Search for Solutions to Problems in Black Cotton Soils*

by

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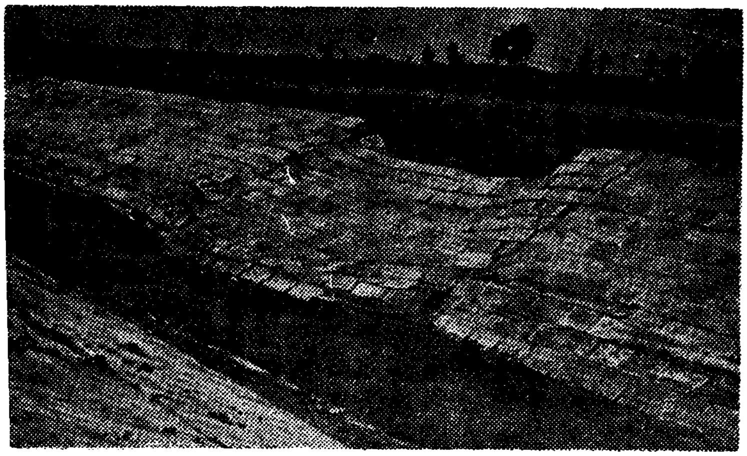
INTRODUCTION

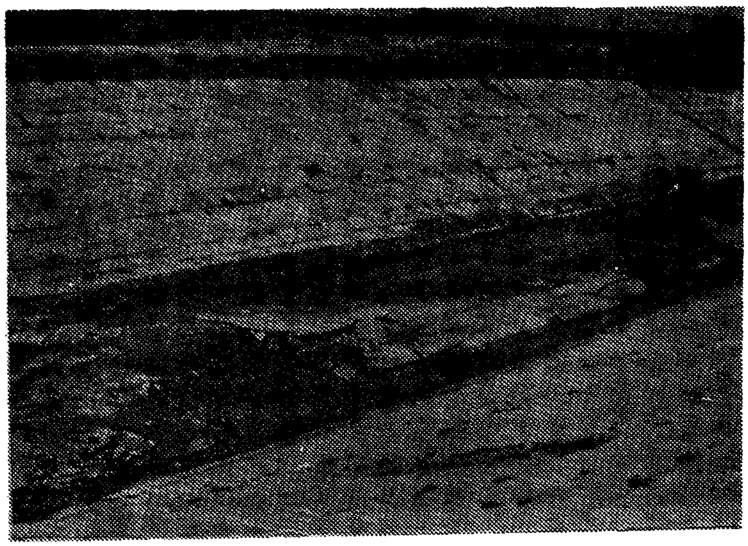
Black cotton soil deposits in India are a boon to farmers but are problematic to civil engineers. Civil engineering structures experience large scale damage due to heaving accompanied by loss of strength of these soils during rainy seasons and shrinkage during summer. Buildings crack, canal linings slide, beds of canals heave, roads get rutted and retaining structures etc. crack as shown in Figure 1, (Jennings 1955, Salas and Serratesa 1957). It is but natural that design and construction engineers are apprehensive of using black cotton soil as a construction material for the construction of earth dams, embankments, filling and preparation of bricks etc.

The problem has assumed economic importance at the national plane, as approximately one third of the surfacial deposits of the country are of black cotton soil. These deposits are predominant in the states of Gujarat, Maharashtra, Karnataka, Andhra Pradesh, Tamilnadu and Madhya Pradesh. All these states are engaged in increasing the irrigation potential of their states. Heavy investments are made for the construction of dams, canals, cross drainage structures, roads and buildings. Several thousand kilometers of canals and roads pass through black cotton soil deposits. A large number of dams and embankments are also constructed in these regions needing gigantic quantities of earth for construction. Development is accompanied by need to construct buildings for human habitation. Thus, it is clear that varied types of civil engineering activities are taking place on black cotton soil deposits. These activities are increasing during the past few plan periods. There is definite indication that these activities will be increasing.

During the last two to three decades several research workers and organizations in the country are making sustained effort to understand the behaviour of black cotton soil deposits and suggest solutions to solve problems arising out of these deposits to civil engineering structures (Grim 1953, Ranganatham and Satyanarayana 1969, Tretoar 1950).



(a)



(b)

FIGURE 1 Damages to various structures in expansive soil (black cotton soil) area of Malaprabha river valley project constructed without using CNS layer (a) Canal lining at Km 22, (b) Bed heaving

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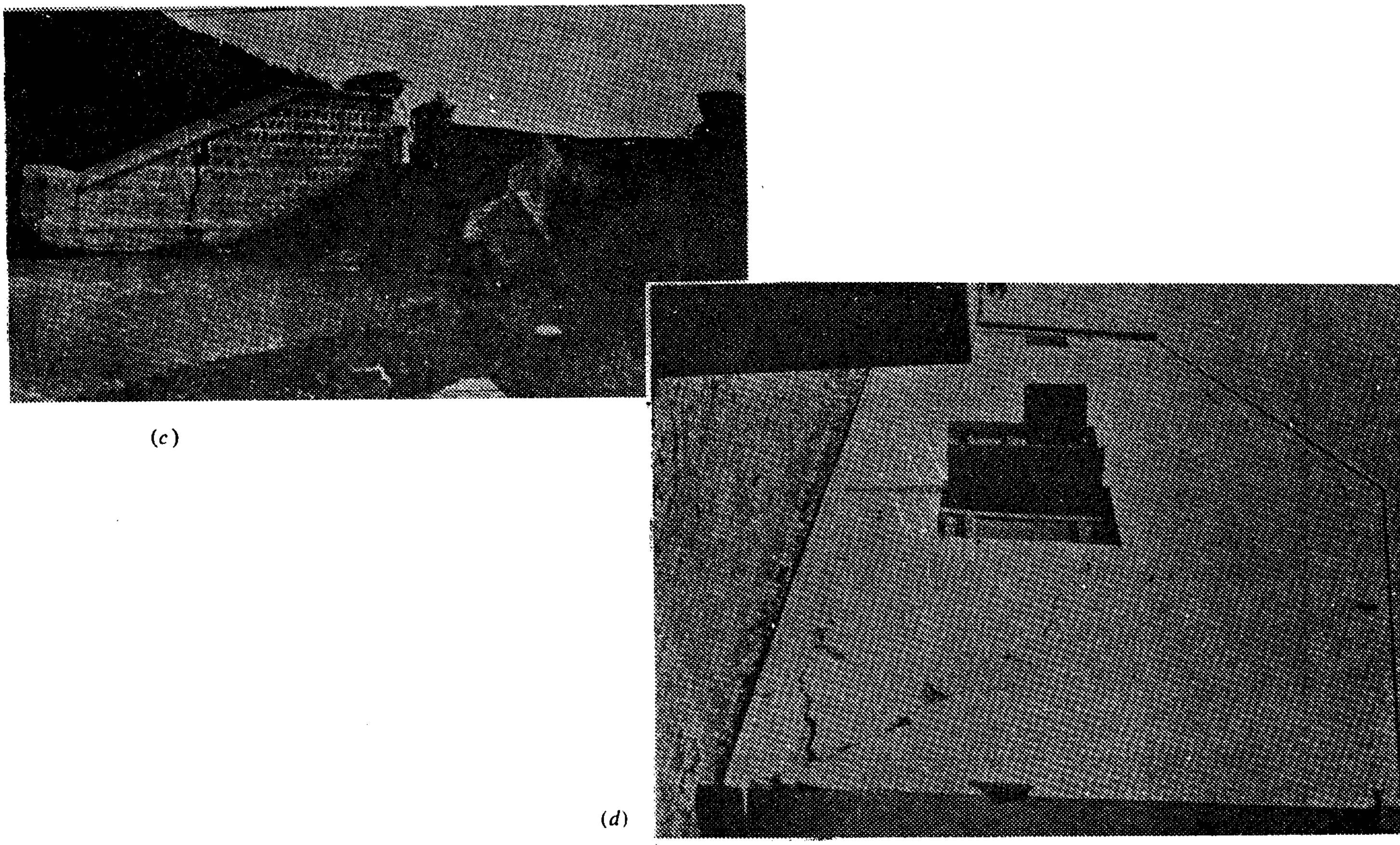


(c) (d)

FIGURE 1 Damages to various structures in expansive soil (black cotton soil) area of Malaprabha river valley project constructed without using CNS layer (c) Inlet at Kiresur pond (d) building at Tirlapur

The author came across these deposits in the year 1952-1953 in Mahuva, Navsari, wherein, he was engaged in constructing a subdivisional irrigation colony on such deposits. It was a disappointing feeling to hear from other senior engineers that the buildings would crack and fall apart in a few years due to swelling and shrinkage of the underlying black cotton soil deposit. A large numer of buildings in the area were heavily damaged.

Since 1958 an attempt is also made by the author to tackle the problems of black cotton soils from various angles. The studies were directed

TABLE 1

GRAIN SIZE ANALYSIS AND INDEX PROPERTIES

Description	Sholapur (1)	Poona (2)	Sidde-shwar (3)	Nasik (4)	Nagpur (5)	Yield Hari (6)	Amr-avati (7)	Vijaya-wada (8)	Malap-rabha (9)	Alam-matti (10)
Gravel, %	21.0	0.0	3.0	2.0	8.5	3.5	0.0	1.5	14.0	0.0
Sand, %	18.0	12.0	21.0	16.6	12.5	10.0	13.5	20.5	13.4	14.0
Silt, %	28.2	32.0	34.5	32.5	28.2	32.5	32.5	17.2	20.0	28.0
5 μ Clay %	-	68.0	41.5	48.5	50.8	54.0	54.0	60.8	63.0	58.0
2μ Clay %	32.8	56.0	-	-	-	-	-	-	52.6	-
1μ Clay %	-	47.0	-	-	-	-	-	-	-	-
Liquid limit, %	69.2	81.5	70.3	72.8	59.2	68.0	81.0	91.8	74.0	59.8
Plastic limit, %	41.9	43.2	41.0	48.2	43.3	46.2	47.0	38.3	34.0	30.4
Plasticity index	27.3	38.3	28.4	24.6	15.9	21.8	34.0	53.5	40.0	29.4
Shrinkage limit, %	12.4	9.1	13.5	7.4	10.3	14.1	10.0	9.8	10.8	7.0
Shrinkage ratio	2.1	-	2.0	2.0	2.1	1.9	2.1	2.2	-	2.3

(i) to understand the nature and properties of various fractions of black cotton soils from the country,

(ii) to evaluate the engineering behaviour of the black cotton soil deposits with depth under various seasonal conditions and

(iii) to develop mechanical and/or chemical methods to tackle the problems created by black cotton soils for engineering construction.

In this lecture an attempt is made to briefly focus the outcome of the studies wherein, a large number of postgraduate students participated in different aspects of the studies.

Prior to planning the studies to evaluate the swelling pressure, heave or consolidation characteristics, lateral pressures etc. the general property studies and insitu field studies were taken up.

ENVIRONMENT

Black cotton soil deposits are found under conditions, wherein, the slope of the terrain is less than 3° and the rainfall is in the range of 300 to 900 mm. The rain normally falls during the monsoon. Summers are hot and the parent material is normally derived from weathering of basalt. However, there are black cotton soil deposits derived from other types of rocks and also from very old sedimentary deposits.

GRAIN SIZE AND INDEX PROPERTIES

In Table 1, grain size and index properties of black cotton soils from various parts of India are given. These soils contain 2 micron clay fraction varying between 50 to 70 per cent, sand 15 to 25 per cent, silt 15 to 30 per cent and gravel less than 10 per cent. Liquid limits, plastic limits, plasticity index and shrinkage limits normally range between 60 to 90 per cent, 30 to 50 per cent, 30 to 40 and 8 to 18 per cent respectively. Low values of shrinkage limit indicate large volume change.

MINERALOGICAL PROPERTIES

Minerals present in the sand fraction, silt fraction and clay fraction were studied. The mineralogical content of sand is quartz and chlorite type. This type of mineral is met with when they are derived from basalts. The minerals present in clay fraction are montmorillonite and a combination of montmorillonite and

TABLE 2

X-RAY DATA ON POONA SOIL SAMPLES (SIZE FARCTIONS)

Sand		Silt		Clay		Soil	
d	I	d	I	d	I	d	I
10.36	10	13.50	7	15.50	10	15.30	6
8.12	9	10.77	9	4.59	1	12.10	10
6.91	2	4.30	1	3.60	2	3.23	4
5.37	5	3.53	1	2.94	2	2.52	3
4.02	1	2.94	3	2.55	1	1.68	5
3.00	2	1.97	1	1.30	2	1.31	3
1.53	1	1.80	4	-	-	-	-

TABLE 3

D.T.A. DATA ON POONA SOIL SAMPLE (SIZE FRACTIONS)

Sand		Silt		Clay		Soil	
Endothermic	Exothermic	Endothermic	Exothermic	Endothermic	Exothermic	Endothermic	Exothermic
C°	C°	C°	C°	C°	C°	C°	C°
113	-	122	-	137	-	122	-
579	-	606	-	508	-	589	-
866	-	761	-	739	-	781	-
891	-	803	-	832	885	812	-

illite, as can be seen from the X-ray, DTA and base exchange capacity data presented in Figures 2 and 3, and Tables 2 and 3 (Baver 1956, Grim 1953, Levine 1946).

CHEMICAL PROPERTIES

The chemical properties shown in Table 4 are for majority of black cotton soil samples in India. The SiO_2 content ranges between 45 per cent and 58 per cent, Al_2O_3 from 13 to 18 per cent, CaO from 1 to 8 per cent, MgO from 2 to 5 per cent. The pH value is around 8 to 8.5. Carbonate contents are high in many cases. Base exchange capacity is in the range of 100 to 130 meq/100g which shows the presence of montmorillonite type of clay minerals (Grim 1953).

IN SITU CHARACTERISTICS

Field observations show that the black cotton soil deposits heave and become slushy during rainy season (Katti *et al* 1973, Katti *et al* 1969, Katti *et al* 1969e). During summer, however, map type cracking is observed. The clods of the black cotton soil taken from the surface are found to be very hard. In view of this, density, moisture content, and shear strength with depth were measured in various places in the black cotton soil deposits. These measurements were made both during summer and rainy seasons (Figure 4).

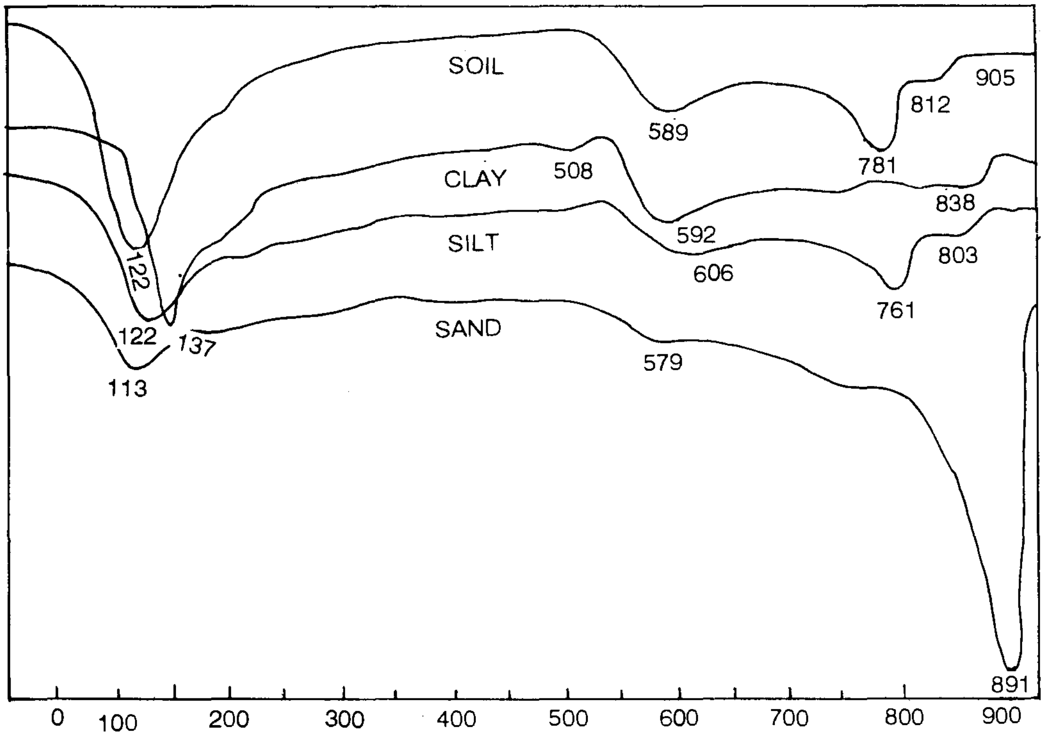


FIGURE 2 Differential thermal curves for Poona black cotton soil fractions

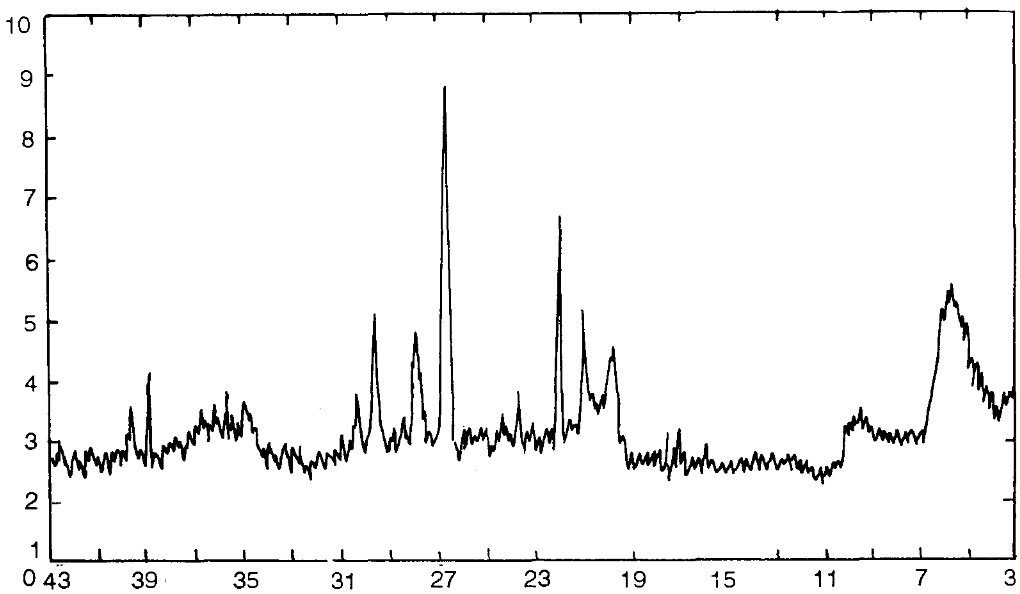


FIGURE 3 X-ray diffraction diagram of Poona black cotton soil

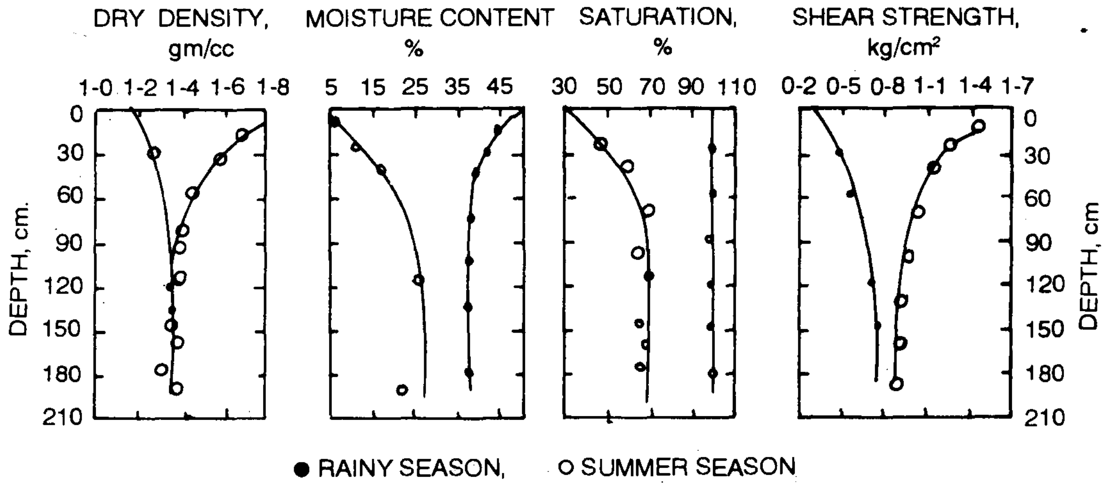


FIGURE 4 (a) Results of field tests during summer and rainy seasons at Poona

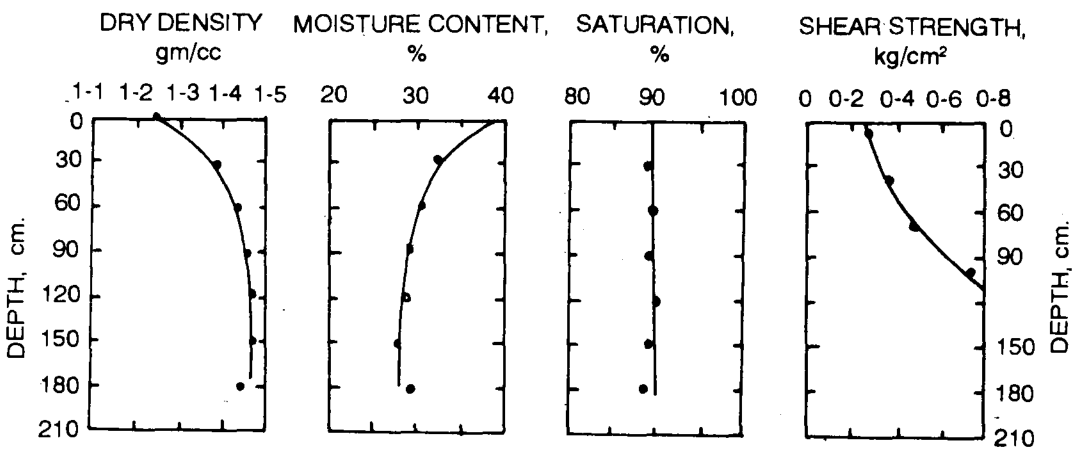


FIGURE 4 (b) Results of field tests at Surat

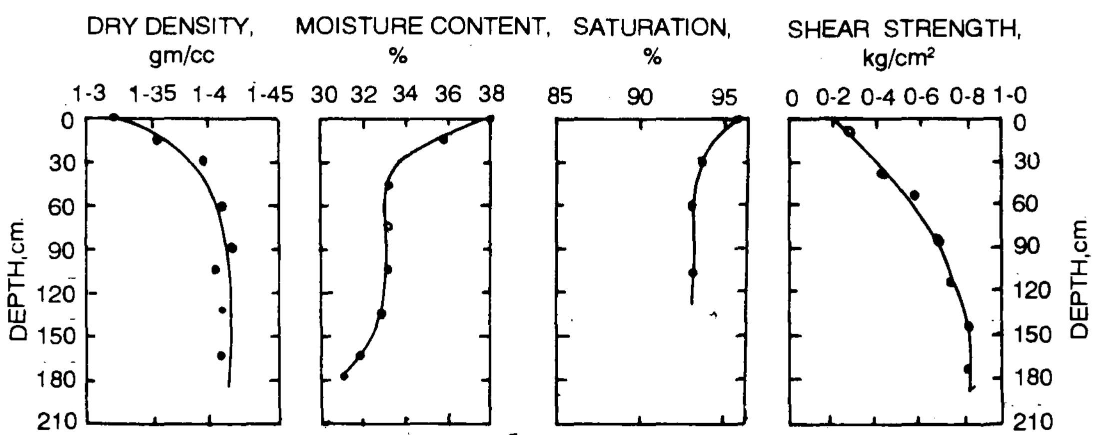


FIGURE 4(c) Results of field tests at Broach

TABLE 4
CHEMICAL PROPERTIES OF SOME BLACK COTTON SOILS

Description	Place								
	Sholapur	Poona	Siddes-hwar	Nasik	Nagpur	Yeldhari	Amr-avati	Vijaya-wada	Malap-rabha
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
p^H, at 27° C	8.75	8.50	8.90	8.50	8.20	8.70	7.40	8.80	8.00
Organic matter content %	0.55	0.96	0.70	0.70	0.40	0.80	0.60	0.40	0.45
Carbonate content, %	2.42	4.35	4.40	3.30	0.50	1.90	0.20	0.40	1.30
SiO_2, %	49.30	52.10	45.60	47.10	58.10	47.70	53.30	57.00	56.20
Al_2O_3, %	13.70	18.10	14.50	16.70	15.60	15.50	15.70	17.50	13.30
CaO, %	6.90	8.50	7.40	6.20	2.70	4.40	2.80	1.60	8.40
MgO, %	4.80	3.90	4.10	3.20	2.50	3.70	2.70	2.60	2.20
Fe_2O_2, %	14.80	1.77	12.60	12.60	10.30	15.10	14.00	10.30	7.30
TiO_2, %	1.90	0.30	2.00	1.50	1.30	2.40	2.00	1.10	-
Sulphate, %	1.60		1.10	1.90	1.80	1.40	1.20	1.30	0.40
Loss on ignition, %	16.50	9.60	13.90	13.00	8.60	10.70	9.20	8.20	12.40
Base exchange capacity meg/100 g. for									
(a) fraction passing 44 sieve	57.60	71.00	57.90	65.30	51.10	58.50	72.40	47.40	60.00
(b) fraction finer than 2 μ	109.20	123.00	84.40	124.60	99.40	160.60	130.40	97.80	107.00

During rainy season the density is very low at the surface around 1 g/cc but increases rapidly with depth and remains almost constant at 1.37 g/cc below a depth of around 1 m. Moisture content is very high at the surface; reduces rapidly with depth and remains constant below a depth around 1 m. The cohesion value, c_u as obtained from vane shear or unconfined compressive strength tests increases rapidly up to 1 m or so and remains constant thereafter. The c_u value ranges between 0.35 to 0.75 kg/cm².

However, during summer the clod density at the surface is very high around 1.6 g/cc and it decreases with depth and remains almost constant below 1 to 1.2 m at 1.37 g/cc. Moisture content is low at the surface which gradually increases with depth and remains almost constant below 1 to 1.5 m. The moisture content values which are remaining constant are lower than those in case of rainy season values. This shows that although there is change in moisture content there is hardly any volume change below 1 to 1.5m depth.

Cohesion value is very high at the surface and decreases with depth upto 1 to 1.5 m and remains constant, thereafter, range of c_u being 1.55 to 0.80 kg/cm².

Cuts in canals are found to stand almost vertical during summer but show heavy sliding during rainy season. Unlined canals stabilize to a slope of almost 1 in 8 or 1 in 10, when they are put to use. The vane shear strength of soil, at a slip in Malaprabha right bank canal at km 22 is found to be very low, around 0.15 kg/cm², above the shear plane. Taking into consideration the characteristics of the black cotton soils both in the field and also their general properties, following studies were conducted in the laboratory.

LABORATORY STUDIES

The laboratory studies consisted of

(i) evaluation of the swelling pressure as well as heave and to ascertain the reasons for the development of swelling pressure and heave.

(ii) simulation of the characteristics such as density, moisture content, swelling pressure and vane shear strength etc. observed with depth in field by conducting large scale tests.

(iii) evaluation of the shear strength development in expansive soils, under various conditions.

Some effects obtained in studies conducted on black cotton soil samples in laboratory under controlled conditions are briefly described here.

SWELLNG PRESSURE AND HEAVE STUDIES

Palit (1953), Komornik and Zeitlin (1965), Ranganatham and Satyanarayana (1969), and other research workers (Ladd 1960), developed methods to measure the swelling pressure. These methods consisted of evaluating swelling pressure using samples of sizes say 7.5 cm diameter and of different thicknesses. The swelling pressure was reckoned as the maximum pressure exerted during the process of saturation. Specially in case of consolidation setups the samples were allowed to swell and brought back to intial state by external force. It was also noted by these workers that the moisture content and method of testing affected the development and measurement of swelling pressure. The swelling pressure of Poona black cotton soil was evaluated by using various methods and also by the open system and close system (Figure 5). From these studies it was realized that for comparison purposes sit was better to evaluate the swelling pressure under constant volume condition of the samples at zero moisture content.

