Studies on mechanism of expansive soil media and its application to certain field problems". *Symposium on Characteristics and Construction techniques in Black Cotton Soil*, Poona, pp. 49-61.

KATTI, R.K., LAL, R.K., FOTEDAR, S.K. and KULKARNI, S.K., (1969, *c*) "Depth effects in expansive clays", *Proc. 2nd Int. Conf. on Expansive Clay Soils*, College Station, Texas, pp. 362-373.

KULKARNI, S.K., (1972), "Studies on expansive soil media in relation to clay content and other characteristics", *Ph. D. Thesis*, Civil Engg. Deptt., I.I.T. Bombay.

KULKARNI, S.K. and KATTI, R.K., (1973), "A micro-particle and micro-anchor approach to mechanics of swelling soil media", *Proc. of the 3rd Int. Conf. on Expanstve Soils*, Haifa, Israel, Vol. 1, pp. 43-52.

ZEITLEN, J.G. and KOMORNIK, (1965), "Measurement of lateral swelling pressure", *Proc. 6th Inter. Canf. on S.M. and F.E.*, Voi. 1, Div. 2, pp. 278-281.

Notations

A	Area of shear surface
A_1	Area of shear surface developed under action of surcharge load.
CNS	Cohesive nonswelling soil
c, c'	Cohesion
$\bar{c}$	Compression
c_u	Developed cohesion in *CNS* layer
c'_u	Developed cohesion in *CNS* layer under the action of surcharge load
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	Actual thickness of cohesive nonswelling soil layer encountered in the process of balancing swelling and sweiling 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
$\bar{h}'$	Vertical distance in *CNS* layer upto which shear planes are mobilized due to the effect of swelling force on an idealized individual particle-under the influence of surcharge load.
l	Dimension of an idealized individual clay particle, considered to be cubical in shape, (2 micron)
N_I	Interference number
$\bar{N}_L$	Number of idealized clay particles oriented in lateral direction in unit area
$\bar{N}_v$	Number of vertically oriented idealized clay particle in unit area
p	Percentage of clay fraction in the soil
q_{sw}	Swelling pressure of soil mass
q_{sw_e}	'Equivalent no volume change' swelling pressure
q_{swi}	Swelling pressure of an idealized individual clay particle
q_{swi_c}	Swelling pressure of an idealized individual particle, evaluated for constant volume condition at a given void rotio

q_u	Shear strength from *UCS* test = $\frac{1}{2}$ *UCS*
γ_{sub}	Submerged unit weight of soil
ϵ	Swelling of an idealized individual particle
θ, θ_1	Angle of shear planes with horizontal
μ	Micron
ϕ, ϕ'	Angle of internal friction
ϕ_s	Coefficient of skeleton friction