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BLACK COTTON SOILS—HIGHLY EXPANSIVE CLAYS OF INDIA

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INTRODUCTION

INDIA has large tracts of expansive soils known as black cotton soils. This name derives from their black colour and cotton being grown in many regions on these soils. The major area of their occurrence is Central India, South of Vindhyachal range, covering an area of about 0.8 million sq. km., thus forming about 20 per cent of the total land area of the country.[1]

The Indian black cotton soils are generally heavy clays containing predominance of clay mineral montmorillonite, exhibiting high shrinkage and swelling characteristic. During shrinkage due to drying there is a formation of hexagonal columnar structure with vertical cracks upto 8cm wide at the ground level, extending upto 2.5m depth.[2] Shrinkage in horizontal direction is nearly two third of the total volumetric shrinkage.[3] Similar soils occur in other countries also e.g. 'Cheronozems' of U.S.S.R., 'Badole' of Japan, 'Pampus of Argentina, 'Tirs' of Morocco, 'Margilatic' of Indonesia and black earths of Australia, Java & Sumatra.[4] Vast areas near Irbid (Jordan) are also full of expansive clays. Geologically their formation is associated with basalts but their occurrence on granite, shale, sandstone and slate is also recognised. They occur both as residual and transported. In the former case, where the parent rock lies underneath, their depth is shallow, averaging about 1m, but in low lying and flat areas when they develop over alluvium, after being transported by the wind and water, they go deep and usually average 5m.

BASIC SOIL PROPERTIES & THEIR CORRELATION WITH ENGINEERING PROPERTIES

The author procured samples of black cotton soil from twenty different places in India (Fig. 1) covering practically all the regions where such soils exist. The depth of the samples varied from 1–3m. The liquid limit ranged between 46 and 97, plasticity index between 22 and 49 and shrinkage limit between 11 and 14. The plot on the plasticity chart (Fig. 2) showed that practically all the soils fall above the Casagrande 'A' line. The organic content varied from 0.4 to 2.4 per cent and specific gravity had an average value[5] of 2.7.

A straight line relationship was observed between shear strength (plotted to log scale) and liquidity index (Fig. 3). For this experiment the soil was compacted at the plastic limit in a Proctors mould by the standard Proctor rammer.

*At Madurai during the INSA Council and General Meetings.