From the mechanical analysis, it is seen that black cotton soil contains sand, silt and clay fractions. The chemical analysis, X ray and differential thermal analysis (D.T.A.) indicate the presence of clay mineral of the type montmorillonite which has an expanding lattice structure. In view of this, a black cotton soil sample was separated into sand, silt and clay fractions. Each fraction was dried to 110° C and was

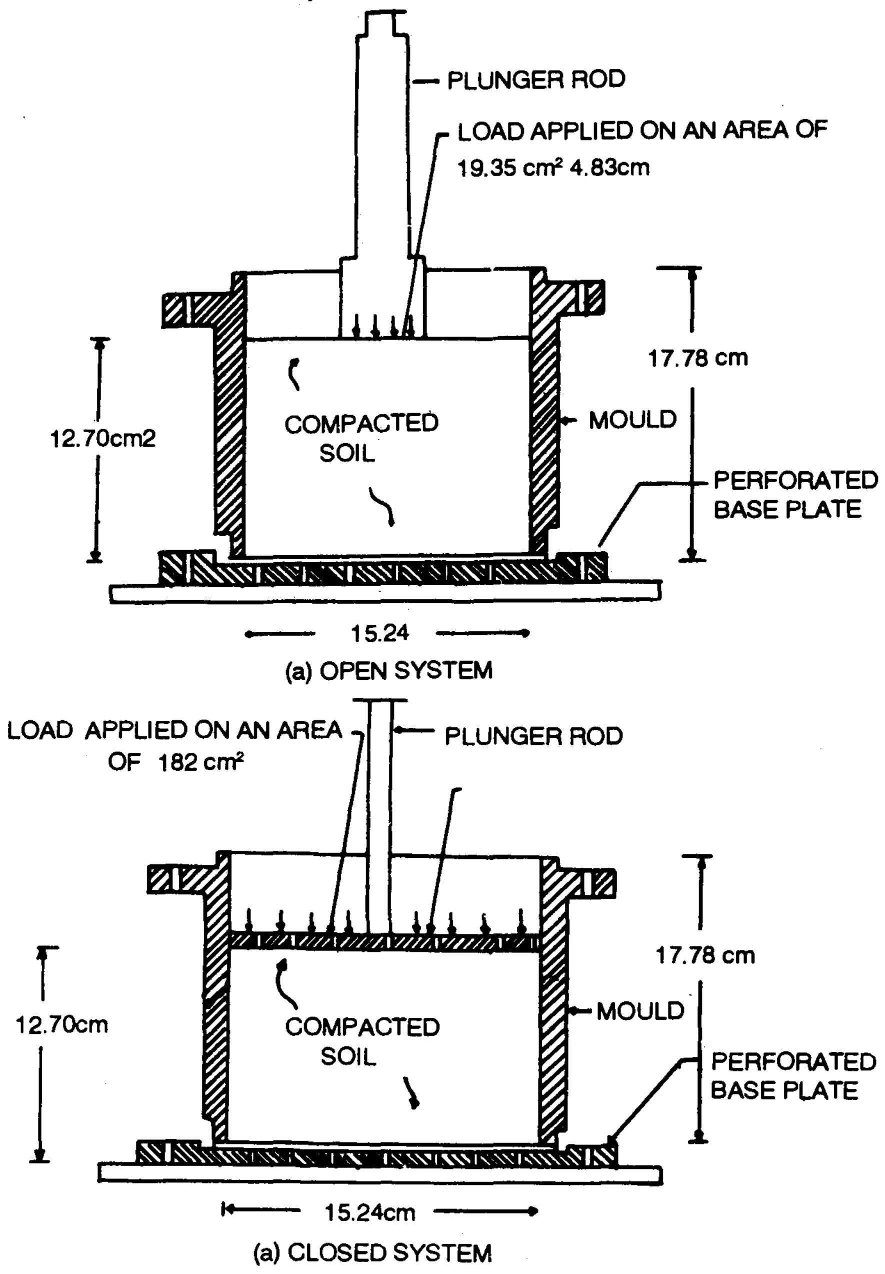


FIGURE 5 Measurement of swelling pressure

compacted to a known density and was subjected to swelling pressure test under no volume change condition. The results showed that sand and silt fractions did not exhibit any swelling pressure and showed slight settlement, whereas, the clay fractions exhibited swelling and swelling pressure of the order much higher than that of the black cotton soil sample from which it was extracted (Katti 1972, Katti et al 1969b.)

For example, the swelling pressure under no volume change condition of Poona black cotton soil sample compacted to a void ratio of unity and zero moisture content was 3.3. kg/cm² whereas, the corresponding value for 2μ clay fraction was 7.9 kg/cm². These studies clearly indicated that clay fraction is the one which is responsible for swelling pressure action.

The swelling pressure test conducted by taking void ratio constant and altering the 2μ clay content showed that as the clay content decreased, the swelling pressure went on reducing and reached a zero value, when the clay content was less than or equal to 25 to 30 per cent. This can be seen from Figure 6. For a given amount of clay content the increase in void ratio is accompanied by decrease in swelling pressure. For example, swelling pressure values recorded from Poona soil samples are 4.18, 3.30 and 2.73 kg/cm² for void ratios 0.8, 1.0 and 1.2 respectively. The relation between void ratio and swelling pressure is not a straight line in nature but is curvilinear one as can be seen from the Figure 7.

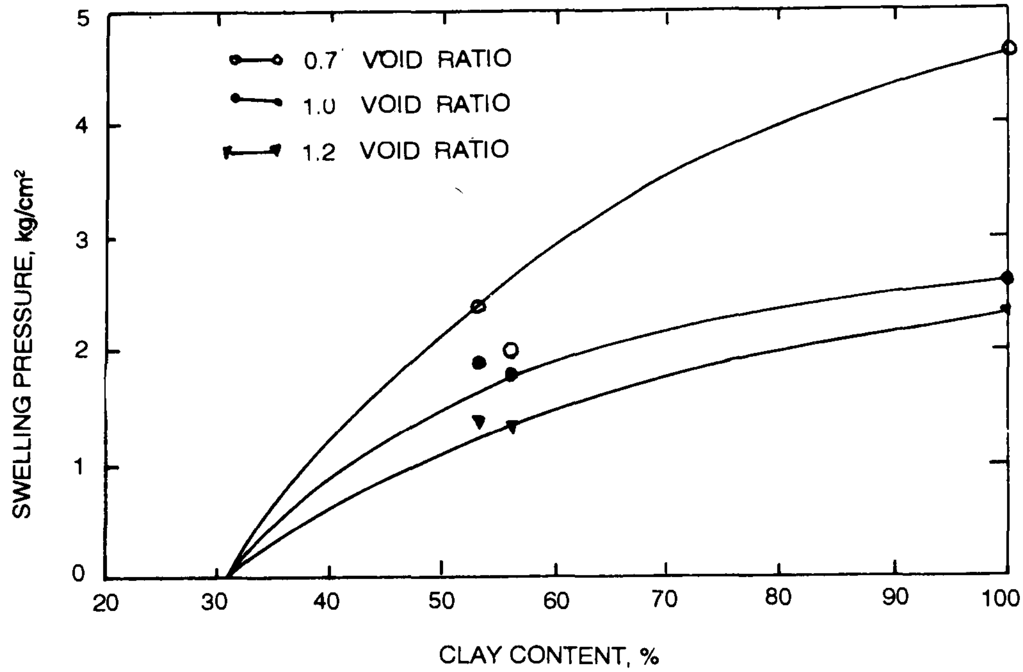


FIGURE 6 Relation between clay content and swelling pressure

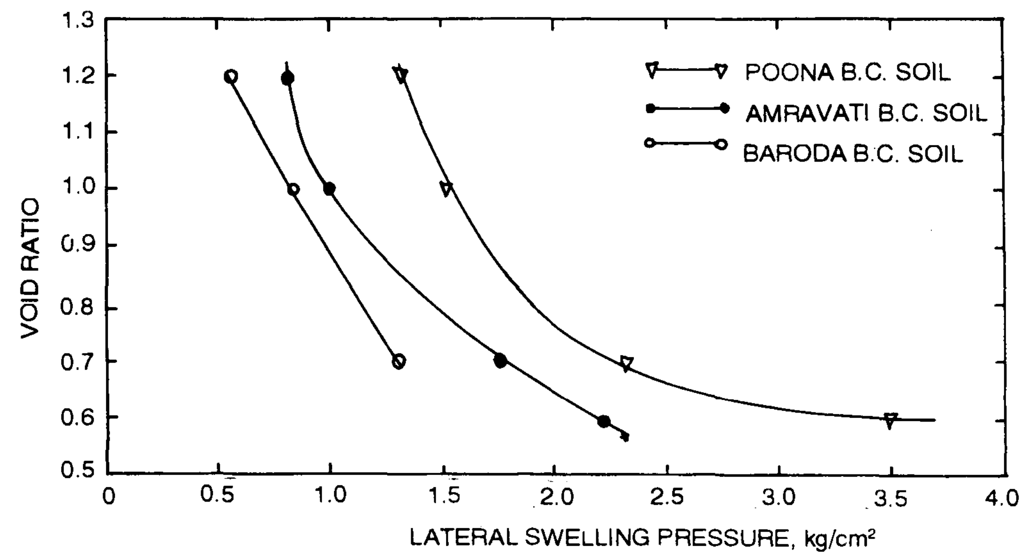


FIGURE 7 Relation between lateral swelling pressure and void ratio

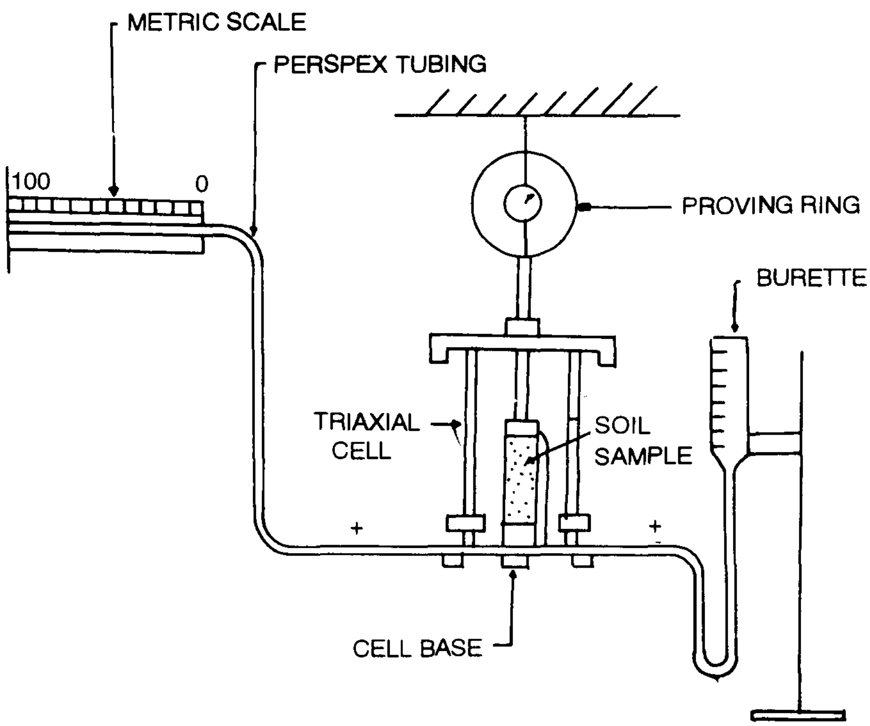


FIGURE 8 (a) Set up for measuring lateral and vertical swelling

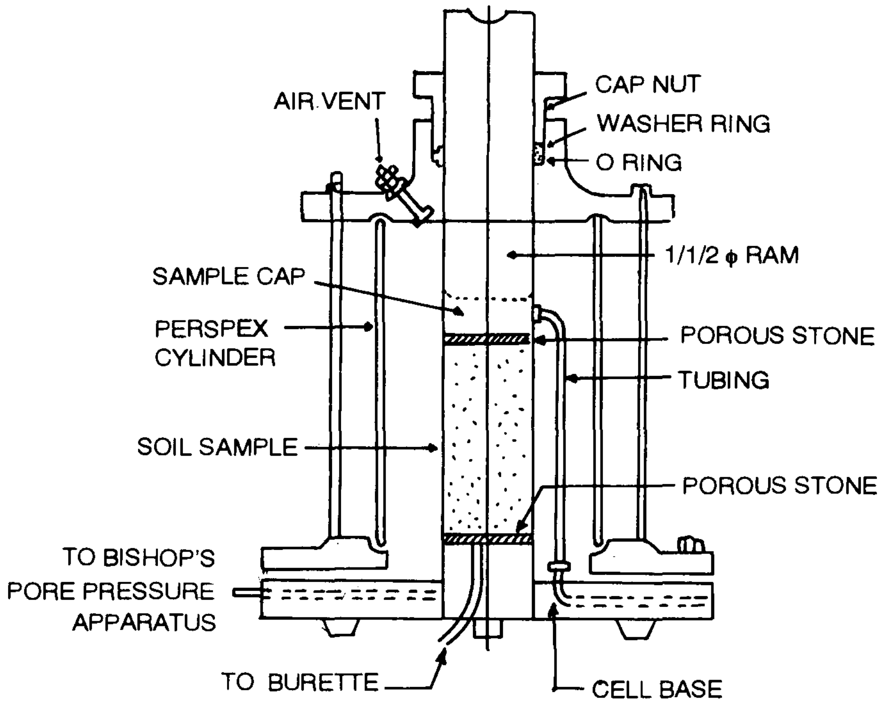


FIGURE 8(b) Modified triaxial cell for measuring lateral and vertical swelling pressure

The swelling pressure tests were conducted to ascertain the nature of the lateral pressure developed during saturation (Katti *et al* 1969*a*, Katti *et al* 1969*b*). This was done by using a triaxial equipment with appropriate accessories to measure both the lateral and vertical pressures of the sample during saturation (Figure 8). These studies indicated that the ratio of lateral pressure to vertical pressure is almost 1. Another observation of interest is that as the sample was saturated the swelling pressure increased and it slightly dropped down beyond a degree of saturation of 95 per cent and remained almost constant. This effect is shown in Figure 9 (Katti *et al* 1969*a*).

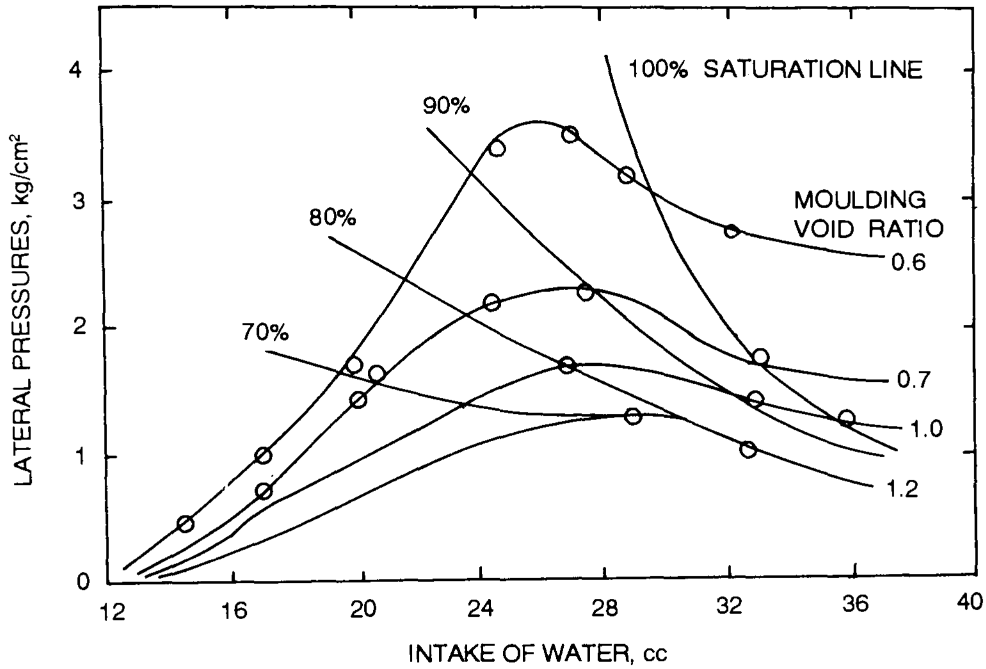


FIGURE 9 Lateral swelling pressure versus intake of water for Poona soil sample

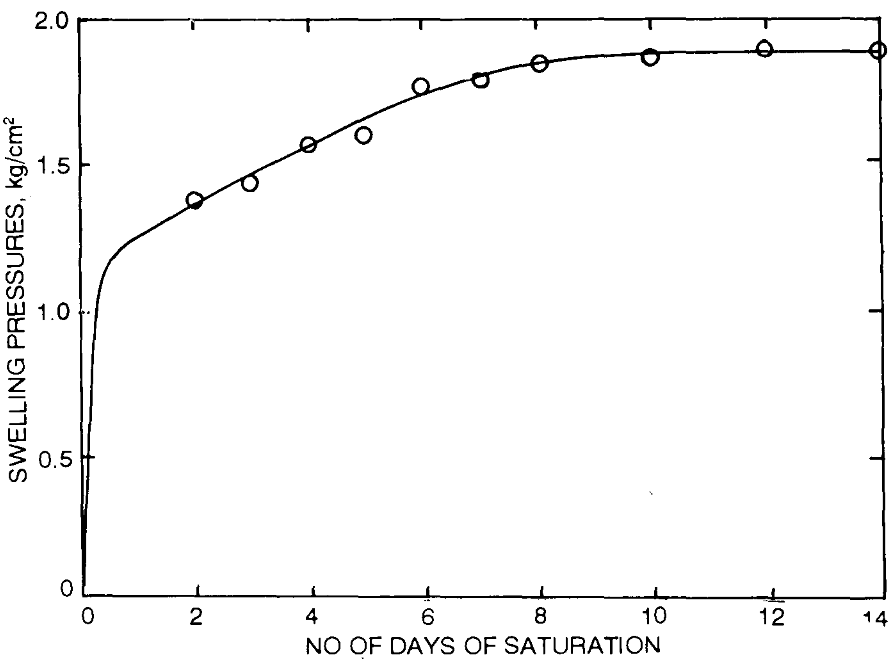


FIGURE 10 Time versus swelling pressure for Malaprabha soil

Time *versus* swelling pressure curve as given in Figure 10, shows that the development of swelling pressure is at a faster rate initially and in about 15 to 20 days the curve tends to be asymptotic in nature (Katti and Kulkarni 1967).

From these studies it was realised that the 2μ clay fraction plays an important role in the swelling and swelling pressure aspects. The montmorillonite clay mineral is supposed to be having predominant swelling in *c* direction than in *a* or *b* directions. Therefore, the mineral is anisotropic electrically and also in relation to expansion characteristics.

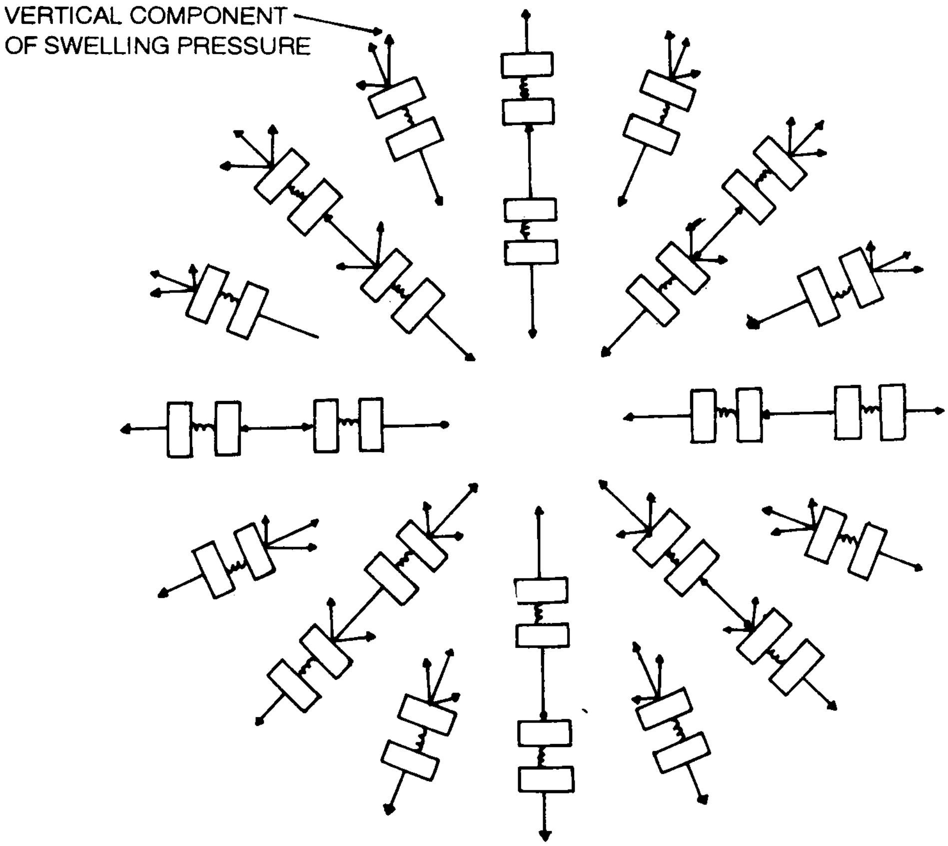


FIGURE 11(a) Micro-particle model-components of swelling pressure in a particular direction for the oriented clay particles

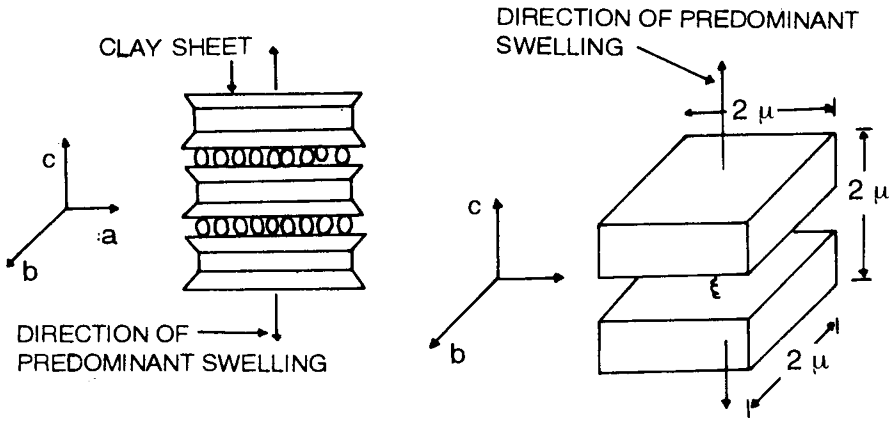


FIGURE 11(b) Micro-particle model-models of clay particle with its swelling

On the basis of mechanical analysis and specific gravity, probable number of 2μ clay particles for a given condition were calculated in a unit volume assuming the swelling particles to be of 2μ X 2μ X 2μ size with a split in the centre (Figure 11a) (Kulkarni and Katti 1973).

The expansion characteristics of these idealised particles are assumed to be similar to those of the montmorillonite units and the particles are assumed to be formed due to stacking, one above the other, of

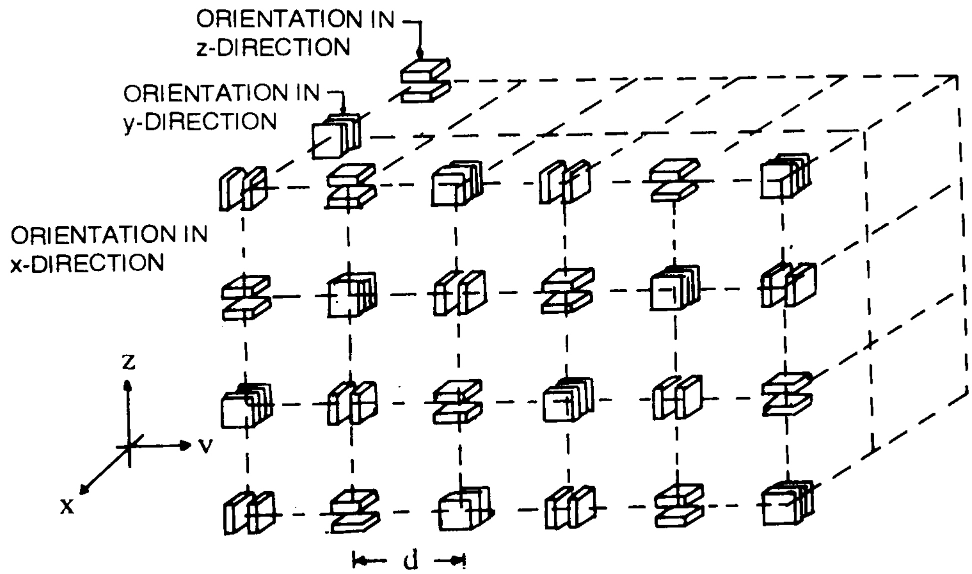


FIGURE 11(c) Micro-particle model-distrubution of oriented clay particles in an idealized system

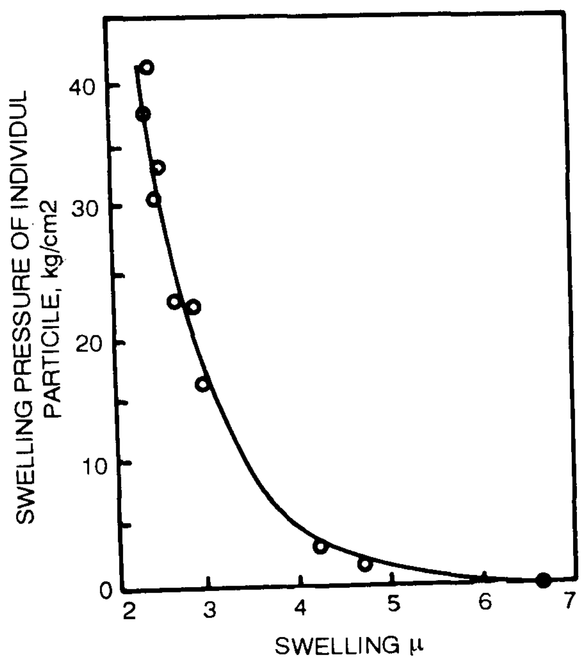


FIGURE 11(d) Micro-particle model-relationship between swelling and pressure of an individual clay particle

the montmorillonite units. It can be seen than the individual particles are anisotropic with respect to swelling. But in the soil mass as a whole, the apparent effect is as though the swelling pressure is of isotropic nature. This may be attributed to the statistical random distribution of the clay particles, as shown in Figure 11 for a given sample.

From Figure 11 it can be seen that the swelling pressure measured in any particular direction is the sum of the components of the force exerted by the individual particles, per unit area.

Resolving the forces along X, Y and Z directions the swelling pressure effect would be equivalent to as though one third of the particles are oriented in each of these directions. This idealised arrangement can be visualised from Figure 11 where the particles are positioned in a rectangular array.

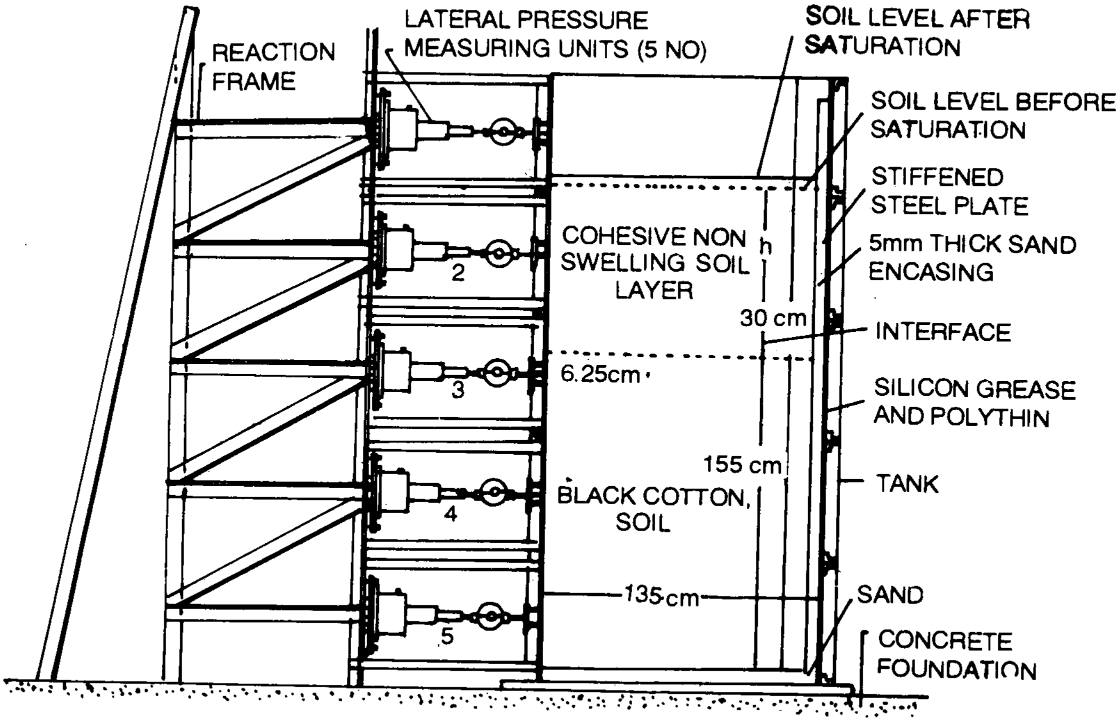


FIGURE 12 Tank and reaction frame of the large scale equipment

Thus, in a unit volume, number of clay particles, would be

$$N = \frac{p}{(2\mu)^3}\frac{1}{1+e_1} \qquad \text{....(1)}$$

where

p= clay content

e_1=initial void ratio

Number of clay particles

$$\text{per unit area} = N_r \qquad \text{...(2)}$$

$$\left[\frac{p}{(2\mu)^3}\frac{1}{1+e_1}\right]^{\frac{2}{3}}$$

Number of equivalent vertically oriented clay particles

$$\text{per unit area} = \frac{1}{3}\left[\frac{p}{(2\mu)^3}\frac{1}{1+e_1}\right]^{\frac{2}{3}} \qquad \text{....(3)}$$

Swelling pressure of an equivalent individual particle Q_{swi} oriented in vertical direction is obtained as follows :

$$\frac{Q_{swi} \times 1 \times 1}{(N_r / 3)} = Q_{swi} \times 2\mu \times 2\mu \qquad \text{....(4)}$$

$$Q_{swi} = \frac{Q_{sw}\,(1\text{x}1)}{\frac{1}{3}\left[\frac{p}{(2\mu)^3\,(1+e_1)}\right]^{\frac{2}{3}}(2\mu \times 2\mu)} \qquad \text{...(5)}$$

For Poona soil having clay content 54 per cent and swelling pressure 3.3 kg/cm² at void ratio 1, the swelling pressure of individual particles works to be 23.7 kg/cm².

In the light of this study it is clear that swelling pressure in vertical and lateral directions observed in the laboratory experiments are same because of random orientation of particles although the individual particles may be anisotropic in nature.

Using test results from swelling pressure on soil having same clay content for various void ratio conditions, an attempt is made to evaluate relation between swelling and swelling pressure for a 2μ clay particle. The curve is plotted in Figure 11d using natural scale.

For developing this curve correction for internal expansion was taken care of. This simplified physical model is designated as *Micro particle Model* (Katti *et al* 1966). The curve is based on logical approach because it is difficult to obtain accurate magnitude of expansion in voids.

From these studies it can be said that when the black cotton soil is being saturated the swelling pressure developed is due to the expanding nature of the 2*μ* clay particles. The swelling pressure of the expanding clay particles in turn is due to the material characteristics of clay system.

In addition the swelling pressure of a black cotton soil system measured is affected by the state of swelling of the clay minerals, their orientation and configuration of other nonswelling clay particles.

LARGE SCALE STUDIES

The results of the variation of engineering characteristics such as density, vane shear strength and moisture content showed that these characteristics are undergoing a marked increase within a shallow depth of around 1 to 1.5m (Katti *et al* 1977*a*). To see whether such an effect is generated due to the internal characteristics of soil mass itself, it was felt advisible to conduct large scale tests, with size of the sample more than two times the depth wherein, the changes were observed in field. The size of the tank selected for study was 1.35 x 0.9 x 3 m as shown in Figure 12.

The density, moisture content and vane shear strength measurements were confined to the central 30X30 cm portion. In some of these experiments, devices were mounted to measure the developed lateral pressure with depth (Katti *et al* 1977a, Katti *et al* 1969*b*, Katti *et al* 1969*c*).

In some large scale experiments attempts were made to measure the vertical swelling pressure and vertical heave with depth. For conducting these studies the expansive soil sample was air dried under summer condition, wherein, the air dry moisture content was 8 to 10 per cent and compacted to field dry density corresponding to that observed in the constant volume zone in field which existed below 1 to 1.2 m. These values are 1.375 g/cc and 1.30 g/cc for Poona and Malaprabha soils respectively.

The result of these studies after complete saturation period which was more than 60 to 90 days are shown in Figures 13 and 14. During saturation the soil showed a cumulative heave of 11 cm at the surface in the case of Poona soil and 13 cm in case of Malaprabha soil. The density *versus* depth, vane shear strength *versus* depth and moisture content *versus* depth showed trends similar to those observed in the field. The initial density of 1.37 g/cc and 1.30 g/cc in case of Poona soil and Malaprabha soil reduced to 0.88 g/cc and 0.79 g/cc at the surface respectively and rapidly increased with depth. However, the values showed no change in density below 1.2 m depth. The vane shear strength as can be seen from figure increased from 0.05 kg/cm^2 at top to 0.78 kg/cm^2 at 1 to 1.2 m for Poona soil.

Figure 13 indicates that lateral pressure is around zero near the surface and increases rapidly up to 1.2 m depth and remains constant thereafter. The lateral pressure exerted by the soil at this depth is 3.35 kg/cm^2.

These observations from large scale studies indicate that the alterations observed in top 1 tc 1.5 m may be due to the internal characteristics of the black cotton soil mass. Such an effect is not observed in sands or silts. Thus, it appears that the clay fraction and its structure seem to be responsible for such an effect during and after saturation.

Another important fact is that the high lateral swelling pressures of the order of 0.66 kg/cm^2 at 22 cm and 3.35 kg/cm^2 at 100 cm depths are registered (Katti *et al* 1969*b*). These lateral pressures are far greater than the lateral pressures that can be expected to be developed due to the vertical stresses acting due to the weight of the soil-water system.

At a depth of 1 m the vertical stresses due to the saturated black cotton soil water system is around 0.22 kg/cm^2 and the maximum lateral pressure expected if k_0=1, is 0.22 kg/cm^2. The lateral pressure that is generated, however, is much higher than this value. In this case it is 18 times the vertical stress.

Such an effect clearly showed that a certain alteration brought out in the physical and physico-

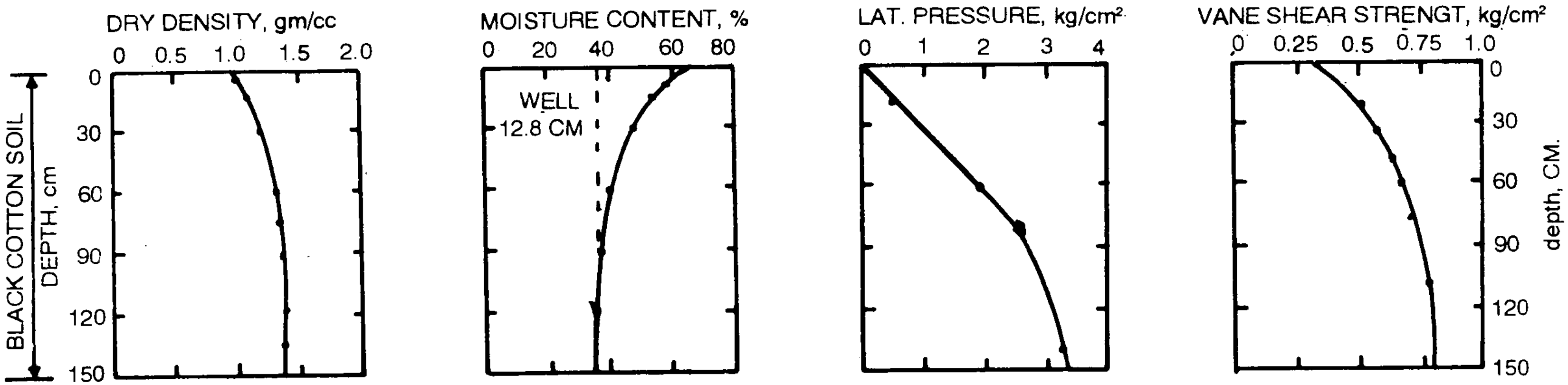


FIGURE 13 Results of large scale test of Poona black cotton soil

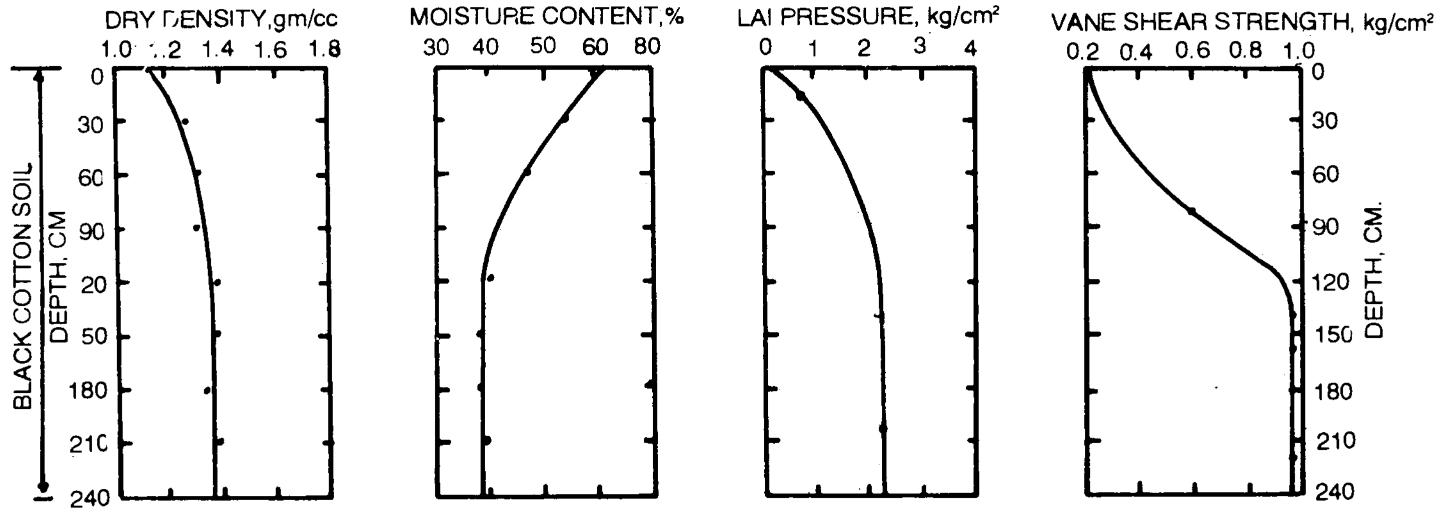
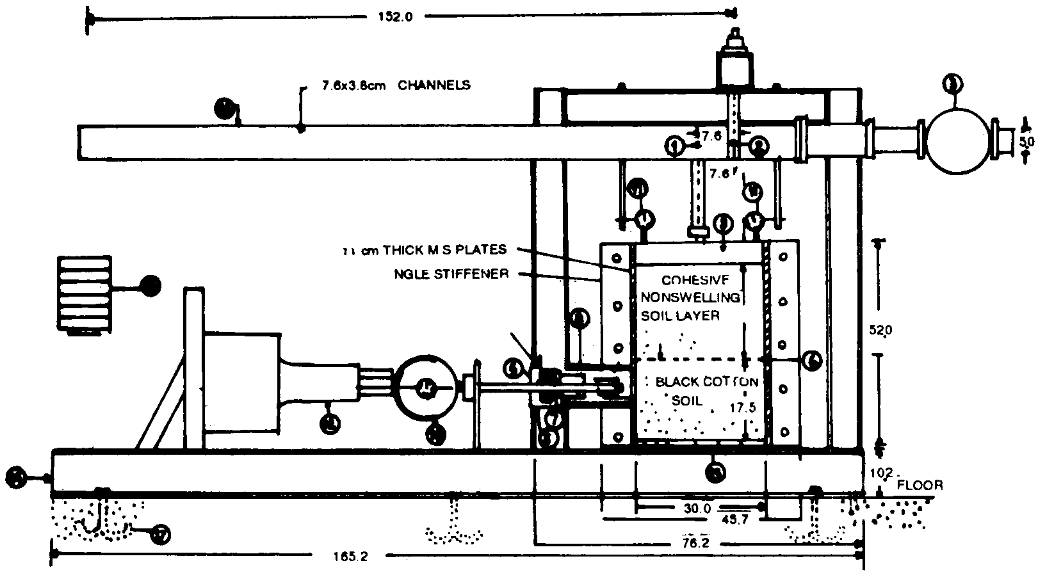


FIGURE 14 Results of large scale test of Malaprabha black cotton coil



1 LOADING POINT	7 INNER COLLAR	WEIGHTS
2 FULCRUM	8 OUTER COLLAR	14 CHANNEL BASE (SIZE 10.2 x 2.5 cm)
3 COUNTER WEIGHT	9 COVER PLATE 30 x 26 x 25 CM	15 BOTTOM SAND LAYER 0.5 x THICK
4 CYLINDRICAL SLEEVE	10 PROVING RING	16 LEVER ARM
5 PISTON 80 mm	11 DIAL GAUGES	17 ANCHOR BLADE
6 CONTAINER TANK	12 ADJUSTABLE SCREW JACK	NOTE: ALL THE DIMENSIONS ARE IN cms

FIGURE 15 Equipment for the measurement of lateral pressure under dead load surcharges

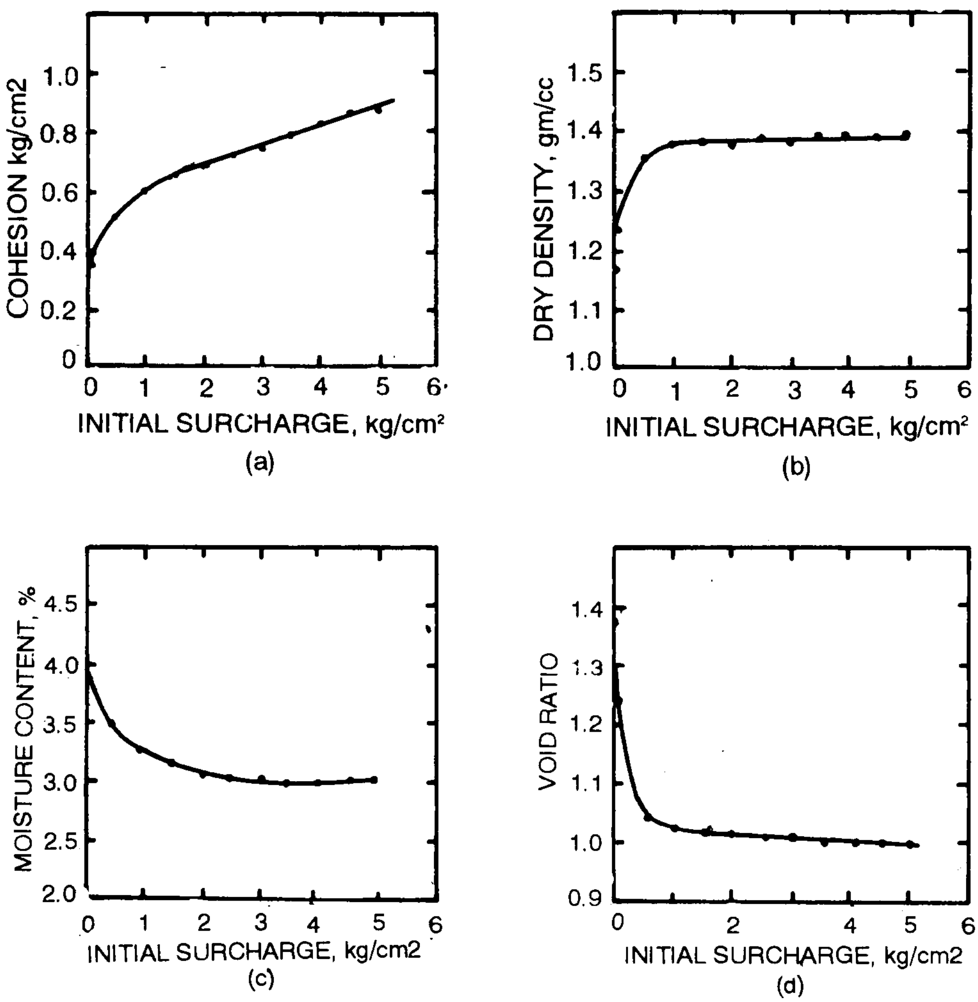


FIGURE 16 Variation of properties of soil at piston level with initial surcharge

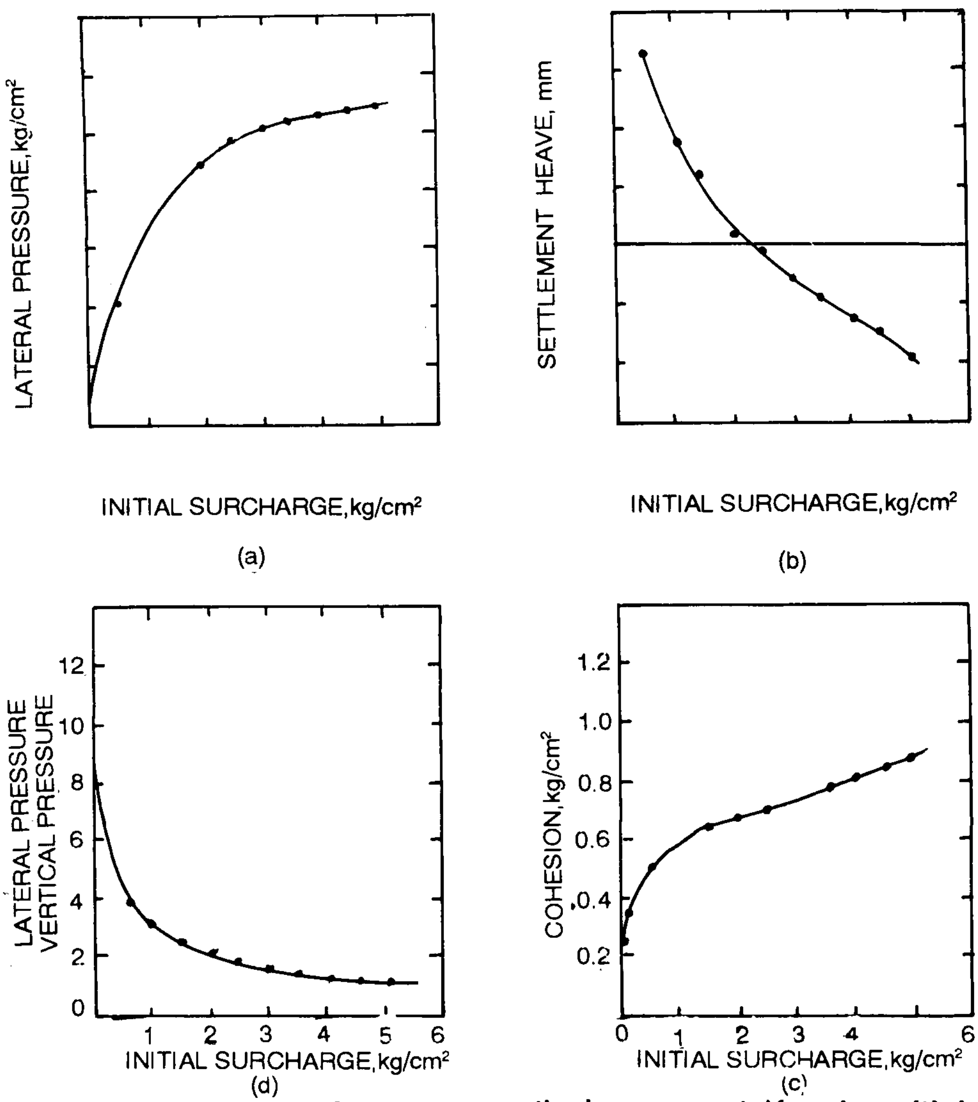
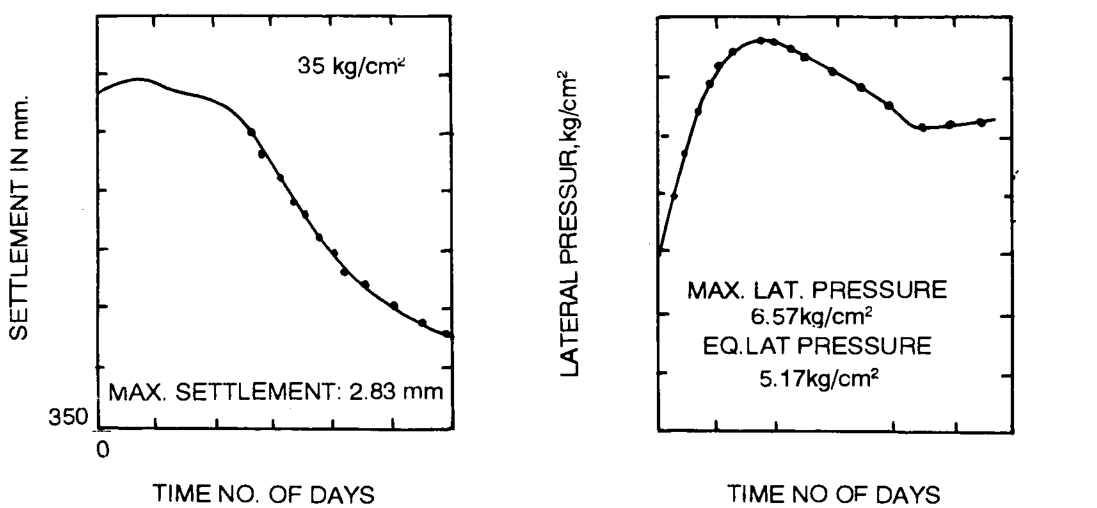


FIGURE 17 Variation of lateral pressure, vertical movement, K and c, with initial surcharge



a) AT 35 kg/cm² SETTLEMENT Vs TIME b) 3.5 kg/cm² LATERAL PRESSURE Vs. TIME

FIGURE 18 Variations in dead load surcharge studies

chemical characteristics of the black cotton soil mass during and after saturation seem to be responsible for such an effect. An examination of the vane shear strength indicate S that Su also developed rapidly with depth as can be seen from Figure 13 and has remained constant below a depth 1 m or so. The cohesion value at a depth of around 1 m for Poona soil is of the order 0.66 kg/cm^2.

It was observed that vane shear strength from embeded vane was much higher than that of the vane shear strength after the removal of the overburden (Katti et al 1977a). The lateral pressure developed is of the order 3.35 kg/cm^2. The ratio of lateral pressure to cohesion is around 5.

Subsequently Joshi (1978) under the guidance of the author conducted the tests to evaluate the lateral pressure development of an expansive soil sample under various surcharges using set up shown in Figure 15 during saturation and also after the release. Some of the important results of the study are shown in Figures 16 and 17.

The ratio of lateral pressure to vertical stress is very high at low surcharges, being around 10 for the surcharge due to weight of soil alone at 28 cm depth. The ratio decreases rapidly to around 2.10 for surcharge of 2.0 kg/cm^2. The decrease of ratio with further increase in surcharge is gradual, but in no case the ratio falls below 1, for the surcharges in the range of 0.0 to 5.0 kg/cm^2.

Even after the release of surcharges, considerable amount of lateral pressures are retained in the soil sample as can be seen from Figure 18. These values are as high as 3.61 kg/cm^2 in certain cases. The retention of lateral pressure of the higher order may be attributed to the developed cohesion in the soil-water system which is existing even after the removal of surcharge, as can be seen from Figure 19. It has already been mentioned that the vane shear strength of soil measured by embedded vanes is nearly 2.0 to 2.5 times that obtained after the removal of overburden. The ratio of retained lateral pressure to developed cohesion is around 4 to 5.

In case of black cotton soils, as mentioned earlier, the type of clay mineral present in 2 micron clay fraction is montmorillonite type having expanding lattice. The electrical charges are present both on the surface of particle and also within the interlayers. When such a particle comes in contact with water adsorbed, water films are formed around the surface of clay particle and also water is drawn into the interlayers. The thickness of adsorbed water layer and cohesive bond depends upon the nature of soil particles, their size and structure as well as the interlayer substitutions. Drawing up of water molecules into the interlayer may give rise to heave and if prevented may give rise to swelling pressure.

In this case with clay mineral having expanding lattice cohesive forces are developed around it due to adsorbed water and also outward forces get developed due to ingress of water into the interlayers. Both these actions take place simultaneously.

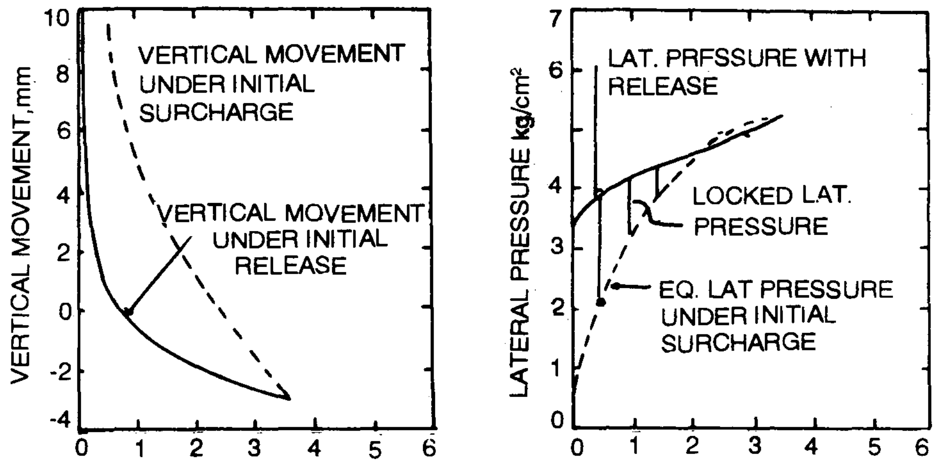
However, from the examination of the structure of clay mineral the adsorbed water film may get formed around the clay particle at a faster rate than the ingress of water into interlayer producing expansion. The cohesive forces thus developed may come into effect and resist heave. In other words, resistance to heaving depends on how much and how fast the cohesive forces are developed around the particle.

The initial condition of saturation are responsible for development of cohesion. The cohesion thus formed is found to be retained in the system, if it is not physically disturbed.

Skempton (1964) in his work on London clays attributed the over-consolidation effect to the presence of glaciers in the past geological history. This in, turn is considered responsible for high unconfined compressive strength of London clay. This indicates that the initial surcharge plays an important role in the development of cohesion in the glacial material and secondly even after receding of glaciers the cohesion formed in the underlying deposit seems to continue to exist for years together.

Even in expansive soil system similar effect seems to take place due to internal intrinsic characteristics of expanding clay mineral and their orientation. When the soil particle comes in contact with plate the adsorbed water bonds are broken and pressure is exerted on the plate. This may be the reason for development of lateral pressure on the plates.

In the large scale tests and field it is observed that the swelling and swelling pressure are balanced to various degrees with depth and almost whole swelling pressure is balanced at 1 to 1.2 m below the surface. As mentioned earlier, to visualise the force system an attempt is made to develop a conceptual physical model as described below.



a) VERTICAL MOVEMENT Vs SURCHARGE

b) LATERAL PRESSURE Vs SURCHARGE

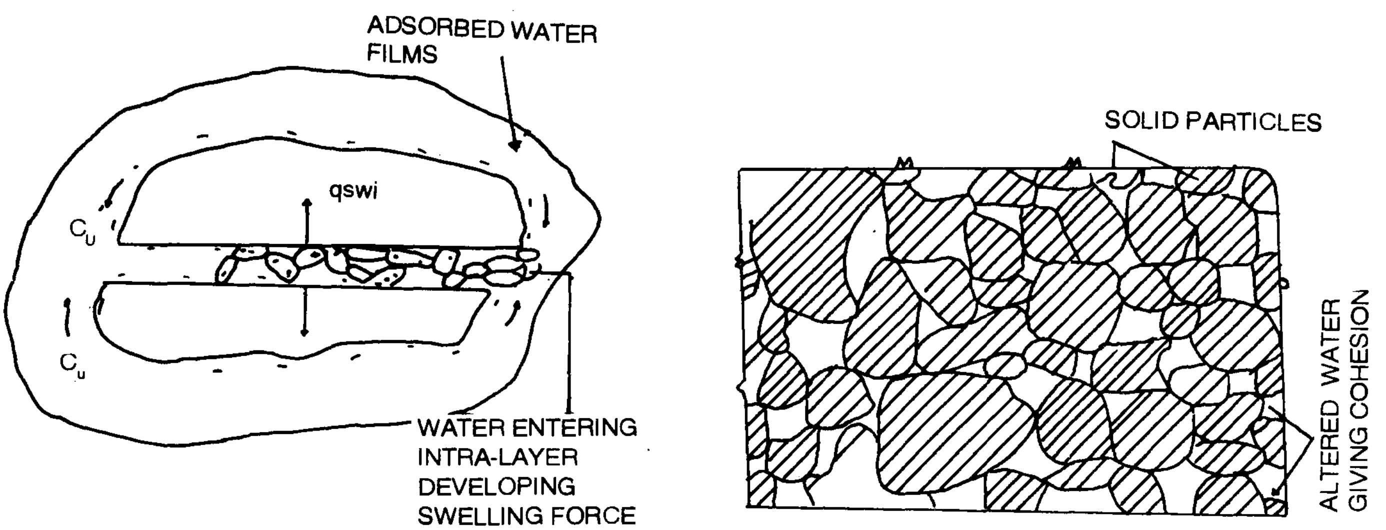
FIGURE 19 Variation of vertical movement and lateral pressure with release of surcharge from 3.5 kg/cm²

CONCEPTUAL MODEL

The conceptual model is proposed on similar lines to that of Kozeny's idealization (Taylor 1948) for flow through large number of capillaries in the soil, applying the principles of flow through pipe. In pipe flow, the pipe is stationary and water flows and the drag is created at the contact surface of water and pipe. In the present case water is stationary and soil particles are expanding in relation to stationary water. This movement produces drag on the clay particle.

A saturated expansive soil mass consists of solid phase due to all size fraction of soil mass and liquid phase due to presence of water. In the soil sample, the clay particles are randomly distributed and in the presence of water they are prone to swell. The clay particles in turn are surrounded by the nonclay particles and the clay particles with different directions of orientations of their c-axis. (Levine 1946, Uppal and Palit 1959).

Thus the incipient movement caused in the clay fraction may also get transmitted to clay particles and also to other soil particles. Such movement results in the mobilization of cohesive forces in the soil-water system, wherein, the water phase is static in relation to soil particles.



a) SINGLE EXPANDING PARTICEL WITH ADSORBED WATER LAYER RESISTING THE EXPANSIVE FORCES

b) SOIL PARTICLES EXPANDING IN RELATION TO ALTERED WATER SYSTEM (KOZENY'S RESERVE MODEL)

FIGURE 20 Conceptual cohesion model

In the present context the area occupied by solid particles can be assumed as the area responsible for transmitting the pressure (Figure 20). In this figure hatched parts are the surfaces of soil particles and the remaining part is water influenced by electrical charges on the surface of soil particles.

Based on this model an attempt is made to relate porosity, size and shape of particles, swelling pressure and the developed cohesion:

Let A = cross sectional area $\quad$ A_s = area of portion occupied by soil particles

n = porosity $\quad$ e = void ratio

Assuming that the area distribution between the solids and the voids area in the same ratio as the volume, then $A_s = A(1 - n)$ ...(6)

In the present case one layer of soil particles at a time is considered.

Let D be the average diameter of the particles such that the surface of all the particles together will be represented by the surface area of N number of particles having average diameter D.

Consider the particles as cylinders having the same diameter D and of height h with a volume equivalent to that of the sphere of diameter D.

Then

$$\text{Volume, } V = \frac{\pi D^2 h}{4} = \frac{\pi D^3}{6} \qquad \text{...(7)}$$

Hence h=2/3 D

The number of particles N in a given area A is given by

$$N = \frac{4\,A_s}{\pi D^2} = \frac{4A(1-n)}{\pi D^2} \qquad \text{..(8)}$$

The total area of N number of particles is given by

$$A_s = N\pi D^2/4 \qquad \text{...(9)}$$

Let Q_{sw} be the lateral swelling pressure as observed on the piston after the loads were released after saturation.

It may be considered reasonable to assume Q_{sw} in the lateral direction being equal to that in vertical direction in the light of previous work.

In the present analysis as there is lateral restraint the incipient movements is in the vertical direction only.

The swelling pressure exerted by the area A_s of the soil particle will be given by

$$Q'_{sw} = \frac{Q_{sw}}{(1-n)} \qquad \text{....(10)}$$

For the equilibrium of the layer under consideration the net vertical force must be equal to zero.

i.e. Upward force due to swelling pressure - resistance due to drag-resistance due to weight = 0

If the contribution due to the weight of soil above the layer under consideration is marginal, then the equation for equilibrium reduces to,

Upward force due to swelling pressure -- Resistance due to drag = 0

i.e.

$$N\frac{\pi D^2}{4} Q'_{sw} - N\pi Dhc'_u = 0 \qquad \therefore \qquad Q'_{sw} = \frac{8}{3} c'_u$$

$$\text{But} \quad Q'_{sw} = \frac{Q_{sw}}{(1-n)} \qquad \therefore \qquad \frac{Q_{sw}}{1-n} = \frac{8}{3} c_u \qquad \text{...(11)}$$

If porosity, n = 0.5

$$Q_{sw} = \frac{4}{3} c'_u \qquad \text{...(12)}$$

Where c'_u is the cohesion value obtained as it existed in the soil water system without any disturbance to the soil mass.

It was observed in earlier work that the cohesion c'_u as it existed without any disturbance of the top layer of the soil is nearly 1.5 to 3.5 times the cohesion value c_u obtained after removing the soil up to the layer under consideration. Considering the value of c_u equal to 2.5 times c'u the above equation obtained as:

$$Q_{sw} = 3.33\ c_u \quad .....(13)$$

Similarly considering particles to be cubes, a similar relation is obtained as

$$Q_{sw} = 5\ c_u$$

The two cases discussed indicate that depending on the shape of particles the developed cohesion c'_u needed to balance the swelling pressure, Q'_{sw}, varies between 3.3. c'_u and 5.0 c'_u

Hence it can be said that $Q_{sw} = \alpha c_u$ ....(14)

Where α is shape factor varying between 3.33 and 5.0

The equivalent average diameter D in case of cylindrical particles or in case of cubic particles can be estimated by referring to the mechanical analysis curve.

The above analysis emphasises the role of developed cohesion in the soil water system in resisting the swelling pressure considering the lateral pressures at piston level when all the dead load surcharge is released and neglecting the effect due to weight of soil above the piston level. The constant α for the given soil varies from 2.80 to 5.05 as determined by experimental values (Joshi 1978).

In the present investigation except in one case the value of α is always less than 5, showing that mobilised cohesion is sufficient to balance the forces due to swelling pressure of soil.

SHEAR STRENGTH STUDIES

In large scale studies it is observed that even at shallow depth of 30, 50 and 100 cm, the lateral pressures of the Poona black cotton soil recorded are 1.2, 2.9 and 3.35 kg/cm² respectively. The vertical stress due to overburden in each of these case is 0.05, 0.09 and 0.18 kg/cm². The ratio of lateral pressure to vertical stress is far greater than 1. The vane shear strength values S'_v prior to removal of overburden, at the above corresponding depths are 0.525, 1.025 and 1.248 kg/cm².

In conventional soil system the relation between the vertical stress and lateral pressure in case of horizontal surface is given by Terzaghi (1943) based on conjugate stress relations as :

$$\sigma_a = \gamma h \tan^2 (45 - \phi/2) - 2c \tan (45 - \phi/2)$$

for active condition $\sigma_p = \gamma h \tan^2 (45 + \phi/2) + 2c \tan (45 + \phi/2)$

for passive condition

wherein ϕ is the unit weight of material and h is the depth. The shear strength condition assumed for failure is

$$s = c + \sigma \tan \phi \quad ...(15)$$

These relations indicate that lateral pressure development in a conventional cohesive soil system is a function of vertical stress, cohesion and frictional parameters of the soil mass. An examination of the equation for active case shows that in a conventional soil system the cohesion parameters bring about reduction in lateral pressure. A cohesion value of say 0.55 kg/cm² according to the following equation indicates that up to a depth of around 6 m there should not be any lateral pressure due to soil.

$$z_o = \frac{2c}{\gamma} \quad ...(16)$$

However, in case of expansive soils, the experimental data have revealed very high lateral pressures of the order of 3.35 kg/cm² at a depth of 1 m. The ratio of lateral pressure to vertical stress is as high as 18 even though there exists cohesion of the order more than 0.5 kg/cm².

Another important observation in case of expansive soil is that no heave is observed at a depth of around 1.0 m. The soil sample having same initial moisture and density conditions under an equivalent dead load surcharge of 0.18 kg/cm² representing a soil overburden of 1.0 m showed heave of around 13 per cent. This heave reduced to zero, only when the dead load surcharge reached a value of 3.3 kg/cm². This value is equivalent to 18 m of soil overburden.

These observations clearly indicate that active lateral pressure development is quite different in black cotton soil from that observed by various research workers in conventional soil system (Leonards 1975).

It may be considered reasonable to assume that the soil element at a depth of 1.0 m in the expansive soil is experiencing reaction force of the same value from the container.

Thus, if the soil mass in the container is to be in equilibrium it should have developed adequate shear strenght within the mass in the form of cohesion to balance such high reaction forces also.

Assuming that the soil element in the soil mass is subjected to the passive forces due to reaction as shown in Figure 21, the value of c in terms of lateral pressure can be given as:

$$c = \frac{\sigma_p - \gamma h \tan^2 (45 + \phi/2)}{2 \tan (45 + \phi/2)} \qquad ...(17)$$

If γ is assumed equal to zero and γh is negligible, the value of cohesion will be

$$c = \frac{\sigma_p}{2} \qquad ...(18)$$

In other words the cohesion developed should be in the neighbourhood of 0.50 σ_p or somewhat less. If ϕ is present, c needed further reduces as shown in Table 5.

It is already brought out that an expansive soil sample develops lateral swelling pressure of the same order as the vertical swelling pressure due to the expanding nature of the clay fraction. It is also indicated that the electrical charges acting on the surface and inside the expanding lattice of the clay particle interact with the dipolar water giving rise to adsorbed water layer around the clay particle. The cohesive force produced in the water at certain distance from the surface of the clay particle may be of lesser magnitude than the cohesive force produced just near the particle due to adsorbed water films.

Table 5

COHESION NEEDED TO BALANCE THE PASSIVE PRESSURE

ϕ Degrees	Cohesion needed Kg/cm²
0	$0.500\sigma_p - 0.500\gamma h$
10	$0.419\sigma_p - 0.596\gamma h$
15	$0.384\sigma_p - 0.651\gamma h$
20	$0.350\sigma_p - 0.714\gamma h$
25	$0.318\sigma_p - 0.785\gamma h$
30	$0.289\sigma_p - 0.866\gamma h$

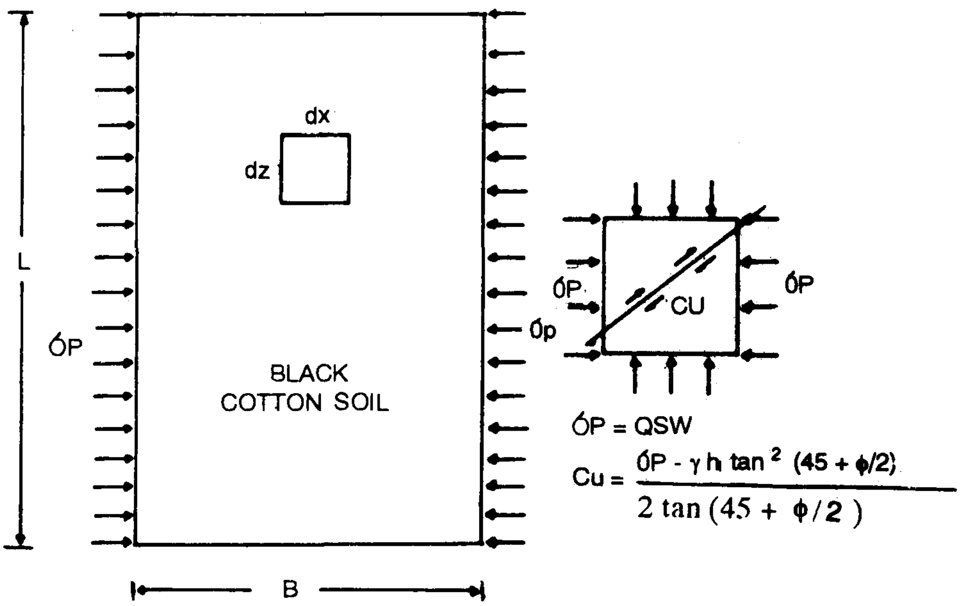
An element of expansive soil at a depth say around 1.0 m, in the large scale tank has to be in equilibrium with respect to two sets of forces:

(i) equilibrium of individual clay particles inside an element against swelling and swelling pressures.

(ii) equilibrium of an element as a whole under the developed reaction forces and the overburden.

It is already indicated that the adsorbed water layer formed around the clay particle can resist swelling pressure of particle. This aspect may satisfy the requirements of equilibrium of individual particles.

The cohesive forces acting between the particles need to be of adequate magnitude to balance the external force system especially arising due to the reaction in lateral direction. The vane shear value represents cohesion between particles to some extent.



BLACK COTTON SOIL SAMPLE IN LARGE SCALE TANK

FIGURE 21 Equilibrium of a black cotton soil element due to cohesion under developed lateral forces

It is observed from the data that the vane shear strength values, as mentioned earlier are nearly 0.3 σ_p to 0.5 σ_p. Earlier it is also shown that S_v values as obtained do indicate possibility of balancing the swelling of individual clay particle under a given condition.

In a semi-infinite swelling soil media specially in the top 1.0 to 1.5 m zone cohesion seems to be developing rapidly with depth and by the time 1.0 to 1.5 m is reached the swelling action of clay seems to have been completely balanced.

It is also observed from the studies that equivalent dead loads or cell pressures may not simulate stress conditions with depth in an expansive soil mass. However, in case of conventional soils such imposed dead loads or cell pressures are expected to simulate stress conditions due to soil overburden etc. in the field.

These observations clearly show that shear strength development in an expansive soil media with depth is different from that observed in conventional cohesive soil system. This difference may be attributed to.

(a) expanding lattice structure of clay particles and their distribution in non-clay particles, voids etc.

(b) interaction between electrical charges on the clay particles and the surrounding dipolar water molecules.

In a conventional soil system main aim of evaluating shear parameters is to evaluate shear strength on various planes in a soil mass by combining these parameters with appropriate stresses computed by knowing the density of soil mass.

In case of expansive soil equivalent dead loads or cell pressures do not represent the effect due to soil as these dead loads do not reflect the characteristics produced in the soil mass due to expanding lattice structure of the clay particle and the interaction between electrical charges on the surface of clay particle and the dipolar nature of water surrounding it.

These observations indicated that it is necessary to evolve appropriate methods to obtain equivalent cell pressure or normal stresses which would give the realistic shear strength existing at various levels in expansive soil mass under fully saturated conditions.

The vane shear strength studies with depth prior to and after the removal of overburden gave some idea about the probable shear strength existing with depth in field or in the corresponding large scale tests shown in Figures 13 and 14.

It is realised that in black cotton soil there is a necessity to evaluate shear strength development with depth, based on small size shear strength tests. To achieve the same it may be necessary to approach the problem in the following way:

(i) to prepare black cotton soil sample of the size used in conventional shear strength testing devices such that the density and internal stress conditions of the sample closely represent the intended corresponding sample from the field or large scale tank,

(ii) to evaluate stress-strain relations under various stress conditions using direct shear or triaxial shear testing devices and also to obtain shear parameters and

(iii) selecting various stress values and appropriate shear parameters to indicate which combination of values would yield similar shear strength values obtained with the help of vane shear equipment in a particular large scale or in field.

Keeping this in view several tests were conducted, using direct shear, triaxial and vane shear equipments (Katti and Kamat 1967, Katti *et al* 1969*b*, Katti *et al* 1977*a*, Katti *et al* 1977*b*). Few typical case studies are presented here.

DIRECT SHEAR TESTS

Case I - Sample was compacted to a ratio corresponding to that observed in the constant volume zone below 1.0 m. The moulding moisture content was zero. The samples were subjected to various normal stresses within and beyond swelling pressure value during saturation. After full saturation the direct shear tests were conducted at a deformation rate of 0.0125 mm/min and the stress-strain characteristics were recorded.

In this case, under stresses less than swelling pressure there was actual increase in void ratio during

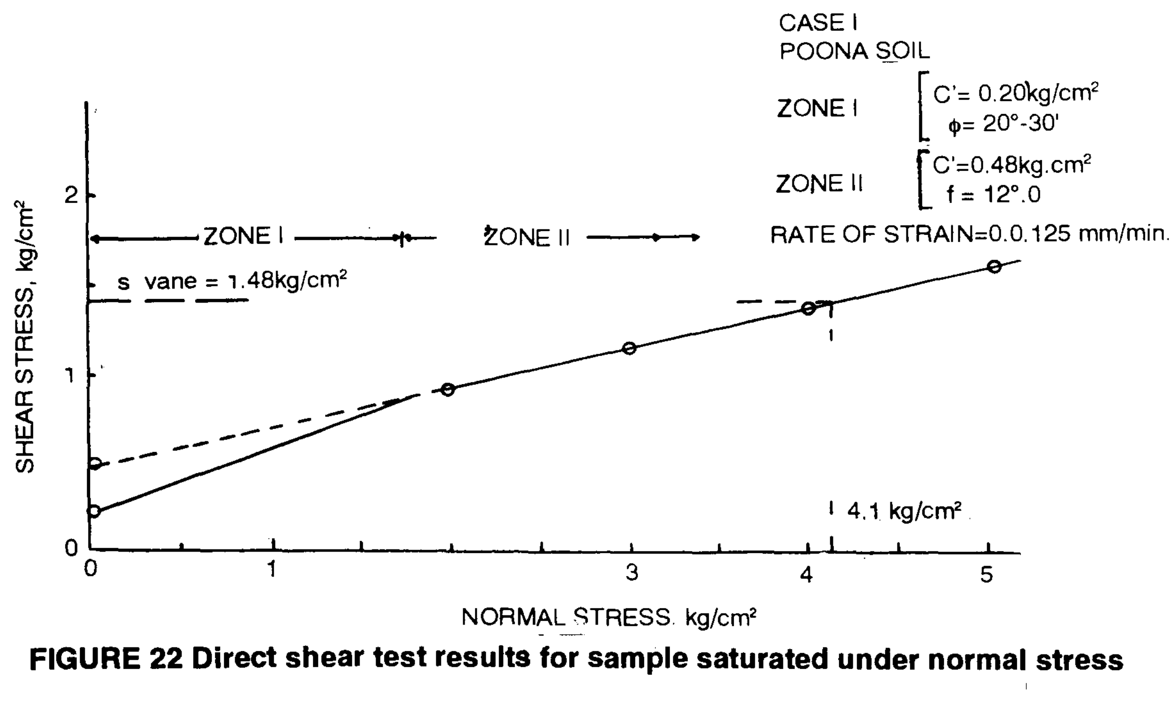


FIGURE 22 Direct shear test results for sample saturated under normal stress

saturation. Beyond swelling pressure, however, there was some amount of settlement. In nearly all the tests conducted it was observed that the vane shear strength prior to removal of overburden was around the shear strength corresponding to normal stress in the neighbourhood of swelling pressure.

The shear stress *versus* normal stress curve shows bilinear trend. A typical curve for Poona soil is shown in Figure 22.

Case II - In this case the samples were moulded to the void ratio corresponding to that in earlier case. Prior to saturation the samples were restrained from swelling, using a frame shown in Figure 23. After full saturation the samples were placed in direct shear equipment and various normal stresses were applied. In this case the samples showed swelling under zero and 0.1 kg/cm² stress.

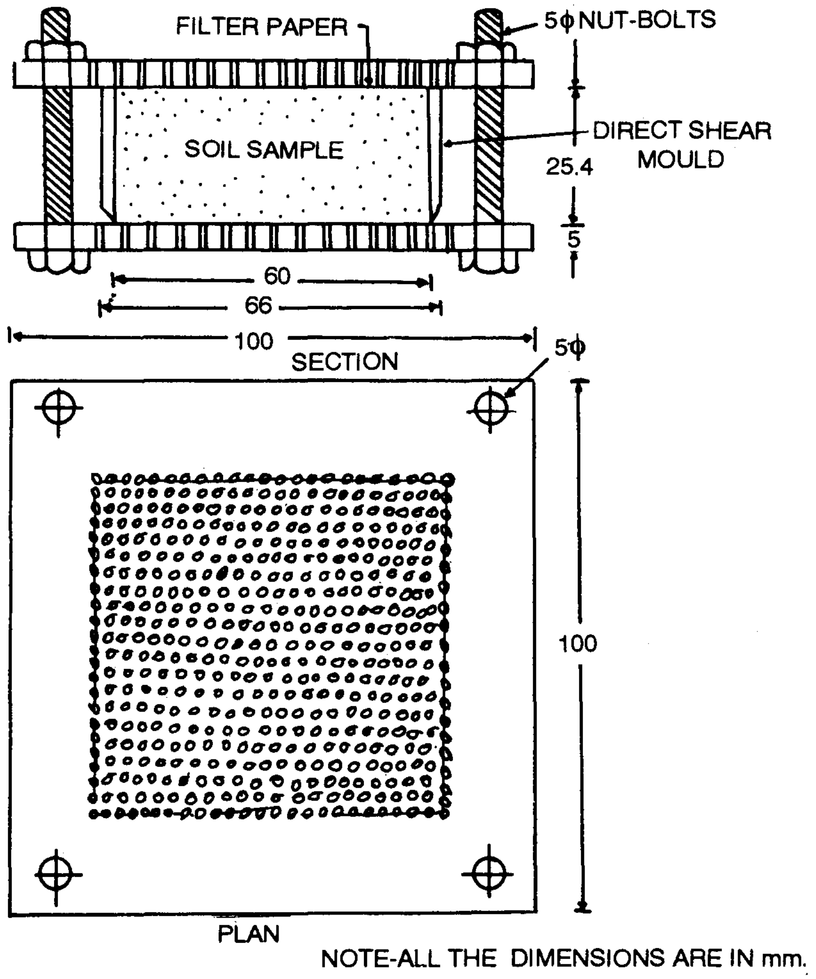


FIGURE 23 Saturation device for direct shear sample

Unlike that observed in case I, under lesser magnitude of normal stress the samples showed marginal settlement. The normal stresses were applied for 24 hours before shearing. The shear stress *versus* strain curves showed initially linear relationship, then it remains constant. The shear stress versus

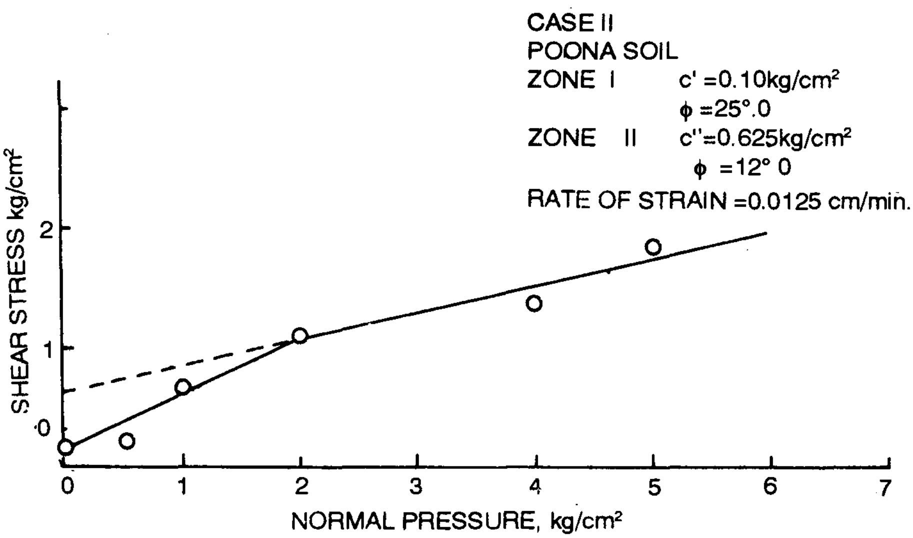


FIGURE 24 Direct shear results for sample restrained during saturation

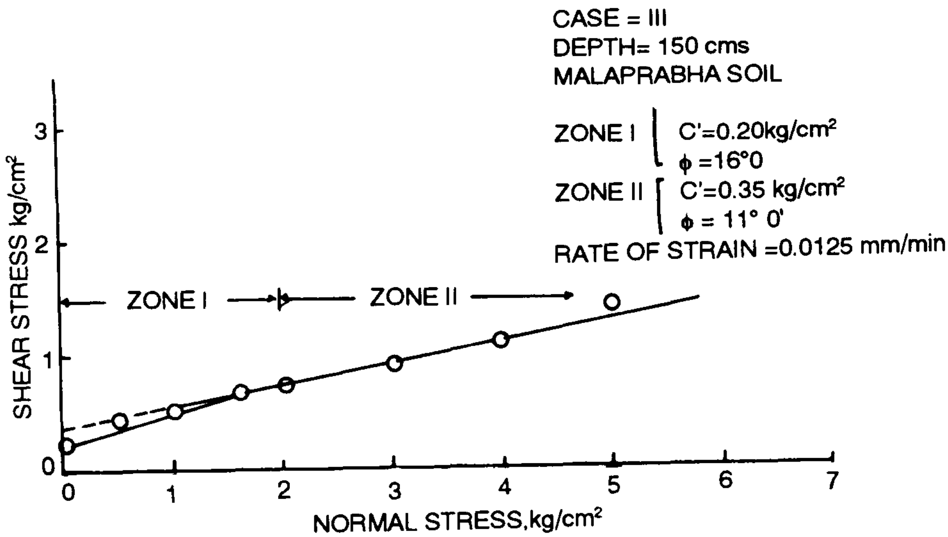


FIGURE 25 Consolidated direct shear test results for samples from large scale tank

normal stress curve showed bilinearity. A typical curve for Poona soil is shown in Figure 24. Bilinearity in curve starts at normal pressure 2.0 kg/cm^2

Normal stress corresponding to the vane shear strength was some what on higher side of swelling pressure.

Case III - The saturated soil samples taken from large scale tank below around 1.5 m depth were subjected to direct shear test. The soil mass showed zero swell at that depth.

The soil samples showed slight heave at zero load. However, they showed marginal settlement at normal stress of 0.1 kg/cm^2.

In this case the samples show shear stress vs strain curve having trend similar to that in Case II. The shear stress vs normal stress curve is shown in Figure 25 which has bilinear trend. The second zone starts from σ_n=1.5 kg/cm^2 for Malaprabha soil.

The vane shear strength value is near about the shear strength value corresponding to normal pressure equal to swelling pressure. A typical stress-strain curve for Malaprabha soil is shown in Figure 26. The swelling pressure of the soil is 2.1 kg/cm^2 and the normal stress corresponding to vane shear value of 0.84 kg/cm^2 prior to release of overburden is found to be around 2.05 kg/cm^2.

From these results the following observations can be made:

In Case I, the process of formation of adsorbed water layer had to take place in the presence of various dead loads. The dead loads do not possess equivalent charges which are observed in the soil system and thus the effect observed at 1.0 m depth takes place only when the dead load is equivalent of swelling pressure.

In Case II, the restraint helped in giving an equivalent swelling pressure load inside the soil system and helped in developing the adsorbed water hull around the particle contacts. As mentioned earlier, the strength of adsorbed water film around individual particles will be much higher than the cohesion in between the particles. When these particles were allowed to swell under load of 0.1 kg/cm^2 they did not show heave but showed some settlement due to particle movement in cohesive mass. However, a detailed study would be necessary to understand the long term behaviour.

The samples from Case III have further indicated that the vane shear strength is near about the shear strength corresponding to the normal pressure around swelling pressure.

For evaluation card c and ϕ parameters it is essential to adopt technique as in Case II and III. But the equivalent value of vane shear strength can be predicted by carrying out a test around $\sigma_n=Q_{sw}$ in all the tests.

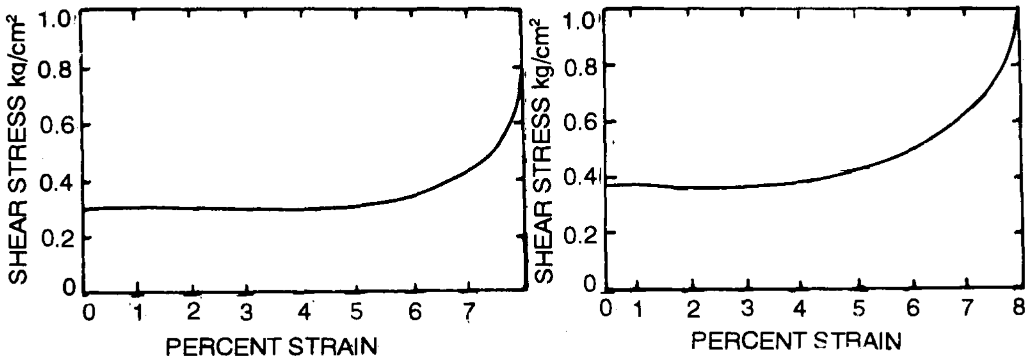


FIGURE 26 Shear stress versus strain curves

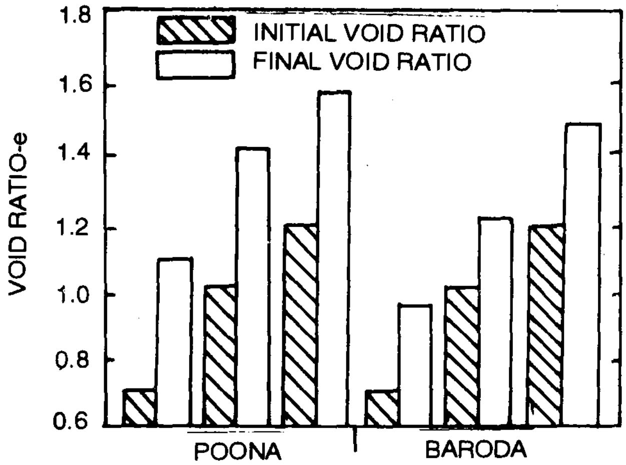


FIGURE 27 Bar graph for moulding void ratio and void rations after saturation

TRIAXIAL SHEAR TESTS

Case IV - The soil samples were saturated in a triaxial cell adopting special technique shown in Figure 8. The moulding void ratio was unity, using minimum moisture content.

The samples were subjected to cell pressures within and beyond swelling pressure range. After the saturation the samples were sheared at 0.025 mm/min without allowing volume change.

During saturation the soil sample showed increase in void ratio as seen from Figure 27. However,

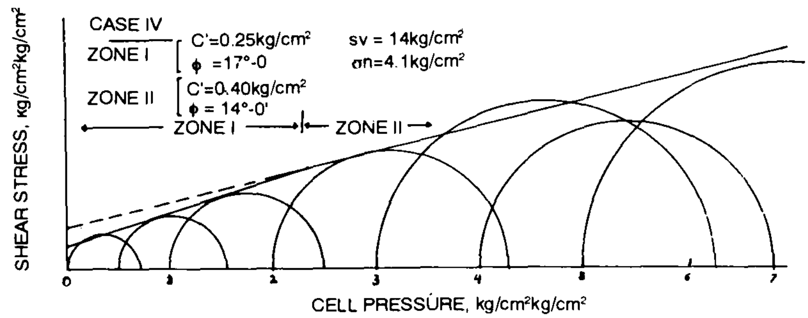


FIGURE 28 (a) Mohr strength envelope for Poona soil sample

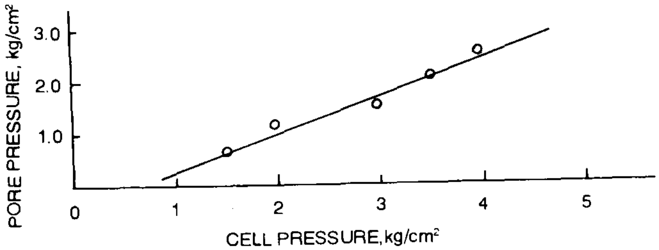


FIGURE 28(b) Peak pore pressure versus cell pressure for Poona soil sample

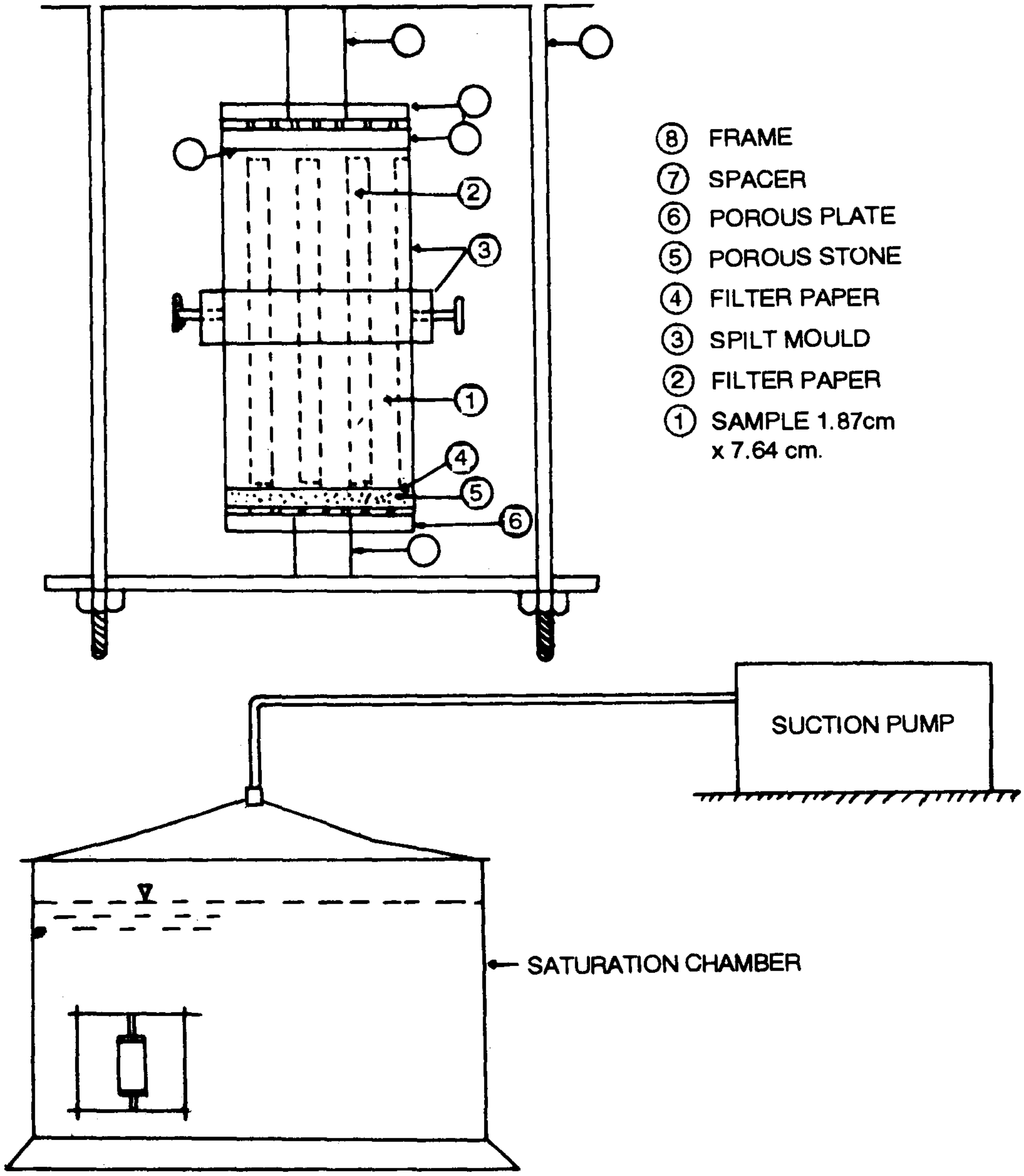


FIGURE 29 Saturation device for triaxial sample

Poona soil, after cell pressure equal to 2.3 kg/cm^2 did not show any increase in volume but reduction in void ratio was observed.

In general the shear stress, τ, *versus* cell pressure showed almost bilinear curve. A typical curve is given in Figure 28a.

The vane shear strength prior to release of overburden may be seen to be in the neighbourhood of swelling pressure from the stress strain curve for Poona soil. The results of the pore pressure development under different cell pressures are shown in Figure 28b.

Case V - In this case soil samples were saturated under constrained conditions. The samples were saturated in a split mould assembly at void ratio of unity and zero moisture content. The frame used for constraining is shown in Figure 29.

After full saturation, samples were removed out of assembly and were subjected to cell pressures lower and higher than the swelling pressure. After cell pressures were applied for a period of 24 hours it was observed that the change in void ratio during each incremental cell pressure was marginal at smaller values of cell pressures. But decrease in volume was significant after cell pressure was equal to swelling pressure.

A typical stress-strain curve is shown in Figure 30a. The Mohr's envelope shows bilinear trend (Figure 30b.). The vane shear strength of 1.4 kg/cm^2 for Poona soil is found to be near about the value of shear strength corresponding to cell pressure of 3.5 kg/cm^2 which is slightly on higher side of swelling pressure.

Case VI - In this case also the soil samples were collected from 1.5 m depth in large scale tank as in Case III.

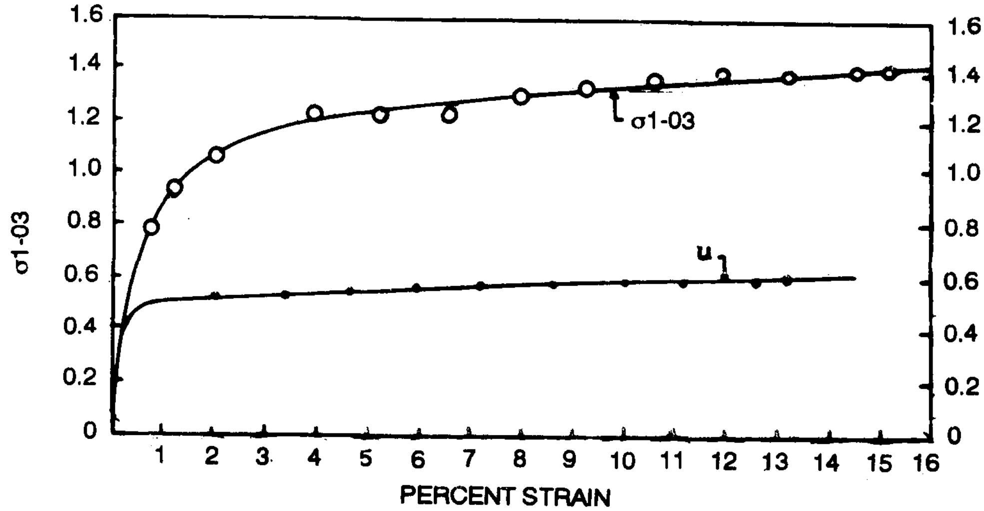


FIGURE 30(a) Deviator stress versus strain

These samples were subjected to cell pressures within and beyond the swelling pressure range and sheared at a rate of 0.025 mm/min.

The Mohr's envelopes show a bilinear trend as is seen from Figure 31.

The stress versus strain curves indicated the linear trend upto strain around 6 per cent which are curvilinear in nature.

As in other cases, the vane shear strength value is near about the shear strength corresponding to normal pressure equal to swelling pressure, around 2.1 kg/cm² for Malaprabha soil.

From these observations following inferences can be drawn:

The trends in direct shear tests and triaxial tests were nearly the same. The shear parameters were different in two cases. In many cases the direct shear test values were lower than the triaxial values.

Effect of rate of strain -Case VII - The direct shear soil samples from the hearting of Narayanpur dam were saturated by confining in the frame shown earlier. The samples were removed after the full saturation and were subjected to direct shear tests after consolidating for a period of 24 hrs. The normal stresses applied were within and beyond swelling pressure range.

The soil samples from the same depth were sheared at two different rates of strain as 0.0125 mm/min and 0.125 mm/min. The shear stress vs normal stress plots are shown in Figure 32. It may be seen that ϕ increases and c decreases after testing at lower strain rate (Katti, 1978).

	Zone I		Zone II	
Deformation rate m/min	**c (kg/cm²)**	**ϕ**	**c(kg/cm²)**	**ϕ**
0.0125	0.10	24° 00'	0.495	14°00'
0.125	0.05	21°30'	0.400	11°30'

CONSOLIDATION CHARACTERISTICS

The consolidation characteristics of black cotton soil from several places were studied under different set of conditions (Katti and Sadasivan 1967).

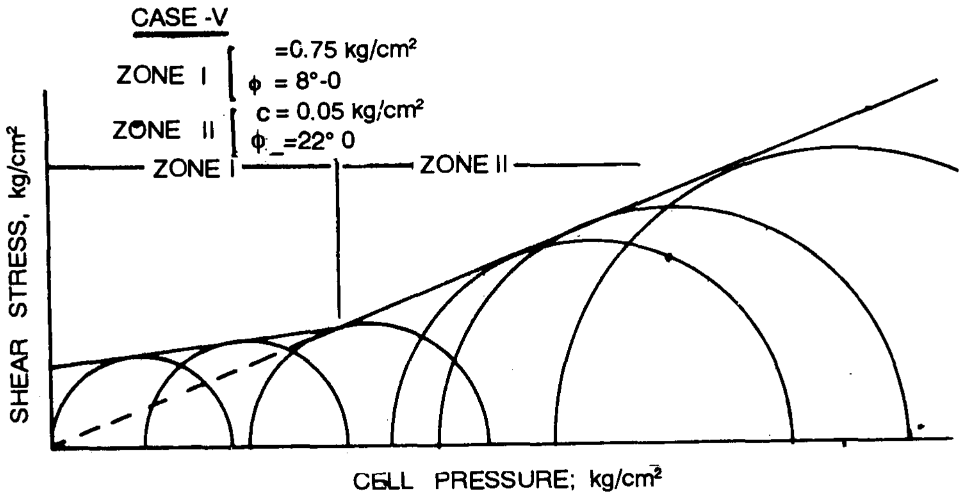


FIGURE 30 (b) Mohar strength envelope of consolidated shear test for Poona soil sample undrained triaxial

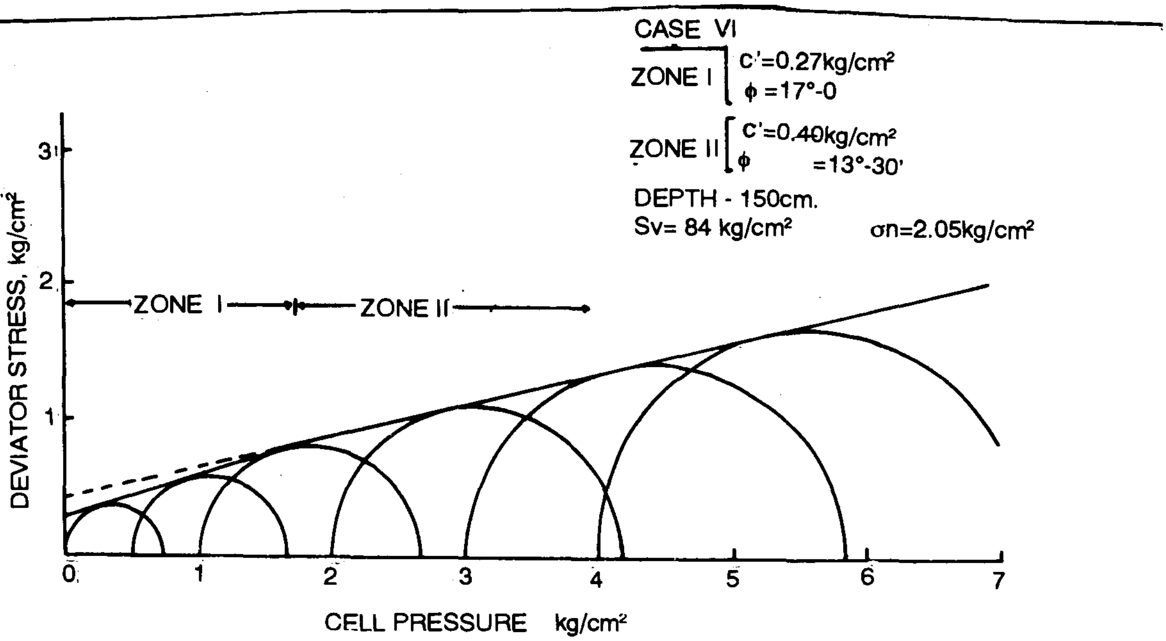


FIGURE 31 Triaxial test results for Malaprabha soil sample from large scale tank

(Consolidated undrained test with pore pressure measurement)

Two cases are presented here:

I. Consolidation of sample moulded at oven dry state and saturated under 0.1 kg/cm².

II. Consolidation of sample saturated in a large scale tank.

The test data are presented under (a) behaviour under 0.1 kg/cm², (b) e-log p characteristics, (c) time settlement characteristics and (d) unloading characteristics.

Case I - The oven dry soil from Poona was subjected to consolidation. For this the samples were moulded at a void ratio of 1.2. The soil was subjected to saturation under 0.1 kg/cm². The void ratio increased from 1.20 to 1.69 after about 48 hours, showing 22 per cent swelling.

The e-log p relation presented in Figure 33 shows a pre-compression value of 0.37 kg/cm² and beyond that the curve shows straight line relationship. The C_c value is around 0.55.

Time settlement curve shows 40 to 50 per cent settlement to be taking place initially in case of a swollen soil.

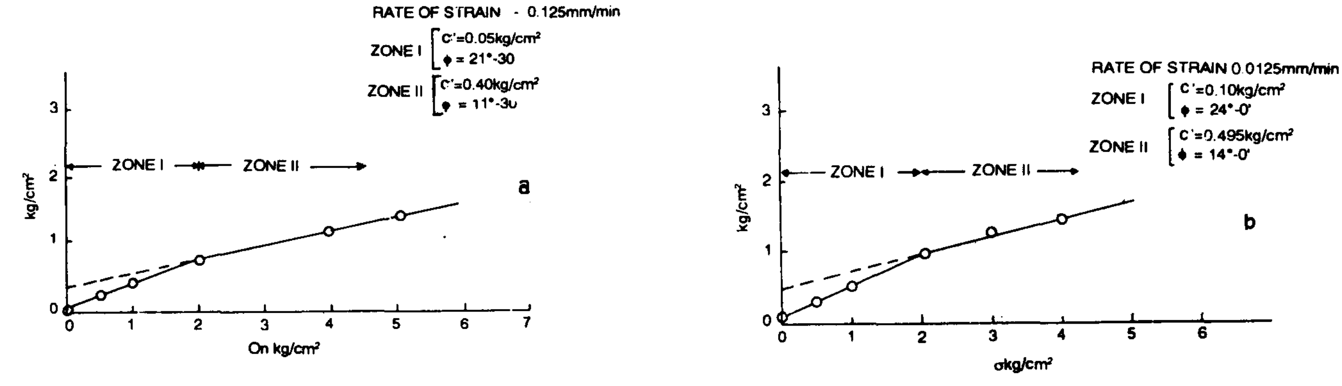


FIGURE 32 Effect of rate of strain on shear strength of Narayanpur earthen dam sample
(Consolidated direct shear tests)

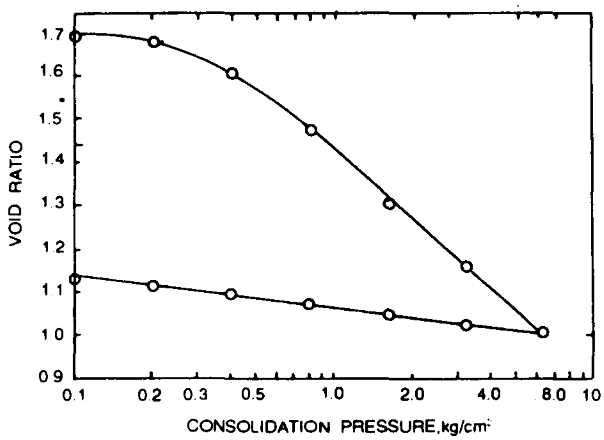


FIGURE 33 e-log p curve for Poonam soil

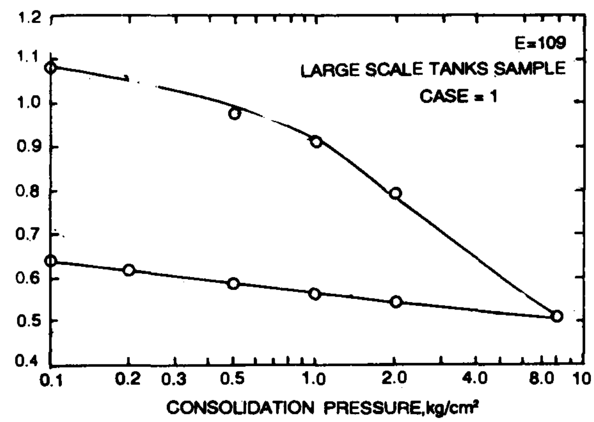


FIGURE 34 e-log p curve Malaprabha soil

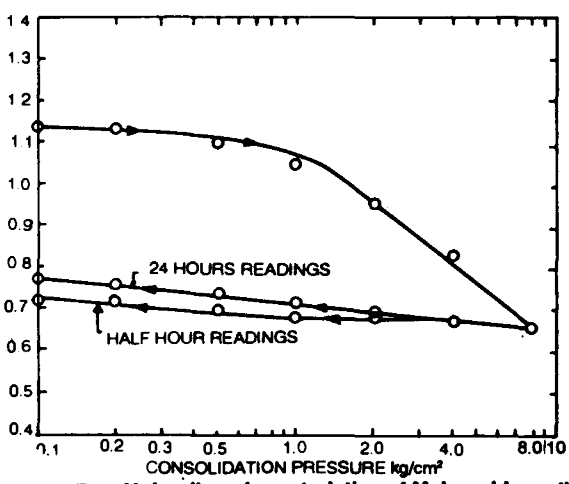


FIGURE 35 Unloading characteristics of Malaprabha soil

The one day rebound value expressed as percentage of compression is around 20 per cent.

Case II - The samples from constant volume zone below 1.5 m depth in a large scale tank filled with Malaprabha soil was subjected to consolidation tests. The samples were completely saturated.

The soil sample was subjected to a pressure of 0.1 kg/cm² and after 24 hrs. it was observed that the dial gauge reading was showing either slight settlement or is nearly constant.

The e-log p curve is presented in Figure 34. The straight line portion starts beyond a pressure of around 1.5 kg/cm². The C_c value is around 0.5. The time-settlement characteristics show that the settlement is gradual and about 30 per cent settlement takes place at higher load increments in first 1 or 2 min.

It is observed that, at higher loads, the soil attains equilibrium void ratio after 30 mm. Below 4 kg/cm² however, the unloading of soil sample does not attain equilibrium but continues to heave as observed after 30 min and 24 hrs as shown in Figure 35. Because of very high amount of swelling the very meaning of consolidation needs to be carefully looked into. In conventional soil-system normally under various loads the samples settle and give much lower void ratios than compaction void ratio or field void ratio. In the present case if the same type of method is adopted the swelling of sample overshadows consolidation. It may be noted that instead of load giving rise to the compaction, it gives rise to heave.

Studies with cohesive nonswelling soil layer

The studies related to the effectiveness of a cohesive non-swelling soil (CNS) layer in altering some of the properties of black cotton soil are indicated here.

Concept

Earlier studies have shown that cohesive forces of significant magnitude are developed in an expansive soil system during saturation with depth and are responsible for reducting swelling and counter-acting swelling pressure (Katti *et al* 1978*a*, Katti and Kate 1975, Katti and Kulkarni, 1967, Katti *et al* 1969*c*). This behaviour is attributed to (i) structure of expanding clay mineral in the clay fraction present in the black cotton soil mass, (ii) influence of electrical charges present on the surfaces of clay particles and other soil particles on the dipolar nature of water molecules producing varying magnitudes of cohesion depending upon the distance of the molecules from the surface of clay particles, (iii) initial placement conditions of soil prior to saturation, (iv) configuration of other particles and voids in relation to an individual clay particle and (v) weight of the soil mass above.

A thickness of 1 m of black cotton soil is found to resist swelling pressure of almost 3.3 kg/cm2 and thus producing an equivalent dead load of soil surcharge of 18 to 20 m of conventional soil mass. The average cohesion developed within 1 m depth is around 0.6 kg/cm² (Katti *et al* 1977*a*).

These observations clearly showed that if an environment similar to that existing at 1 m depth in the black cotton soil with equivalent cohesion is produced and at the same time the system used in producing this environment does not swell or exert swelling pressure, then it may be possible to counteract the swelling pressure and heave of a swelling soil mass such as black cotton soil.

There are clay minerals other than montmorillonite which do not expand but can develop cohesion in a soil system if they are present. Some of the murrums existing in and around black cotton soil area do possess such properties. The soil system having above-mentioned characteristics is designated as cohesive non-swelling soil.

A series of experiments were conducted by the author and his associates Kate, Kulkarni, Bhangale (Katti *et al* 1969*c*, Katti *et al* 1978*a*), at I.I.T. Bombay, in large scale tanks to evaluate alterations brought out in underlying black cotton soil covered with murrum of varying thickness and cohesion.

A few typical results of the study are shown in Figures 36, 37 and 38. Properties of some of the murrums used in the study are given in Table 6.

It is observed that:

(i) cohesion remaining constant, the heave of the underlying black cotton soil reduced rapidly with increase in thickness of murrum and attained a value of no heave when the thickness of murrum reached 1 to 1.2 m. Decrease in heave with increase in thickness is not linear but exponential in nature.

(ii) Thickness of murrum needed to resist a prevalent swelling pressure of the underlying black cotton

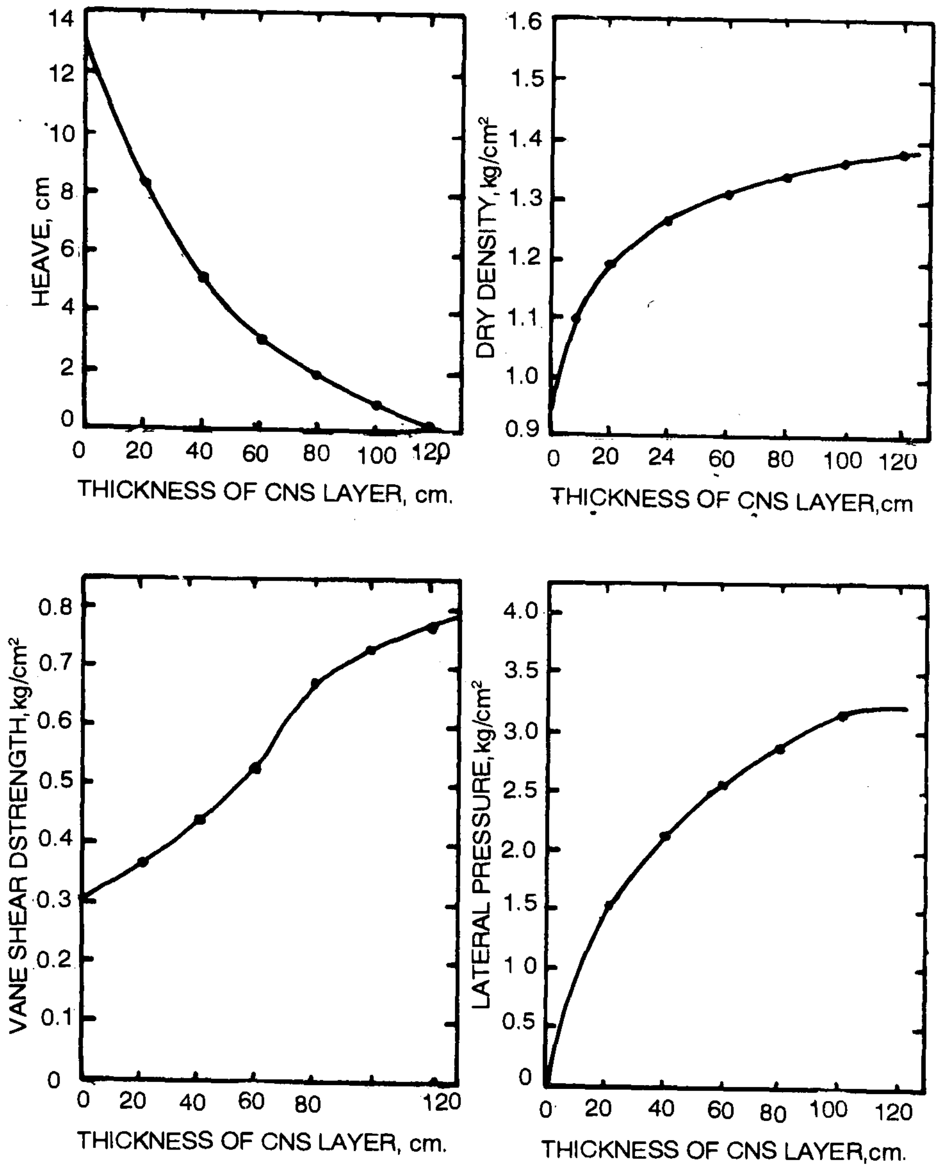


FIGURE 36 Properties of expansive soil at interface versus thickness of CNS layer.

soil varies with cohesion. Normally increase in cohesion reduces thickness and the relation may not be linear.

(iii) Shear strength of the underlying black cotton soil at the interface and below, increases with thickness of murrum and attains the shear strength value nearly equal to that at no volume conditions, when thickness of murrum reaches a value of which prevents a whole system from upward heave. The increase in shear strength is not linear.

These experiments clearly showed that in addition to cohesion developed in the black cotton soil, the cohesion of the CNS layer and its thickness are the governing criteria in counteracting swelling and swelling pressure.

Taking into consideration the following characteristics of black cotton soil media.

(i) random distribution of clay particles in the soil mass,

(ii) structure of montmorillonite clay mineral having expanding characteristics in the c-direction which is predominant in the clay fraction of the black cotton soil,

(iii) orientation of individual clay particles in various directions in the black cotton soil mass and

(iv) developed cohesion in the CNS environment in the black cotton soil and coupling it with shear strength characteristics of CNS layer a simplified physical model for the balance of forces as shown in Figure

TABLE 6

PROPERTIES OF VARIOUS CNS MATERIALS

		CNS as blended mix of treated B.C. soil and murrum	Betsur murrum	Powal murrum	Talimorab murrum
1.	**PHYSICAL PROPERTIES**				
	Liquid Limit, %	50.6	34.1	48.2	47.0
	Plastic Limit, %	27.6	22.3	27.3	24.0
	P.I.	23.0	11.8	20.9	23.0
	Shrinkage Limit, %	20.7	20.4	18.3	
	Specific Gravity	2.61	2.55	2.71	2.74
	Swelling pressure at constant volume condition at zero m.c., kg/cm^2	0.00	0.10	0.00	0.05
	Standard Proctor Density, g/cc	1.30	1.51	1.47	1.88
	Standard Proctor O.M.C., %	18.0	20.4	26.1	15.2
	Modified Proctor Density, g/cc	-	-		1.94
	Modified Proctor O.M.C., %	-	-		11.60
2.	**TEXTURAL COMPOSITION**				
	Gravel>2 mm, %	5.0	9.0	4.0	17.0
	Sand 2 to 0.06 mm, %	32.3	49.0	43.0	30.0
	Silt 0.06 to 0.002 mm %	34.7	30.0	35.0	20.0
	Clay <0.002 mm, %	28.0	12.0	18.0	33.0
	Textural classification (USBPER SYSTEM)	-	Hard clay loam	Clay loam	Gravelly clay
	Engineering classification (AASHO SYSTEM)	-	A.6	A.(7)7-6	A.2.7
3.	**CHEMICAL ANALYSIS**				
	pH	-	7.00	7.00	7.00
	Organic matter, %	-	0.76	0.25	0.79
	Carbonate content,%	-	0.58	0.00	0.60
	SiO_2 %	-	58.30	51.90	58.20
	Al_2O_3, %	-	17.86	15.63	17.80
	CaO, %	-	3.19	2.40	3.19
	Fe_2O_3, %	-	11.14	16.92	11.20
	Sulphate, %	-	-	-	0.69
	Loss of ignition, %	-	5.25	10.51	4.87
	Base exchange capacity, (m.eq/100 g)				
	200 Sieve Fraction	-	28	32	32
	5μ fraction	-	35	43	35
	2μ fraction	-	38	49	37

39 is indicated. It may be noted that significant amount of swelling pressure is counteracted by developed cohesion. The remaining is balanced by the weight and cohesive forces developed in the CNS layer.

FIELD TRIALS WITH CNS LAYER

Prior to using the process of CNS layer technique in large scale construction, an opportunity was

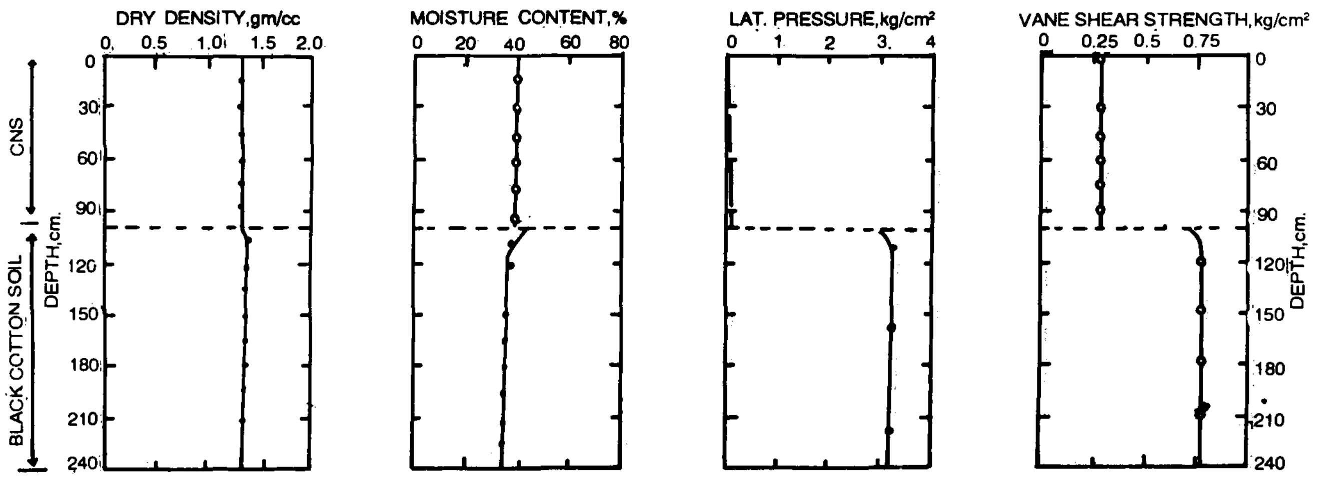


FIGURE 37 Results of large scale test with CNS layer of 1 m thickness after saturation for Poona soil

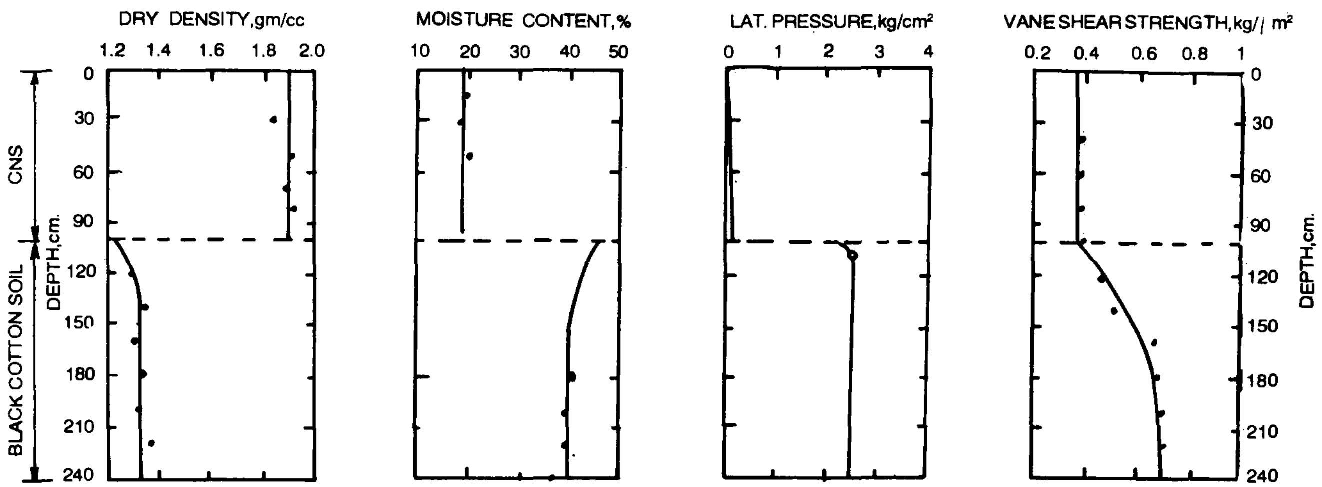


FIGURE 38 Results of large scale test with CNS layer of 1 m thickness after saturation for Malaprabha soil

availed to conduct prototype field studies in Malaprabha Right Bank Canal System which passes through deep seated black cotton soils. In this canal system following conventional methods (Katti *et al* 1978*a*) for linings were used earlier in different stretches of the canal.

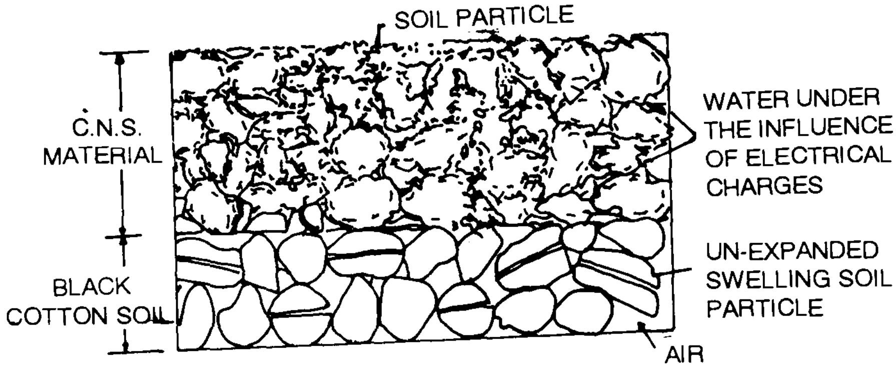
(i) 45 cm thick sand layer

(ii) polythene sheet backing

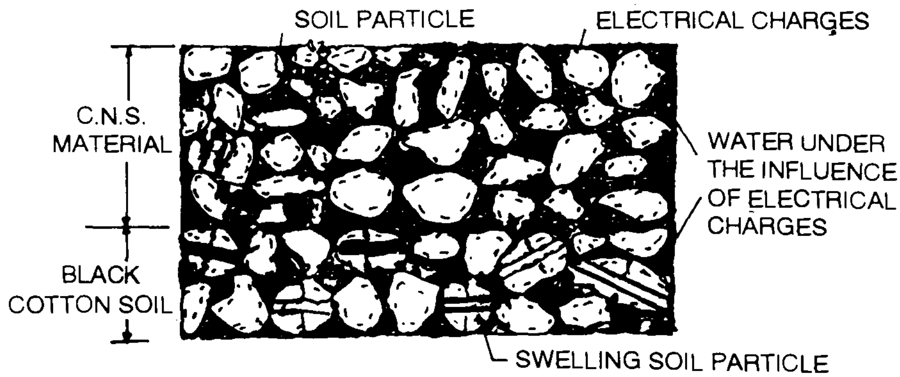
(iii) sand cushioning and

(iv) various linings with drainage arrangements.

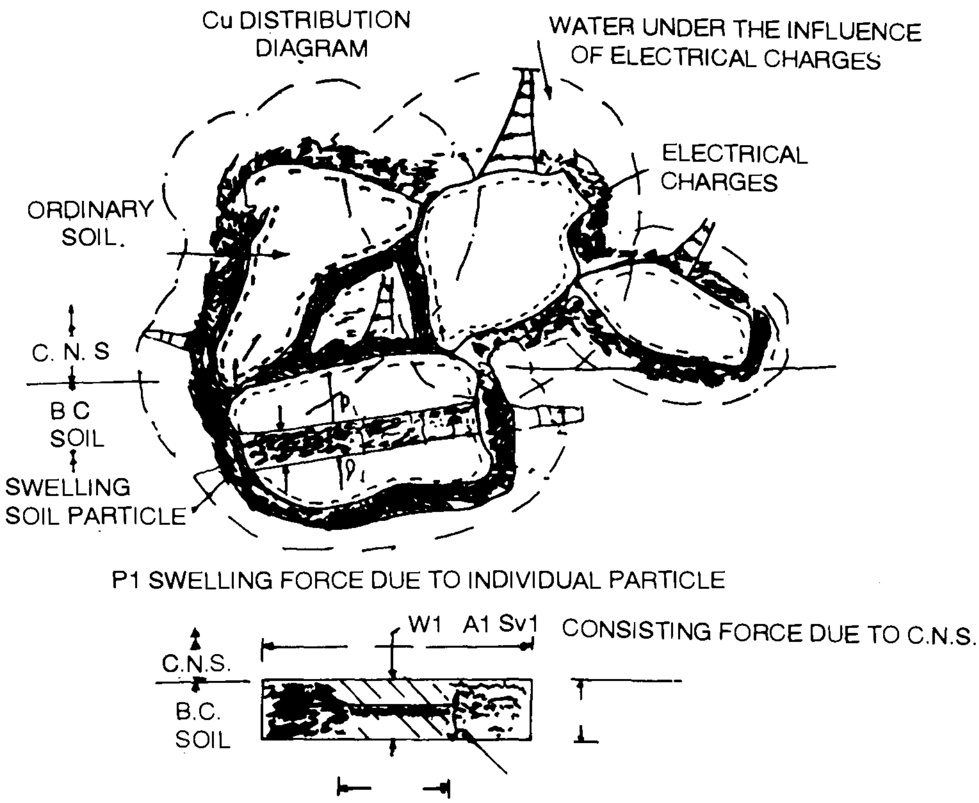
All these arrangements had shown distress as can be seen from Figure 1. A stretch using CNS layer was constructed by producing a CNS by admixing black cotton soil treated with 3 per cent commercial lime



(a) CNS layer on compacted air dry black cotton soil



(b) CNS layer on compacted black cotton soil after saturation



Qsw= SWELLING PRESSURE OF EQUIVALENT CUMULATIVE SOIL PARTICLE

P = AS Qsw2 = A2 Sva'W' A1 Sv1

(c,d) one layer or black cotton soil with volume A.d at interface

FIGURE 39 Conceptual cohesion model with CNS layer.

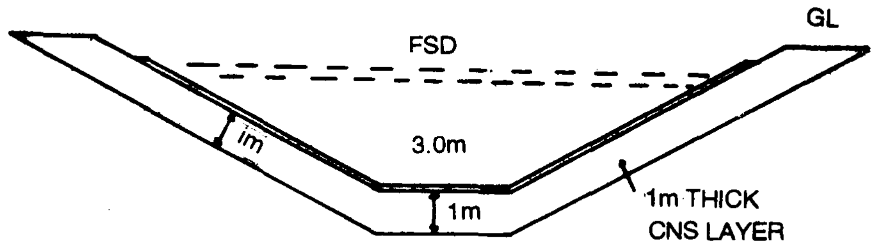


FIGURE 40 (a) Canal Section at MRBC Km 21

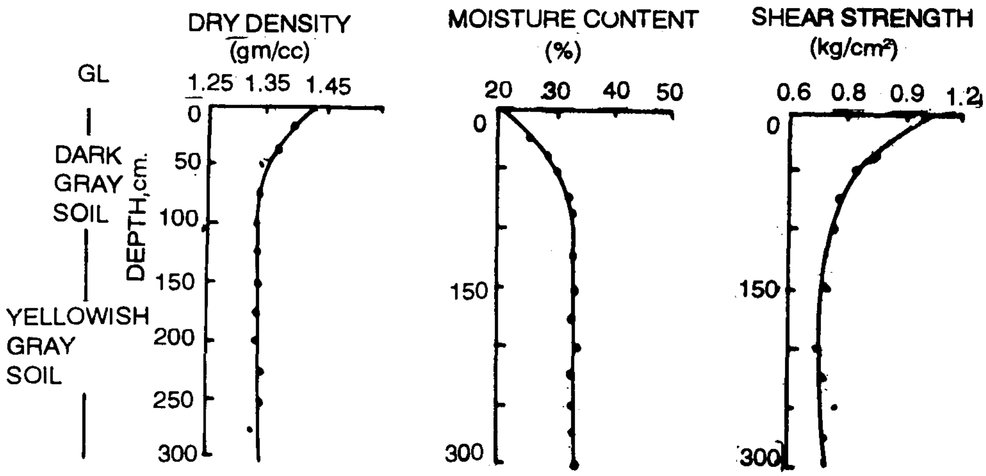


FIGURE 40(b) Variation of density, moisture content and shear strength with depth at MRBC-Km 21 in mid winter.

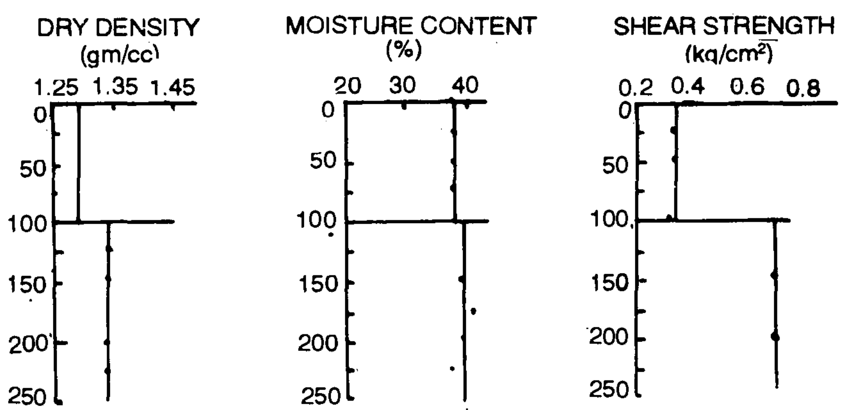


FIGURE 40(c) Variation of density, moisture content and shear strength with depth below canal bed after saturation.

and thoroughly mixed with 33 per cent murrum. The CaO content of lime was 45 per cent. The CNS layer of 1.0 m thickness was compacted in layers of 10 cm of standard Proctor density at O.M.C. The density and shear strength vs depth in black cotton soil in winter is shown in Figure 40.

After saturation for six months, the visual observation and detailed soil investigations were carried out. It was observed that there was no heave or slip on canal bed and side slopes respectively. From density vs depth curve it was observed that there was no change in the density, indicating that the effect observed in laboratory are equally valid in the field (Figure 40.).

An examination of various materials adjacent to the canal alignment indicated that in some places sand was available in plenty, in some places murrum was in plenty and in some places both the sand and murrum were to be procured from a distance.

In view of this following trials were conducted to select proper mix for CNS.

(i) 1.0 m thick CNS layer of murrum

(ii) 1.0 m thick backing material prepared by admixing 50 per cent murrum and 50 per cent sand by weight.

These materials were used in canal lining having slope of 2:1 as shown in Figure 40. The properties of CNS material procured from Betsur quarry are given in Table 6. All the backing materials were compacted in 10 cm thickness to standard proctor density at O.M.C. A 10 cm thick bed of concrete and PCC slabs on side slopes were provided.

At the end of six months of saturation period it was observed that 1.0 m thick CNS material showed satisfactory performance, whereas, in the case of 50 per cent of sand and 50 per cent murrum slight bulging at junction of bed and side was observed. In case of 33 per cent murrum and 67 per cent sand cracking of linings, bed heaving and side slipping were observed.

Since December 1972, till todate the canal section with CNS material is performing satisfactory after charging of canal. From these observations it can be seen that the murrum having 9 per cent gravel, 49 per cent sand, 30 per cent silt and 21 per cent clay fall in the category of CNS material.

It was realised that the adequate thickness of CNS layer not only resisted upward heave and transmission of swelling pressure but also helped in increasing the shear strength of the underlying black cotton soil to considerable extent. In view of this it was felt that it may be possible to use CNS approach for providing stable foundation for cross drainage structures, transmitting loads much less than the swelling pressure of soil. Secondly, the effect of increase in shear strength to the shear strength of saturated black cotton soil at no volume change condition indicated that the system may help in providing the foundation for the structures transmitting loads of higher order.

BEARING CAPACITY STUDIES

Bearing capacity studies using plate load tests were undertaken in the field at km. 35 (Katti *et al* 1978*a*), using a plate of size 60X60 cm. Proper arrangements were made for loading and recording the settlements as shown in Figure 41. The plate load tests were conducted under the following conditions:

(i) at the surface of black cotton soil having field moisture content in winter.

(ii) at the surface of 1.0 m thick CNS layer overlying black cotton soil at field moisture content immediately after compaction.

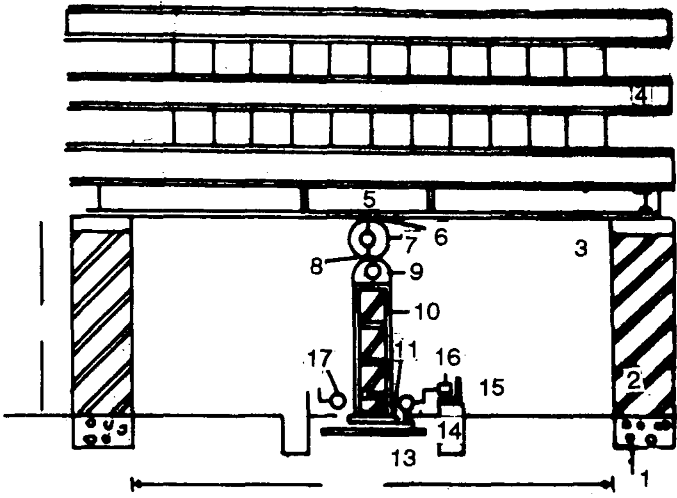
(iii) at the surface of 1.0 m thick CNS layer overlying black cotton soil after saturation.

(iv) at the interface of 1.0 m thick CNS layer and expansive soil after saturating the whole system and

(v) at 1.0 m below interface after saturation of the whole system.

The load settlement curve of these tests are given in Figure 42. The ultimate bearing capacity by tangent method for each of these cases is given in Table 7. Ultimate bearing capacity for expansive soil at field moisture content in winter was 4.7 kg/cm^2. Ultimate bearing capacity values observed were 1.7 kg/cm^2 and 0.96 kg/cm^2, immediately after compaction and after saturation. The ultimate bearing capacity after saturation at interface and 1 m below interface were 5.25 kf/gm^2 and 8.3 kg/cm^2 respectively. The influence of CNS material with respect to strength in expansive soil can be observed by comparing the ultimate bearing capacity of expansive soil in winter and at interface after saturation. The ultimate bearing capacity at interface is more than that of expansive soil in winter.

Undisturbed samples were taken to evaluate the shear strength, density etc. Terzaghi's (1943)



17 DIAL GAUGE
16 MAGNETIC BACE
15 M S ANGLE ISA 50X50 8mm THICK
14 CONCRETE BLOCK
13 SAND LAYER
12 M.S. PLATE 60X60X2.5cm.
11 M.S. PLATE 45X45X2.5cm.
10 LOADING COLUMN
9 HYDRAULIC JACK
8 M.S. BALL
7 PROVING RING
6 PLATGE FOR FIXING
5 H. FRAME
4 REACTION LOAD
3 R.C.C COPING
2 REACTION WALL
1 FOUNDATION COURSE

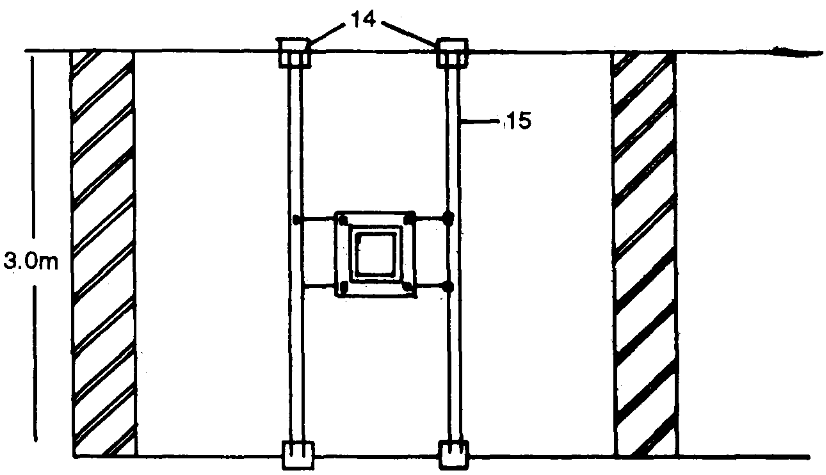


FIGURE 41 Loading arrangement for plate load tests PLT - 1,2 and 3

TABLE 7

ULTIMATE BEARING CAPACITY OF BLACK COTTON SOIL AND CNS SYSTEM

S. No	Plate Location	q_{ult} from plate load test Kg/cm²	c_u Kg/cm²	ϕ_u deg	q_{ult} from shear parameters Kg/cm²
1.	On top of expansive soil at field moisture content in winter	4.70	S_{av}= 0.83		3.13 (local failure)
2.	On top of CNS layer prior to saturation	1.70	0.45	20	1.29 (punching failure)
3.	On top of CNS layer after saturation	0.96	0.42	14	1.07 (punching failure)
4.	At interface after saturation	5.25	1.21	0	4.68(local failure)
5.	1 m below interface of CNS and expansive soil	8.30	1.80	0	7.15 (local failure)

equation for local failure condition has been used for determination of ultimate bearing capacity from shear parameters for comparision as shown in Table 7. This shows that the values observed in plate load tests are higher than the ultimate bearing capacity from Terzaghi's equation. In case of CNS layer, punching type of shear failure was observed. The ultimate bearing capacity determined using Terzaghi's equation is shown in Table 7.

An important aspect of safe bearing capacity in case of black cotton soil is that it has to satisfy the following three distinct conditions:

(i) if loads are lighter than the swelling pressure, the structures should not undergo upward movement and experience cracking etc.

(ii) if the loads are higher than swelling pressure of soil, the system should not fail under shear and

(iii) foundation load intensity should be checked with the bearing capacity in winter conditions.

The bearing capacity studies have focussed the point that the CNS process helps in design and construction of foundations when the intensity of loads are less then swelling pressure of black cotton soil and also more than swelling pressure. In view of this following equation has been suggested for evaluation of safe bearing capacity of black cotton soil as follows:

$$q_{safe} = q_{sw} + \frac{q_{ult} - q_{sw}}{FS} \qquad \text{...(19)}$$

where q_{safe} = safe bearing capacity, kg/cm²

q_{sws} = swelling pressure of blackcotton soil compacted to standard proctor density at zero moisture content at no volume change condition, kg/cm²

q_{ult} = ultimate bearing capacity, kg/cm²

FS = factor of safety

Another important point to be noted is that if CNS layer is not present on expansive soil, there is a possibility of differential heave and loss of shear strength at the edges of foundation. For heavier loads it is necessary to design the CNS layer by proper granular stabilization for upper portion.

BENNIHALLA APPROACH EMBANKMENT

As the CNS layer was found to be effective in retaining and developing high shear strength in black cotton soil, it was proposed to use it for the approach embankment of Bennihalla aqueduct (Katti *et al* 1977*b*, 1977*c*). Malaprabha right bank canal system crosses the Bennihalla which is a major nalla at km 68. The structure provided is canal trough on piers supported on well foundations. At both the flanks it has heavy embankment of the order of 14 m. The soil locally encountered is expansive in nature having swelling pressure around 1.0 to 2.5 kg/cm².

The conventional method proposed by Malaprabha authorities, for construction was using murrum procured from Byahatti quarry which is 9 km away from the site.

Based on the experience of experiments conducted with CNS layer in bearing capacity studies in

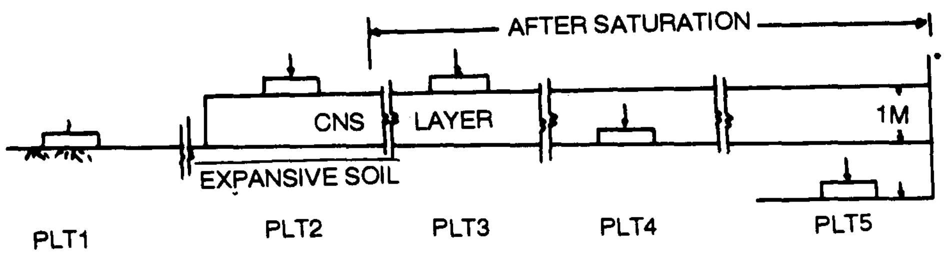
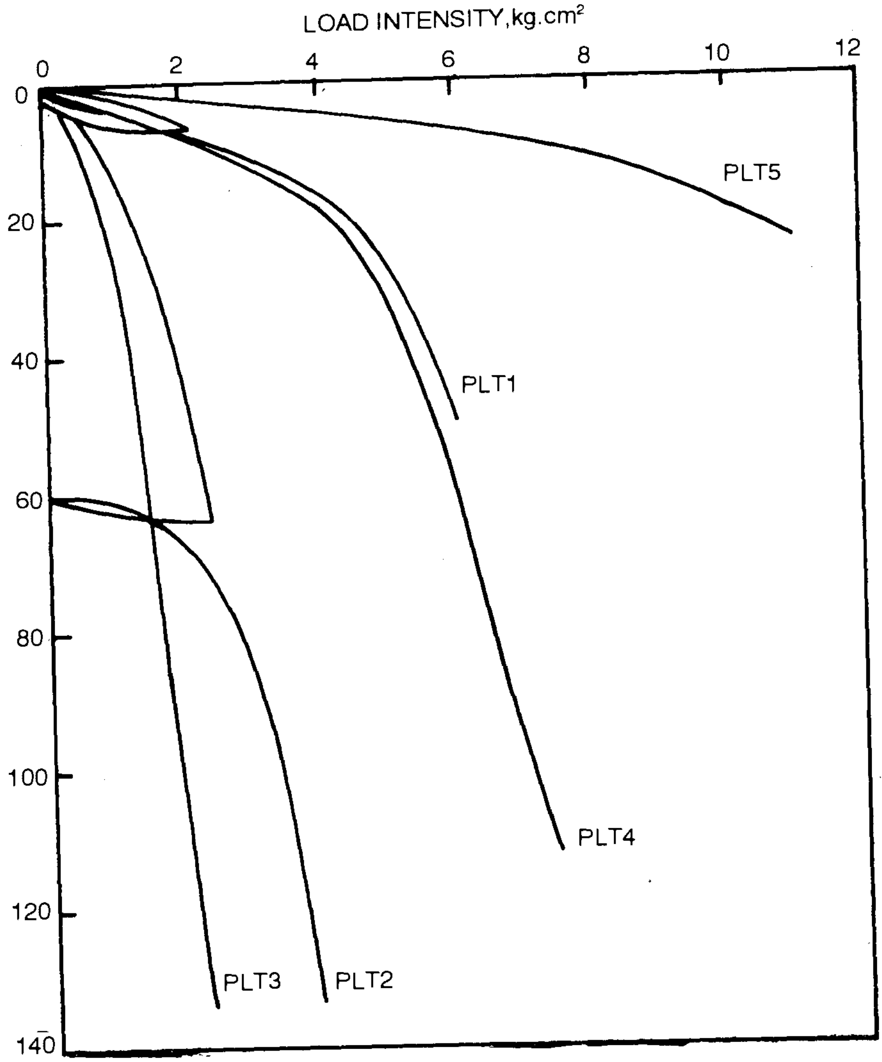


FIGURE 42(a) Test locations for load test



60m

FIGURE 42(b) Load settlement curves

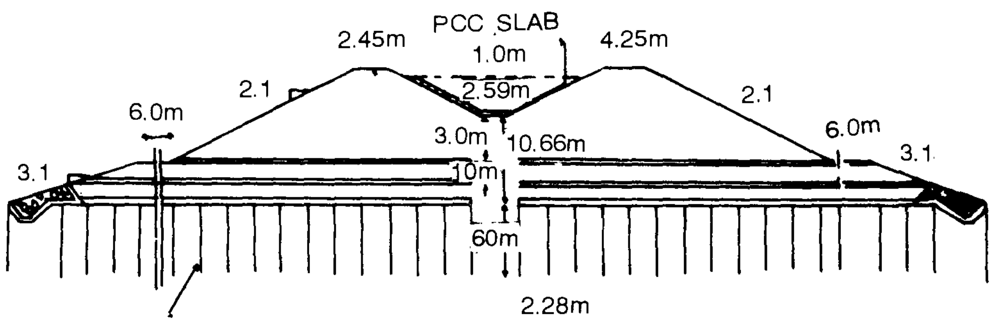


FIGURE 43(a) Bennihalla approach embankment (conventional approach)

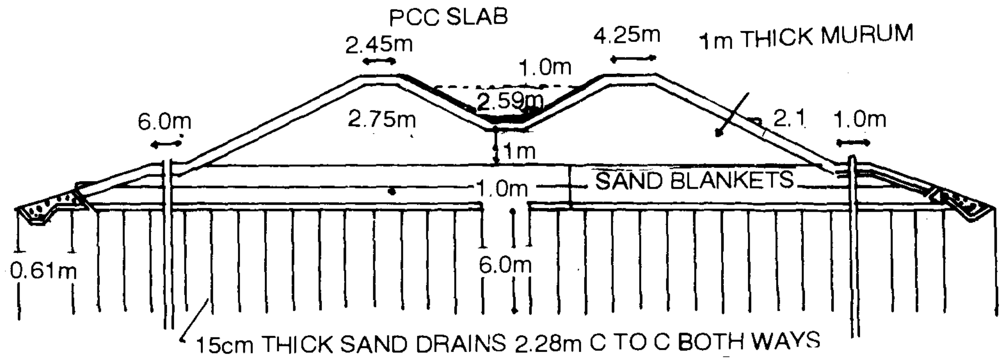


FIGURE 43(b) Bennihalla approach embankment (earlier approach)

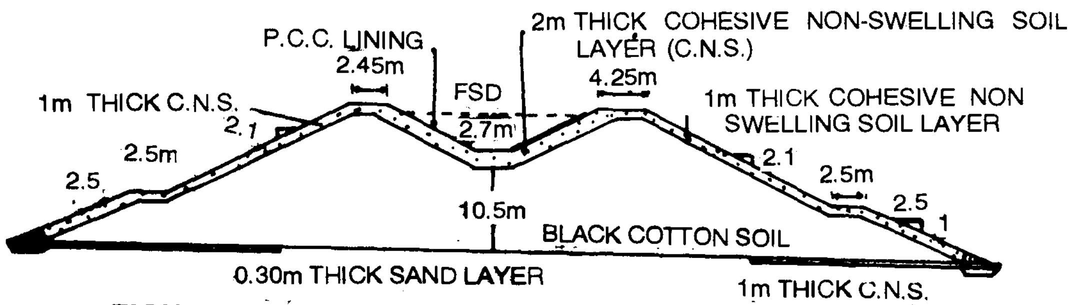


FIGURE 43(c) Bennihalla embankment newly adopted approach recommended by I.I.T. Bombay

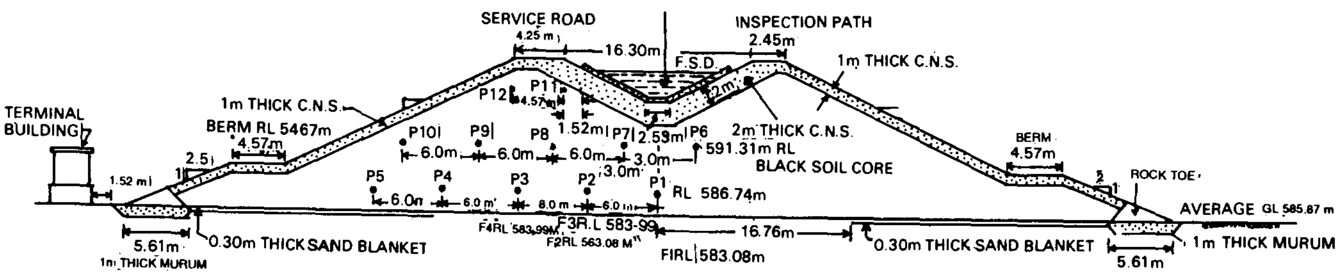


FIGURE 43(d) Proposed Instrumentation

canal it was realised that the CNS layer in canal prevented heaving of side and reduction in shear strength. The canal in the cutting where the experiments were conducted at slope of 2:1 showed stable conditions.

As a first step it was proposed to use major part of the approach embankment with black cotton soil covered by murrum on outer surface which has properties of CNS layer (Figure 43). It was necessary to examine the stability of section, using c and ϕ values of the soil. The shear parameters were evaluated from consolidated direct shear test at 0.125 mm/sec deformation rate. The c and ϕ values adopted in design were 0.45 kg/cm^2 and 8° 42' respectively. The c'_u and ϕ'_u values adopted for the CNS layer were 0.35kg/cm^2 and 29° respectively. The design was based on total stress analysis by the Swedish slip circle method. The factor of safety was 1.35. Design was checked for effective stresses also.

This embankment is a critical part of Malaprabha river vallery project. In view of this additional care was taken by providing double the thickness of the CNS layer for inner slopes. For outer slopes, however, 1 m thick CNS layer was provided. The revised section proposed is shown in Figure 43. Appropriate filter with drainage arrangement and toe drains are provided. To help the future designs a part of embankment is instrumented as shown in figure. A major portion of the construction of embankment is over.

By changing over from conventional design to CNS layer technique, it is understood that the cost of construction was reduced to Rs.47.73 lakhs, instead of Rs. 100 lakhs. This brings out a saving of Rs.52.27 lakhs in cost of construction of approach embankment to Bennihalla Aqueduct.

SPECIFICATION FOR CNS MATERIALS

The CNS material technique has been used widely in civil engineering construction activities in

FIGURE 44(a) Canal linning in distress without CNS layer backing

FIGURE 44 (b) Canal lining intact with 1 m thick CNS layerbacking

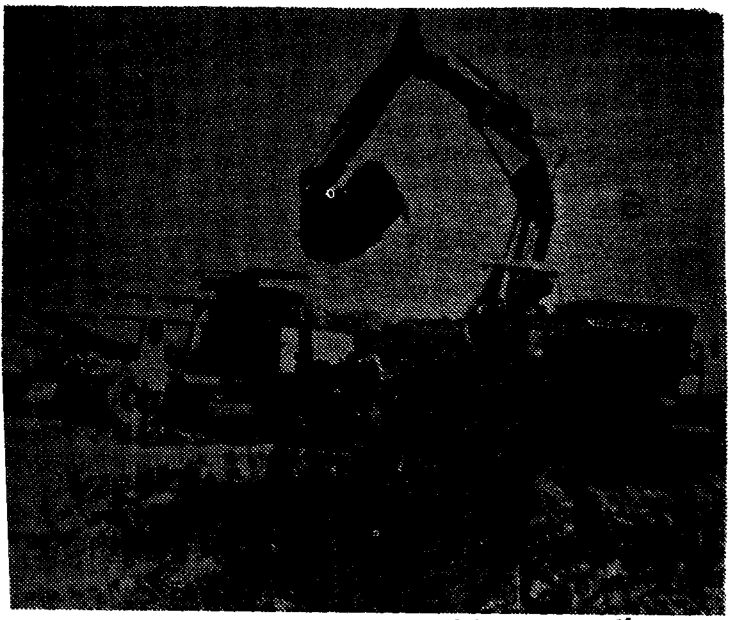


FIGURE 45(a) Póclan used for excavation

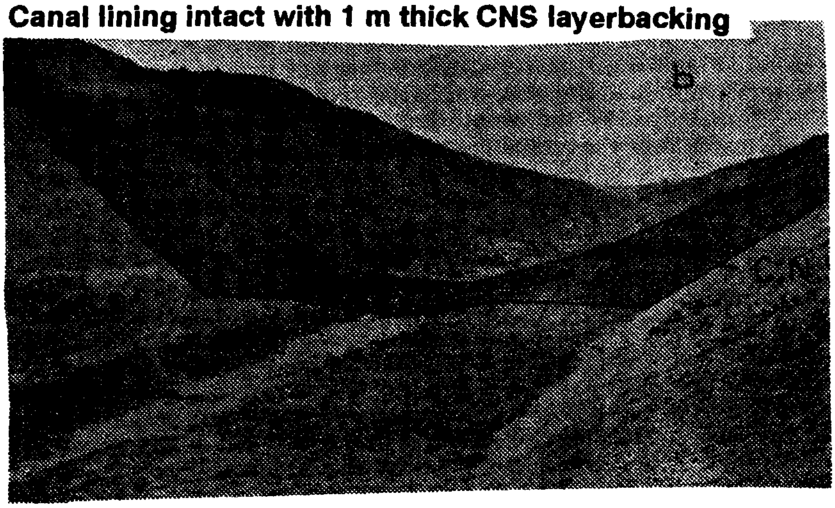


FIGURE 45(b) Canal section showing CNs layer in Km 36

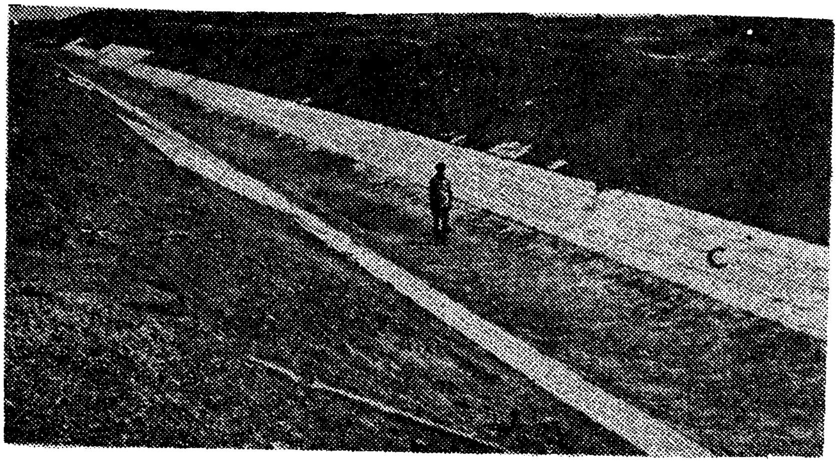
FIGURE 45 Stages pf canal of lining in expansive soil at Malaprabha Project using CNS layer backing

expansive soil region e.g. canals, cross drainage structures, roads, bridges, buildings and dams etc. In view of this certain range of specifications were evolved, based on the large scale studies conducted at Indian Institute of Technology, Bombay, and the field trials in Malaprabha Project as mentioned in Table 8 (Katti et al 1978a, 1978b).

PERFORMANCE OF STRUCTURES

On the basis of experience from large scale tests and the field trials in the Malaprabha river valley project the CNS layer technique has been used for canal linings, foundations of the cross-drainage structures and buildings etc. Since 1973 this technique has been used for the construction of canal linings. The canal is charged for irrigation purpose and has undergone 5 to 6 rainy seasons without showing any damage or distress to the lining as can be seen in the Figures 44 and 45. (Till todate, 1993 the canal and structures are performing satisfactorily)

Considering the satisfactory performance of canal linings and the field trials in Malaprabha Projects,



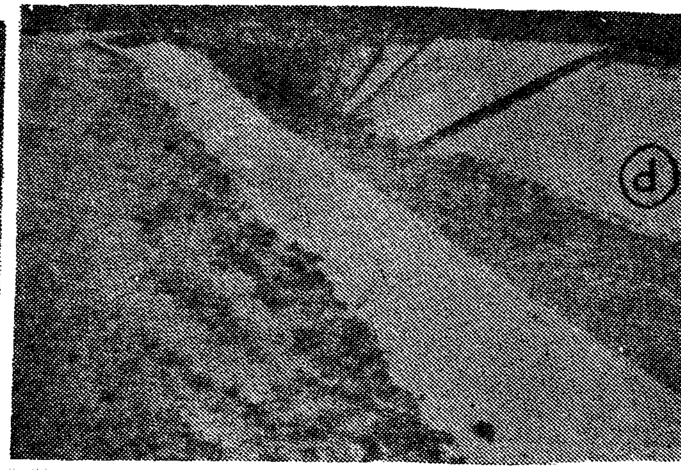
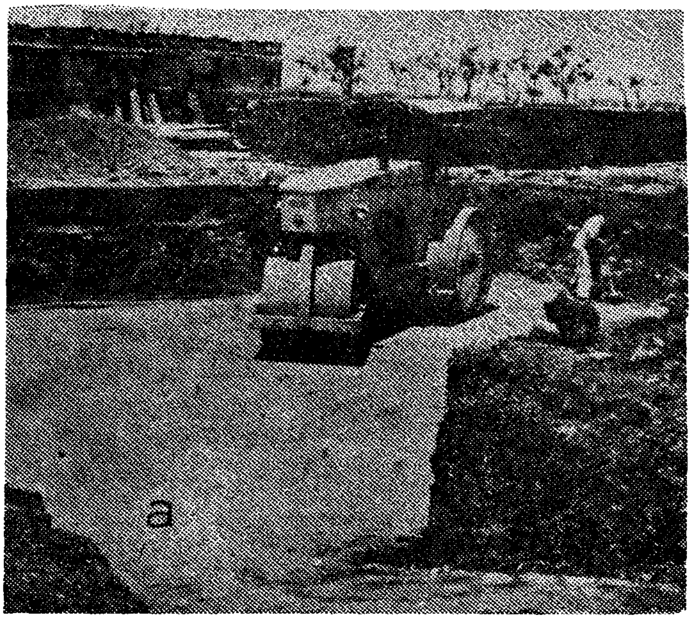


FIGURE 45 (c) PCC Slab lining in progress in Km 29 **FIGURE 45(d) Completed canal section at Km 26**

FIGURE 45 Stages pf canal of lining in expansive soil at Malaprabha Project using CNS layer backing



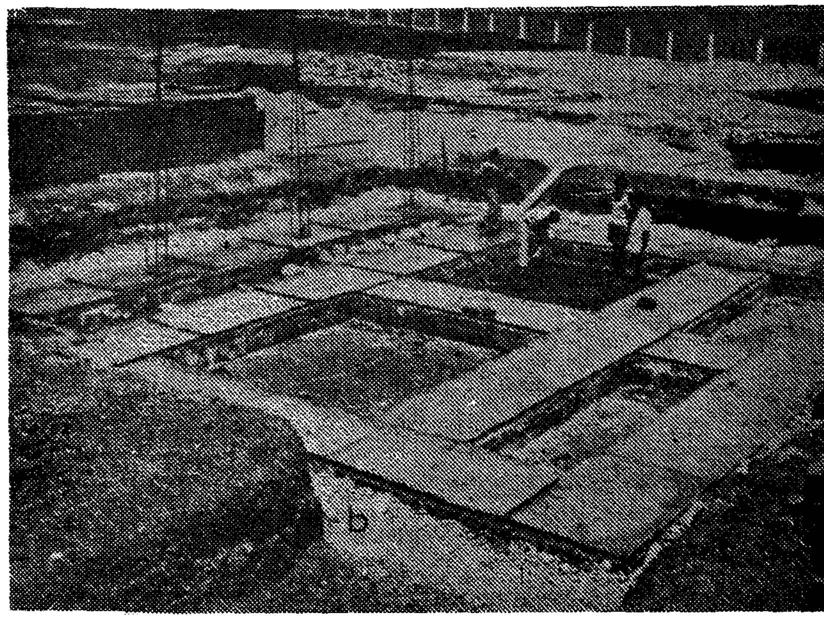


FIGURE 46(a) compaction of CNS layer footing in progress
FIGURE 46(b) Footing on CNS layer at Navalgund
FIGURE 46(c) Completed building on CNS layer at Tirlapur

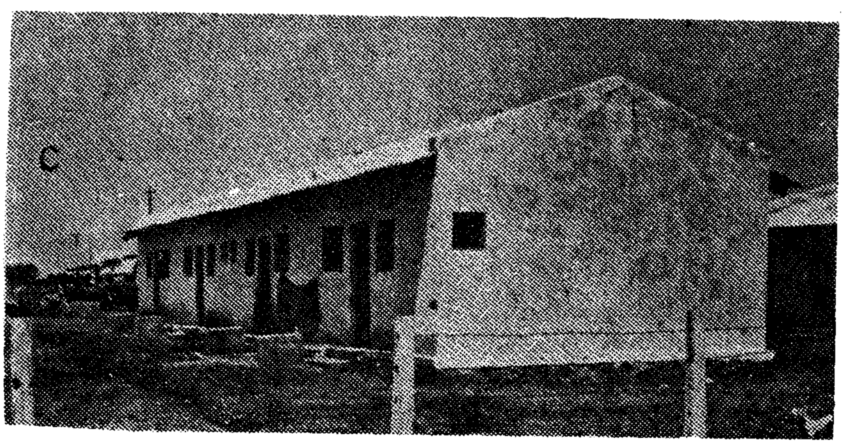


FIGURE 46 Construction of single storey building in black cotton soil areas using cohesive non-swelling soil CNS layer of 1 m thickness in foundation system

TABLE 8

TENTATIVE SPECIFICATIONS FOR SOIL TO BE CONSIDERED AS CNS MATERIAL

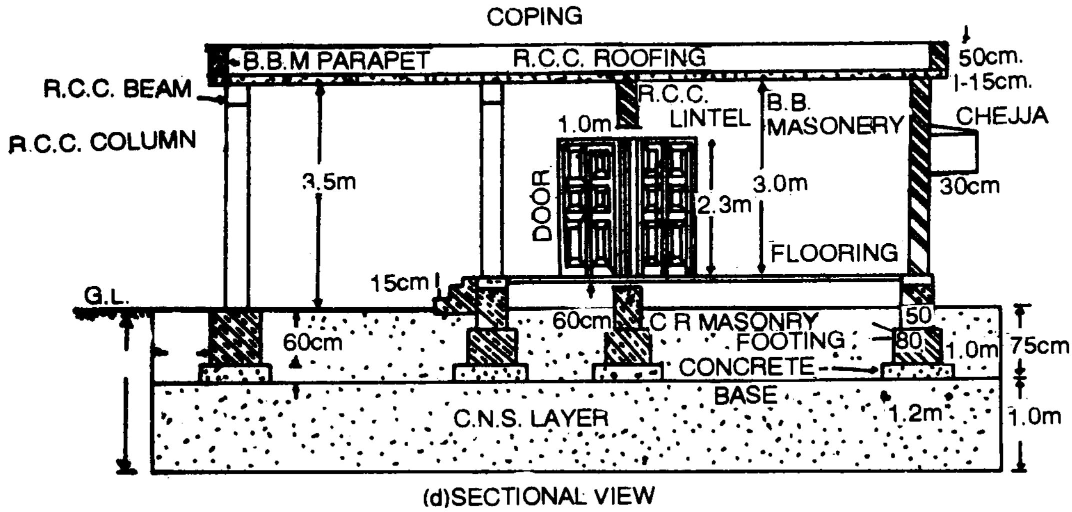
S.No.		Properties	Specifications range
1.	Grain size analysis		
		Clay (<0.002 mm) %	15-25
		Silt, (0.06 to 0.002 mm)%	30-45
		Sand, (2 to 0.06 mm)%	30-40
		Gravel (>2 mm)%	10
2.	Consistency limits		
		Liquid Limit, %	30-50
		Plastic Limit, %	20-25
		Plasticity Index,	10-25
		Shrinkage Limit,%	15 and above
3.	(a)	Swelling pressure when compacted to Standard Proctor optimum with moisture content andat no volume change condition, (kg/cm²)	Less than 0.1
	(b)	Swelling pressure when compacted to Standard Proctor optimum conditions at no volume change condition, (kg/cm²)	Less than 0.05
4.	Clay minerals		Preferably Kaolinite and Illite
5.	Shear strength of compacted samples to Standard Proctor optimums condition, after saturation		
	(a)	1/2 UCS (kg/cm²)	0.15-0.35
	(b)	consolidated direct shear test @ 0.0125 mm/min.	
		c_u (kg/cm²)	0.1-0.3
		ϕ_u (deg)	8-15
6.	Approximate thickness of CNS layer		
	swelling pressure (kg/cm²)		Thickness (cms.*)
	1 to 1.5		75-85
	2 to 3		90-100
	3.5 to 5.0		105-115

*It is necessary to conduct large scale tests to arrive at optimum thickness of CNS layer with available CNS material.

this CNS layer technique has been used for the foundation of the cross drainage structures, buildings, and for backing material of the retaining structures.

The buildings at Tirlapur camp of Malaprabha Project are not showing any cracking or distress wherein, CNS layer has been used in the foundations, Figure 46 (Katti *et al* 1978*a*). The cross drainage structures are also performing well without showing any cracks wherein CNS layer has been used in foundation and as a backing material between structures and expansive soil. (Performance is satisfactory till todate, 1993).

While using CNS layer technique in construction of above-mentioned structures in Malaprabha project, suitable construction methods were developed. (Katti *et al* 1978*b*). The method of construction is briefly described in the following.



(d) Irrigation inspection bungalow at MCC colony Navalgund

FIGURE 46 Construction of single storey building in black cotton soil areas using cohesive non-swelling soil CNS layer of 1 m thickness in foundation system.

FIGURE 47(a) Canal in cutting at Km 36

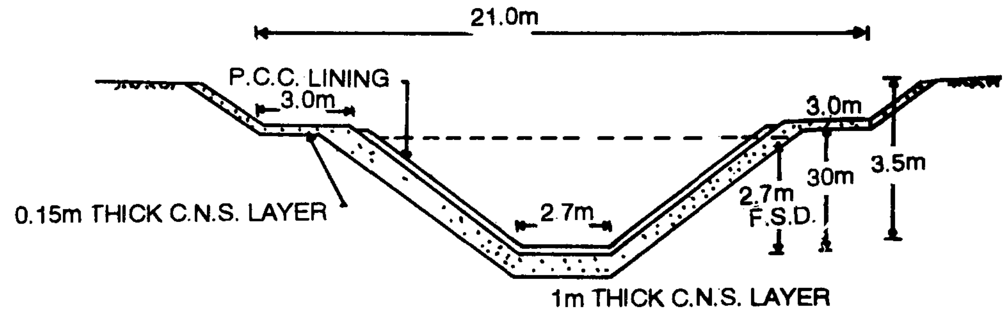
FIGURE 47 Construction of Malaprabha right bank canal in expansive soil using 1 m thick CNS layer

CONSTRUCTION PROCEDURE

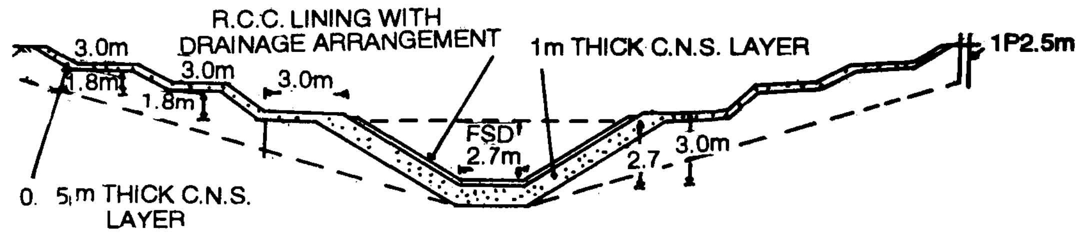
CANAL IN CUTTING

The canal section is excavated to the required depth, taking into consideration the thickness of CNS layer and the slope of canal. Sides are trimmed to get the slope of 2:1 in water prism portion and 1.5:1 in the berms. The berm width of 3 m is provided at 3 m depth interval (Figure 47).

The CNS layer of 1.0 m thickness is compacted in layer to standard Proctor density with proper optimum moisture content. The CNS layer is compacted with 8-10 ton rollers in canal bed and on slopes. Above first berm level compacted 100 cm thick CNS layer is provided for canals in deep cutting. Refer Figure 47.



TYPICAL SECTION IN CUTTING AT MRBC km. No.36



TYPICAL SECTION IN DEEP CUTTING AT MRBC. km No.36

FIGURE 47 (b) Canal section in cutting

1:2:1 PCC of 10 cm thickness is provided on bed of canal. The side slopes are trimmed to get the required thickness of CNS. PCC slabs are provided on trimmed CNS layer using lug slabs and guide slabs for porper placement

CANAL IN PARTIAL CUTTING

For canal construction in partial cutting, the excavation is done upto the required depth. The excavated section is trimmed to slope of 2:1. The CNS layer of 1.0 m thickness is compacted in layers of 20 cm to the standard Proctor density in bed as well as on sides upto ground level.*

Above the ground level the canal section is made up of black cotton soil covered by 1m thick CNS layer on inner and outer slopes. The CNS layer and black cotton soil are compacted simultaneously.

Generally the earth work is provided 1 m above the full supply depth PCC slab lining is provided with the help of lug slabs and guide slabs upto 20 cm above full supply depth. The bed concrete of 1:2:4 of 10

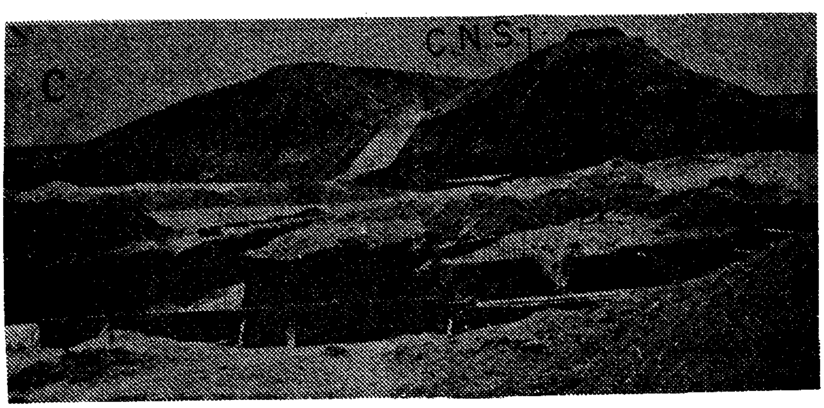
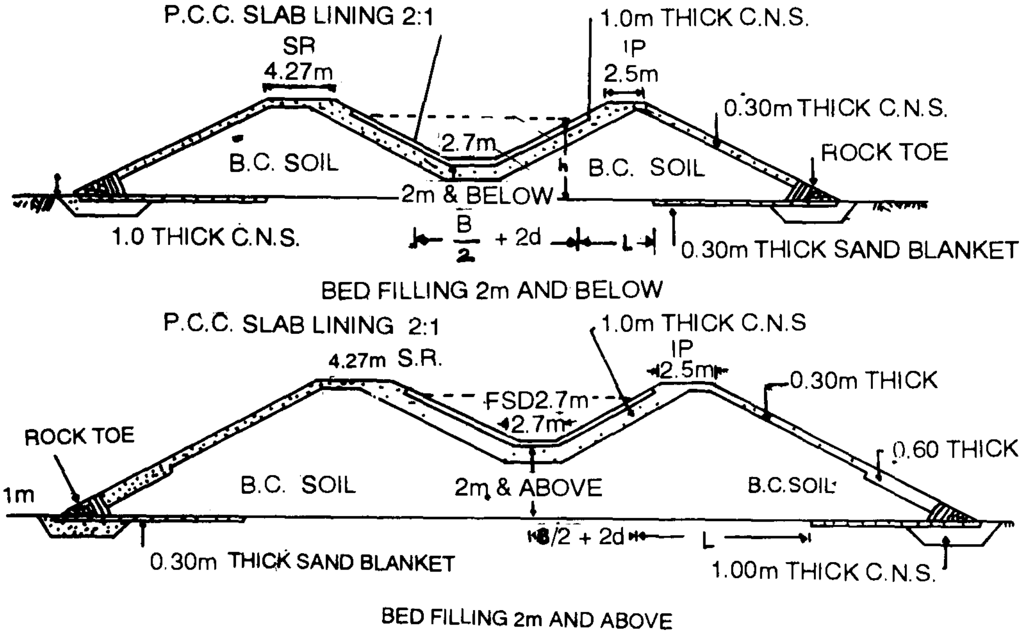


FIGURE 47 (c) Canal in embankment at Km 34

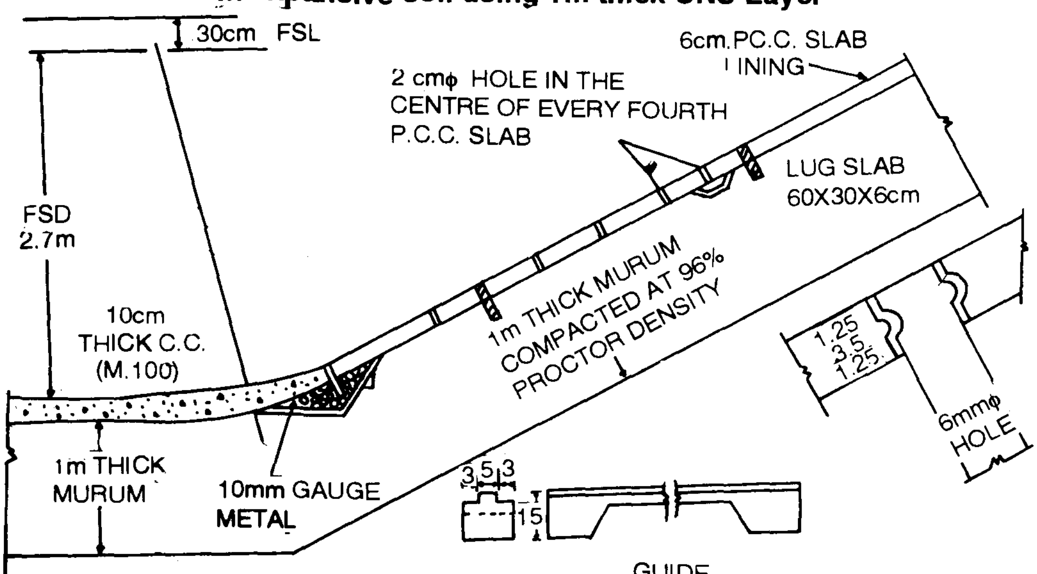
FIGURE 47 Construction of Malaprabha right bank canal in expansive soil using 1m thick CNS Layer

* In cutting it is necessary to provide CNS layer throughout the excavation, eventhough the depth of water prism may be small. If it is not provided the unprotected portion will heave and cause both surfacial and deep slides resulting into major distress to the canal. Care is to be taken to prevent contact slide between CNS layer and black cotton soil and CNS layer and lining.



(d) Canal section in embankment

FIGURE 47 Construction of Malaprabha right bank canal in expansive soil using 1m thick CNS Layer



NOTE - ALL THE DIMENSIONS ARE IN Cm.

FIGURE 48 Canal lining details

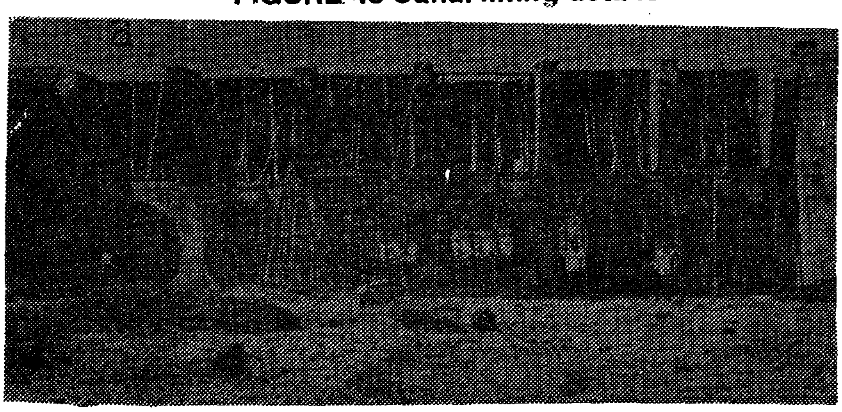


FIGURE 49(a) Mass concrete pipe culvert

Figure 49 Construction of cross drainage structure in expansive soil for Malaprabha project using CNS layer

cm thickness is provided in bed canal (Figure 48).

Rock toe is provided when the bed filling exceeds 2 m.

CANAL IN EMBANKEMENT

The bank work of expansive soil covered with CNS layer of appropriate thickness is raised simultaneously. The compaction is done with 8-10 ton roller to standard Proctor density with optimum moisture content. The rock toe with inverted filter arrangement is provided as shown in Figure 47.

The bed of canal is trimmed to the required bed width and slopes. The concrete of 1:2:4 of 10 cm thickness is laid as a trough in bed of canal. The slopes are trimmed to slope of 2:1 and PCC lining is provided 30 cm above F.S.D. Outer slopes are trimmed to the required slope.

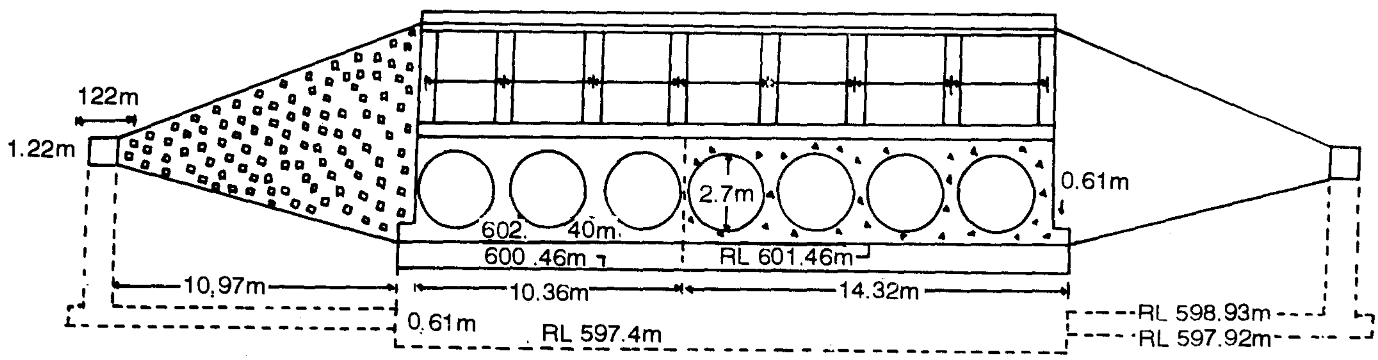
Berms are provided on outer slopes after checking slopes with the help of proper stability analysis.

CROSS DRAINAGE STRUCTURES

The CNS layer technique is used for foundation of the cross drainage structures i.e., mass concrete pipe culvert, pipe culvert, superpassage, road bridge, cut and cover conduit etc. Salient features of construction procedure are mentioned below.

In all these structures, the construction sequence consists of excavating the expansive soil for 1m below the footing level. CNS layer of required thickness is compacted to modified AASHO with proper moisture content. CNS layer is provided for 50 cm beyond the footing dimensions. For the compaction of CNS layer sheep foot rollers or 8-10 ton rollers are used. Properties of CNS layer should be checked with specifications. Above the CNS layer a levelling course of concrete is placed prior to raising the rest of structure.

In wing walls of the structures, to resist the swelling pressure transmitted by expansive soil, 1.0 m thick CNS layer is used. The CNS layer is compacted to modified AASHO by pneumatic tampers, with



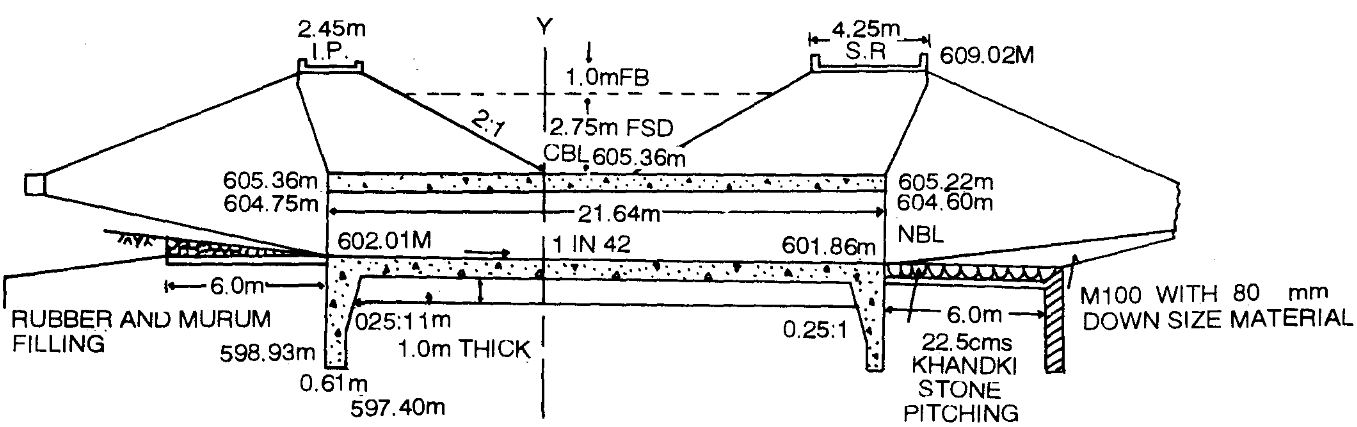


FIGURE 49(b) Sectional view

Figure 49 Construction of cross drainage structure in expansive soil for Malaprabha project using CNS layer

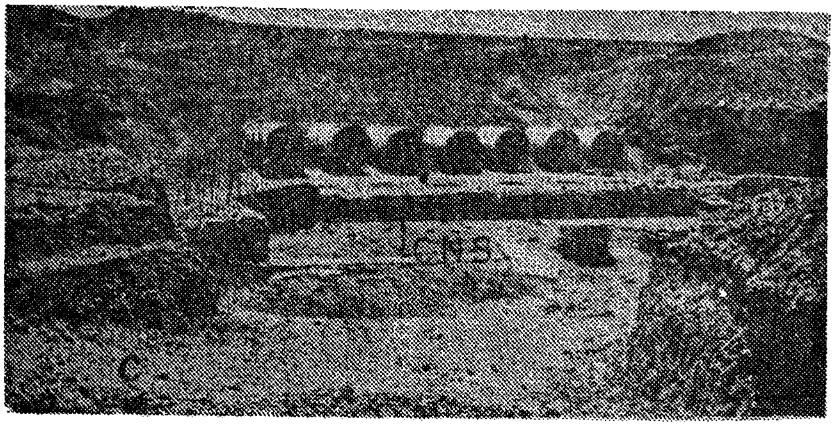


FIGURE 49(c) Pipe culvert at Km 29

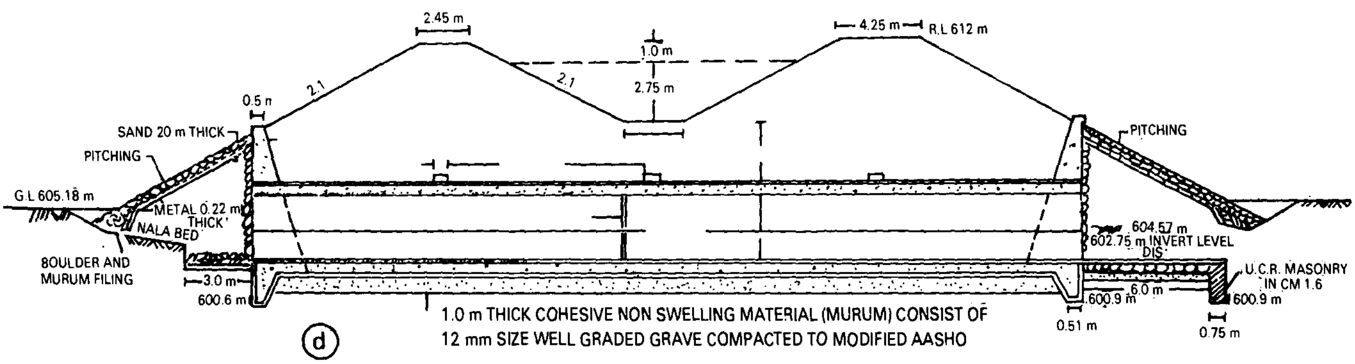


FIGURE 49 (d) Cross drainage structures for Malaprabha project— Sectional View

Figure 49 Construction of cross drainage structure in expansive soil for Malaprabha project using CNS layer

proper moisture content. Figures 49, 50 and 51 show cross drainage structures.

BUILDINGS

The expansive soil is excavated 1 m below footing level. The CNS layer of required thickness (1m) is compacted to modified AASHO with the help of 8-10 ton rollers. The CNS layer is provided 1 m beyond the footing dimensions (Figure 46). For the sequence of the construction procedure Katti et al 1978a, Katti et al 1977b and Katti et al 1978c may be referred to. Foundation course of PCC is placed on CNS layer. The building shall be constructed on the foundation course. A CNS layer of 1.0 m may be compacted on side of the foundation course up to ground level.

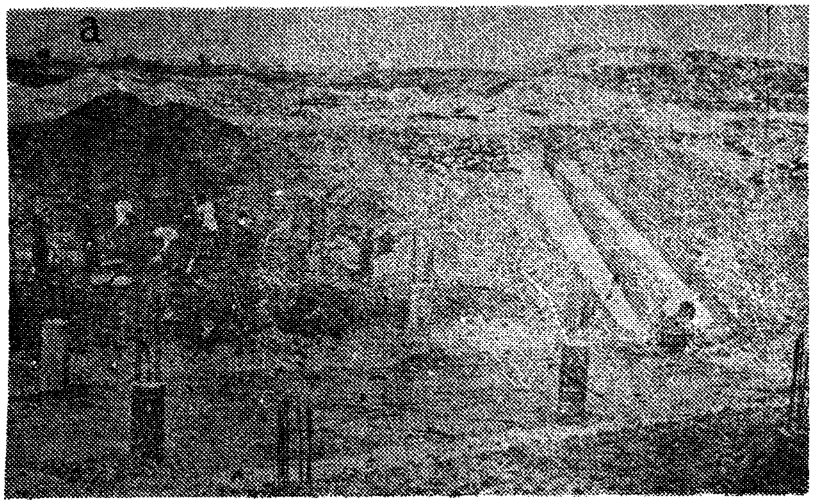


FIGURE 50(a) Super passage at Km 41

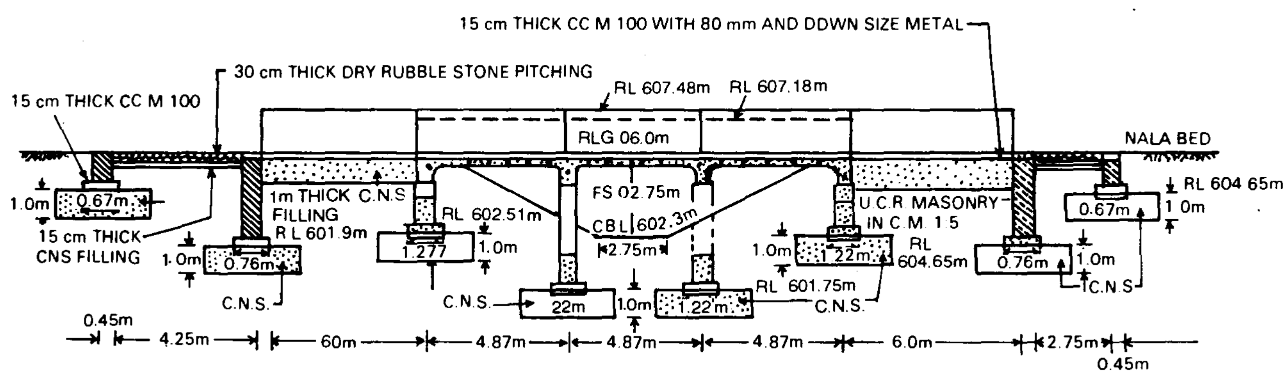


FIGURE 50 *(b)* Sectional view of super passage at Km 36

FIGURE 50 Structure across canal in expansive soil for Malaprabha Project using CNS layer

DAMS

The CNS layer technique has been used in construction of dams. The hearting material used is expansive soil whereas the casting material used is similar to CNS soil. This technique is used in Almatti earthen dam and Narayanpur right bank earthen dam of Upper Krishna project (Katti 1978, Katti et al 1978d). Similar technique is adopted in some of the irrigation projects in Maharashtra (Katti et al 1978b).

STABILIZATION OF BLACK COTTON SOIL

The studies on various soil fractions of black cotton soils indicated that the swelling and swelling pressure characteristics of the soil are due to the presence of montmorillonite and combination of

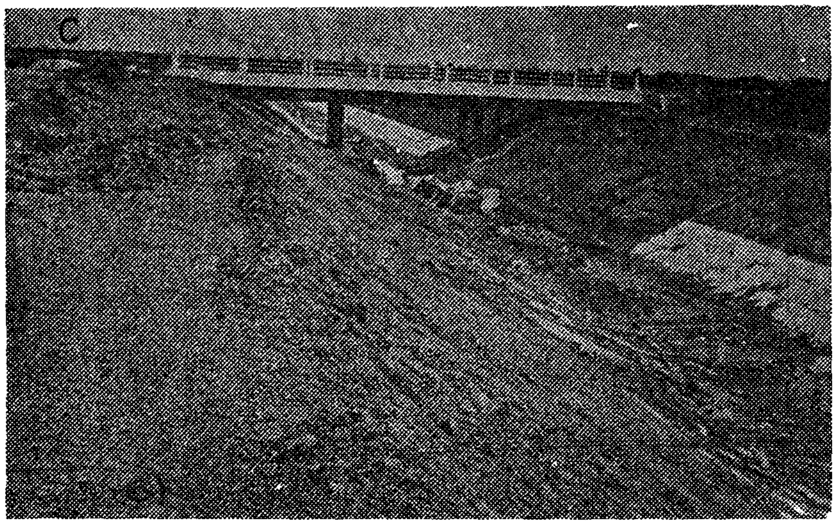


FIGURE 50(c) Road bridge at Km 32

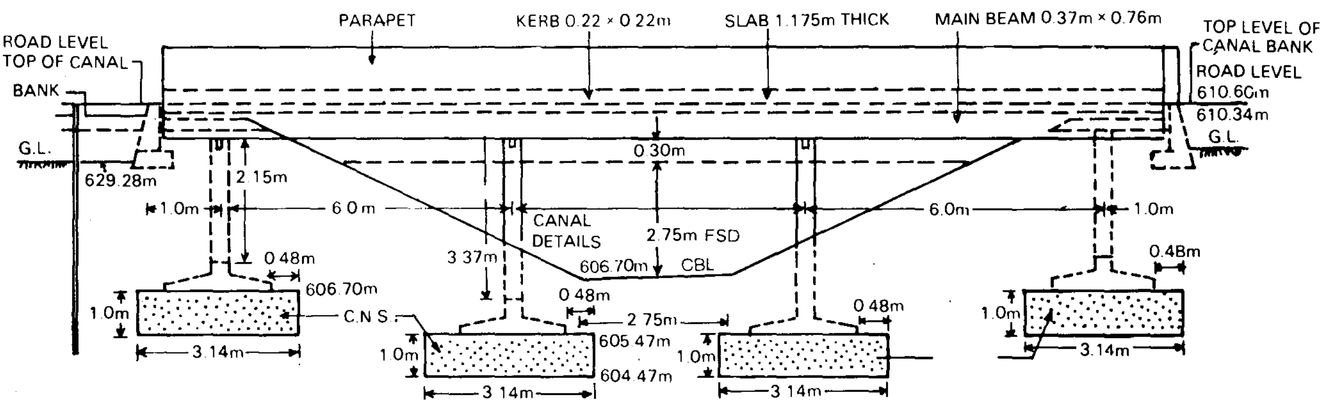


FIGURE 50*d* Sectional view of road bridge Km 32

FIGURE 50 Structure across canal in expansive soil for Malaprabha Project using CNS layer

montmorillonite and Illite type of clay mineral in clay fraction. This in turn is attributed to electrical charge imbalance in the structures and also the cation exchange capacity. Keeping this aspect in view, an attempt was made to see the alterations brought out by various inorganic chemicals, shown in Table 9, on index properties of black cotton soils. (Katti and Gupta 1970, Katti et al 1973, Katti and Kakroo 1969, Katti and Rao 1963, Katti and Kulkarni 1962, Kulkarni et al 1967, Radhakrishnan et al 1967).

These studies indicated that some of the chemicals such as hydroxides of Na, K, Ca, KCl, $MgCO_3$ and phospates of Ca, Mg etc., are quite effective in altering the index properties and changing the texture (Katti and Barve 1962, Katti and Kulkarni 1962, Katti *et al* 1966, Tyagi and Katti 1973).

It was also observed that KOH, NaOH, $CaCl_2$, KCl, NaCl, K_2CO_2 increased the permeability values, whereas, less soluble compounds like $Ca(OH)_2$ decreased the permeability (Katti and Bhandari 1966, Katti and Parikh 1961).

Studies were also conducted to improve the strength characteristics of black cotton soils using lime, cement and combinations of lime and cement. (Katti and Gupta 1970, Katti and Kulkarni 1962, Katti and Sridhar 1962). It was revealed that lime alone or cement alone is not adequate in obtaining required seven days strength. However, 4 per cent lime and 4 per cent cement in case of Poona black cotton soil yielded

TABLE 9(a)

CONSISTENCY LIMITS FOR BLACK COTTON SOIL TREATED WITH VARIOUS CHEMICALS

	Barium chloride				Barium carbonate				Portland cement			
Chemicals %	LL %	PL %	PI %	SL %	LL %	PL %	PI %	SL %	LL %	PL %	PI %	SL %
0.00	67.2	48.9	18.3	8.2	67.2	48.9	18.3	8.2	67.2	48.9	18.3	8.2
0.10	70.8	41.3	29.5	10.3	78.4	46.1	32.4	19.5	70.8	44.2	26.5	9.1
0.25	71.0	42.4	28.6	9.3	70.5	47.7	22.8	15.2	62.0	44.0	18.0	12.6
0.50	69.5	43.3	26.2	9.0	70.6	47.8	22.8	13.3	68.0	40.7	28.3	12.8
0.75	70.0	44.2	25.8	11.7	77.5	47.6	29.9	16.4	73.0	44.3	28.7	8.6
1.00	69.0	43.5	25.5	9.4	76.2	49.2	27.0	11.6	70.0	43.5	26.5	11.0
1.50	69.0	40.1	28.9	10.5	70.5	50.8	19.7	11.8	69.5	40.7	28.8	14.1
2.00	62.8	40.9	21.9	10.1	69.5	43.9	25.6	10.1	69.2	44.4	24.8	14.3
3.00	67.0	39.8	27.2	10.1	72.0	42.6	29.4	10.8	66.0	43.5	22.5	21.7
5.00	61.0	38.7	22.3	9.8	74.3	42.8	31.5	8.5	68.0	43.7	24.3	23.0
7.00	59.3	38.8	20.5	9.4	70.8	43.0	27.8	10.3	65.0	44.2	20.8	27.1
10.00	55.8	39.2	16.6	14.2	66.0	43.7	22.3	13.8	65.0	45.6	19.4	37.4

TABLE 9(b)

CONSISTENCY LIMITS FOR BLACK COTTON SOIL TREATED WITH VARIOUS CHEMICALS

	Magnesium phosphate				Magnesium chloride				Megnesium carbonate			
Chemicals %	LL %	PL %	PI %	SL %	LL %	PL %	PI %	SL %	LL %	PL %	PI %	SL %
0.00	67.2	48.9	18.3	8.2	67.2	48.9	18.3	8.2	67.2	48.9	18.3	8.2
0.10	70.3	43.9	26.4	17.2	62.5	42.9	19.6	11.1	66.7	40.7	20.0	12.7
0.25	69.5	43.8	25.7	14.6	67.5	42.9	24.6	10.1	63.8	43.9	19.9	17.0
0.50	71.9	44.7	27.2	13.4	65.7	42.7	23.0	1.9	66.0	40.8	25.2	15.5
0.75	72.9	45.3	27.6	17.8	70.0	40.0	30.0	9.9	69.9	42.1	26.9	12.2
1.00	71.9	44.5	27.4	16.2	66.8	36.9	29.9	8.9	70.6	42.7	27.9	14.6
1.50	65.2	43.3	21.9	15.4	65.5	35.9	29.6	11.9	68.5	44.1	24.4	14.8
2.00	70.8	43.8	27.0	12.1	69.0	37.3	31.7	10.3	69.5	45.2	24.3	11.9
3.00	68.8	43.2	25.6	15.2	67.5	36.9	29.6	11.2	72.0	44.5	27.5	10.9
5.00	68.0	43.6	24.4	15.4	59.8	38.2	21.6	9.3	76.5	47.8	28.7	15.5
7.00	66.0	44.6	21.4	14.3	58.6	38.5	20.1	10.8	78.0	48.9	29.1	28.4
10.00	66.0	44.7	21.3	12.8	54.5	32.3	22.2	12.5	82.0	49.1	32.9	24.5

required strength in seven days. Lime alters the clay size to silt or sand size; cement helps in providing proper bonding in stabilized mix. In addition, it was also found that by properly adjusting lime cement content it would be possible to design mixes of high strength (Figure 52) (Katti and Kulkarni 1962).

These studies indicate that black cotton soils do respond to stabilization and undergo change in swelling and swelling pressure characteristics.

It may be possible to suppress the swelling and swelling pressure of the black cotton soil by adding controlled amounts of inorganic chemicals.

While preparing CNS material by this method care should be taken to see that the material does not attain rigidity.

TABLE 9 (c)

CONSISTENCY LIMITS FOR BLACK COTTON SOIL TREATED WITH VARIOUS CHEMICALS

	Calcium hydroxide				Calcium chloride				Calcium Phosphate			
Chemicals	LL	PL	PI	SL	LL	PL	PI	SL	LL	PL	PI	SL
%	%	%	%	%	%	%	%	%	%	%	%	%
0.0	67.2	48.9	18.3	8.2	67.2	48.9	18.3	8.2	67.2	48.9	18.3	8.2
0.1	71.2	40.1	30.0	10.3	70.0	41.0	29.1	11.6	73.5	42.1	31.4	13.1
0.25	72.2	42.2	30.0	13.5	69.9	40.7	29.2	15.0	73.8	43.1	28.7	10.9
0.5	72.9	41.0	31.9	13.1	70.0	36.6	33.4	8.6	73.8	47.0	26.8	10.8
0.75	68.0	45.1	22.9	17.4	66.5	36.5	30.0	14.0	69.5	44.6	24.9	11.2
1.0	67.0	43.7	23.3	16.8	65.2	48.8	26.4	8.0	72.5	43.1	29.4	13.8
1.5	68.0	50.0	18.0	12.1	65.4	39.1	26.3	11.4	64.8	39.4	25.4	8.2
2.0	66.0	-	-	11.1	69.0	38.5	30.5	8.1	65.2	39.8	25.4	9.2
3.0	64.5	-	-	16.3	67.9	36.1	31.8	8.1	68.5	44.6	22.9	14.7
5.0	68.0	-	-	31.6	64.0	38.0	26.0	6.5	72.0	45.5	26.5	14.4
7.0	68.5	-	-	34.0	58.0	42.0	16.0	6.5	70.5	46.8	23.7	14.5
10.0	68.0	-	-	44.5	56.3	39.0	17.3	10.4	72.5	46.4	26.1	15.2

TABLE 9(d)

CONSISTENCY LIMITS FOR BLACK COTTON SOIL TREATED WITH VARIOUS CHEMICALS

	Potassium chloride				Potassium dichromate				Potassium hydroxide			
Chemicals	LL	PL	PI	SL	LL	PL	PI	SL	LL	PL	PI	SL
%	%	%	%	%	%	%	%	%	%	%	%	%
0.0	67.2	48.9	18.3	8.2	67.2	48.9	18.3	8.2	67.2	48.9	18.3	8.2
0.1	68.5	42.2	26.3	12.3	66.2	40.6	25.6	10.4	68.8	47.3	21.5	9.8
0.25	68.0	43.0	25.0	13.9	67.5	48.0	24.5	12.6	69.8	45.5	24.3	10.4
0.5	67.0	42.1	24.9	12.0	68.0	42.3	25.7	10.1	70.0	44.8	25.2	15.2
0.75	67.2	40.9	26.3	13.2	70.5	41.0	29.9	9.8	71.0	45.7	25.3	7.4
1.0	66.0	40.8	25.2	15.5	67.0	40.8	26.2	9.5	66.5	44.2	22.3	10.7
1.5	63.5	40.8	22.7	16.0	64.8	39.4	25.4	8.2	67.2	45.8	21.4	9.7
2.0	63.5	39.9	23.6	18.3	65.2	39.8	25.4	9.2	63.8	TNP	TNP	14.9
3.0	58.0	41.4	16.6	20.9	64.5	39.0	25.5	12.9	64.7	TNP	TNP	27.7
5.0	52.5	41.8	10.7	24.4	56.5	41.2	15.3	11.9	TNP	TNP	TNP	37.9
7.0	48.0	38.4	9.6	26.4	61.8	42.0	19.8	14.9	TNP	TNP	TNP	30.5
10.0	43.5	32.6	10.9	37.5	52.5	40.1	12.4	17.7	TNP	TNP	TNP	36.4

SUGGESTED APPROACH TO ROAD CONSTRUCTION

Problems faced in road construction are due to the swelling and swelling pressure of the black cotton soils. If the cohesive nonswelling soils is effective in counteracting both of these aspects roads can be constructed on black cotton soils by providing around 1 m of CNS layer if the swelling pressures of the soil is in the range of 2 to 4 kg/cm^2.

A sketch shown in Figure 53 shows the CNS layer compacted to standard proctor density. The base course, sub base and surfacing can be designed by considering the CBR of CNS material.

The stabilization studies have shown that it is possible to improve the strength characteristics of black cotton soil by addition of lime and cement. Such mixes can be used for base courses and sub base course. it is necessary to provide surface course commensurate with the nature of traffic and the wheel load

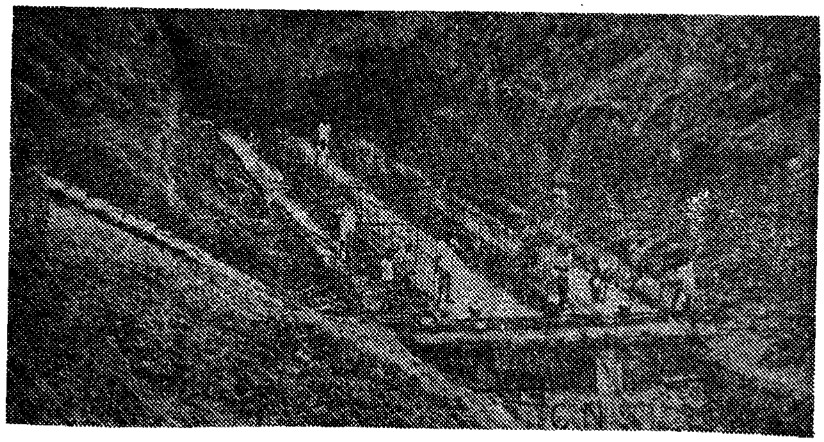


FIG 51(a) Cut and cover conduit at Km 36

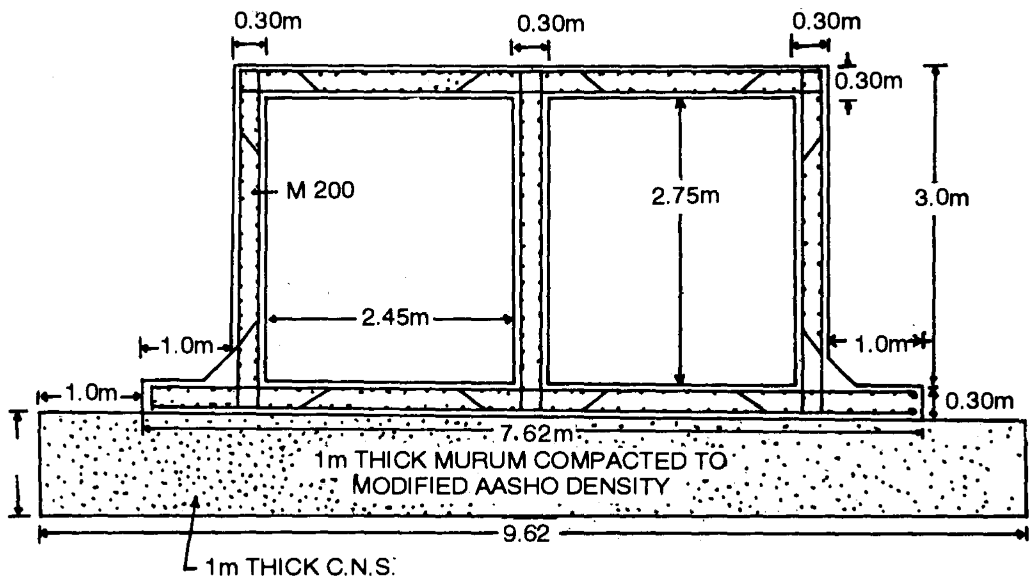


FIGURE 51(b) Sectional view of cut and cover at Km 36

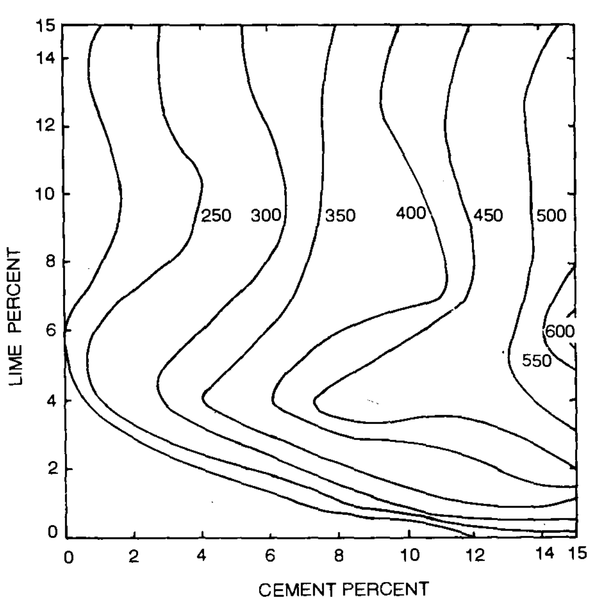


FIGURE 52 7 day iso-strength curves for Poona black cotton soil

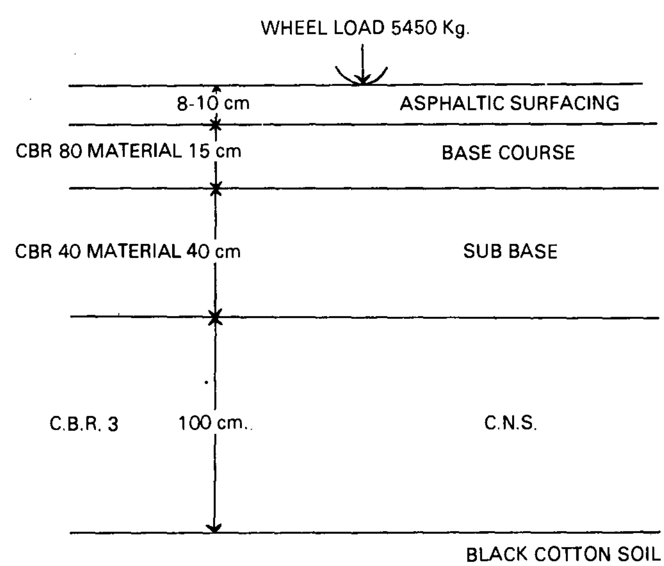


FIGURE 53 Section of road in black cotton soil.

transmitted to the surface.

SUMMARY AND CONCLUSIONS

These studies have shown

(1) The swelling pressure in a bloack cotton soil is due to the presence of montmorillonite type of clay mineral, which is having expanding type of lattice structure,

(2) The influence of the electrical charges in the clay fraction in particular and on the soil particles in general play an important role in development of adsorbed water film around the clay particles on one hand and development of cohesive forces in black cotton soil between particles to particle on the other.

(3) The water having dipolar nature trying to enter the expanding lattice structure seems to be responsible for the development of swelling and swelling pressure. Whereas the water film forming around the particle may be responsible for counter balancing the swell and swelling pressure action depending upon the rate at which the films are formed and the water is absorbed in the clay mineral.

(4) The interaction between the dipolar water and the electrical charges on clay mineral seems to be responsible for the development of cohesion of various magnitude in black cotton soil system. Because of this it appears that 1 m thickness of CNS produced an environment equivalent to 18 m height of soil at the level of 1 m in black cotton soils. This effect seems to be responsible for distinctly different lateral pressure development and shear strength development etc. with depth in black cotton soil.

(5) The concept of CNS layer is based on principle that CNS material of 1 m thickness provides an environment similar to that observed at 1 m below to develop adsorbed water film. At the same time the CNS layer itself does not heave and produces swelling and swelling pressure. The CNS Process which is found to be effective in counteracting swelling pressure and also in retaining high shear strength in black cotton soil seems to be a viable construction method for constructing stable canal, embankments, foundations of cross drainage structure and protection of earth retaining structures from high lateral pressures, construction of buildings foundation, roads etc.

(6) Based on experience gained at Malaprabha Canal system the specifications have been given for identification and preparation of CNS layer. Method of construction of various civil engineering structures in black-cotton soil region using CNS layer is also described.

(7) The studies with addition of various inorganic chemicals to black cotton soils have indicated that it is possible to suppress swelling and swelling pressure of black cotton soils. It is also observed that black cotton soil do respond to stabilization by lime and cement combination and considerable strength gain is attained.

The studies have brought out the distinct difference in behaviour of black cotton soils due to the presence of montmorillonite type of clay mineral in the soil system. It is realized that this may help in giving guidance for more detailed work on such a type of system.

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