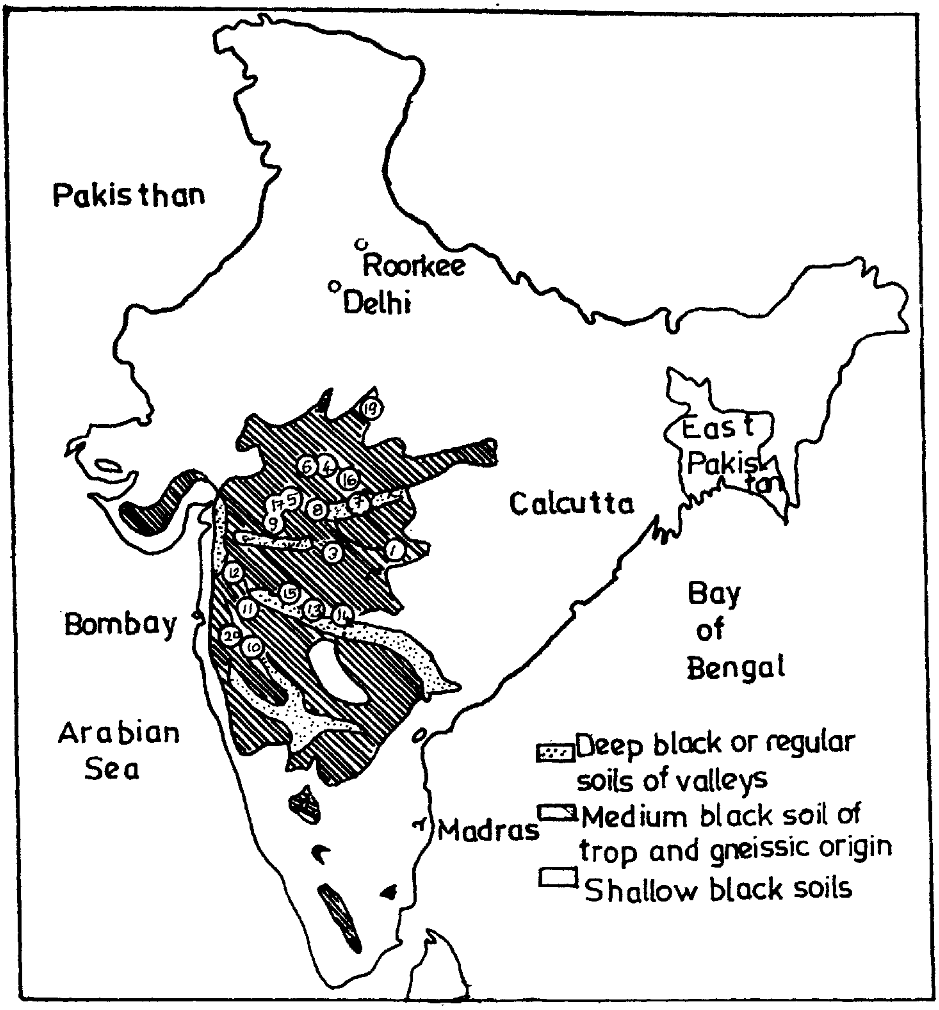

FIG 1 Soil map of India showing location of black soils

Cylindrical test specimen of diameter 5cm and height 10cm were obtained from the compacted soil mass in the mould by pushing in a thin walled steel sampling tube. These were subjected to gradual air drying and compressive strength was determined at four different moisture conditions in a decreasing order in a compression testing machine working at a uniform rate of loading. The curve has been plotted only upto liquidity index range of 0.3 which gives the minimum moisture content at which black cotton soil usually exists at a depth of 1m. The liquidity index has a negative value in all cases as the moisture content was below the plastic limit. The concentration of points is along a straight line and at the plastic limit the strength of the soil is very low.

No appreciable difference was found in strength between undisturbed and remoulded samples (Table I). This was checked by taking a portable unconfined

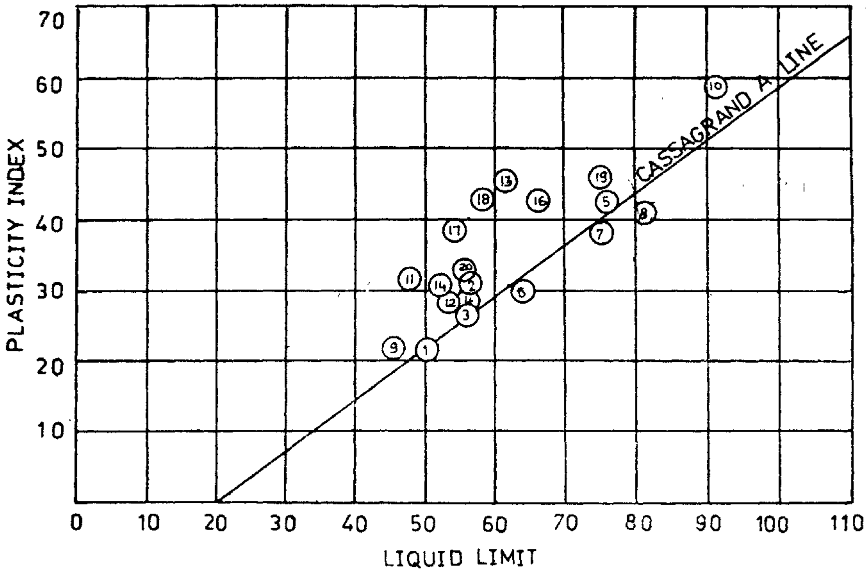

FIG 2 Relation between liquid limit and plasticity index of black cotton soil samples

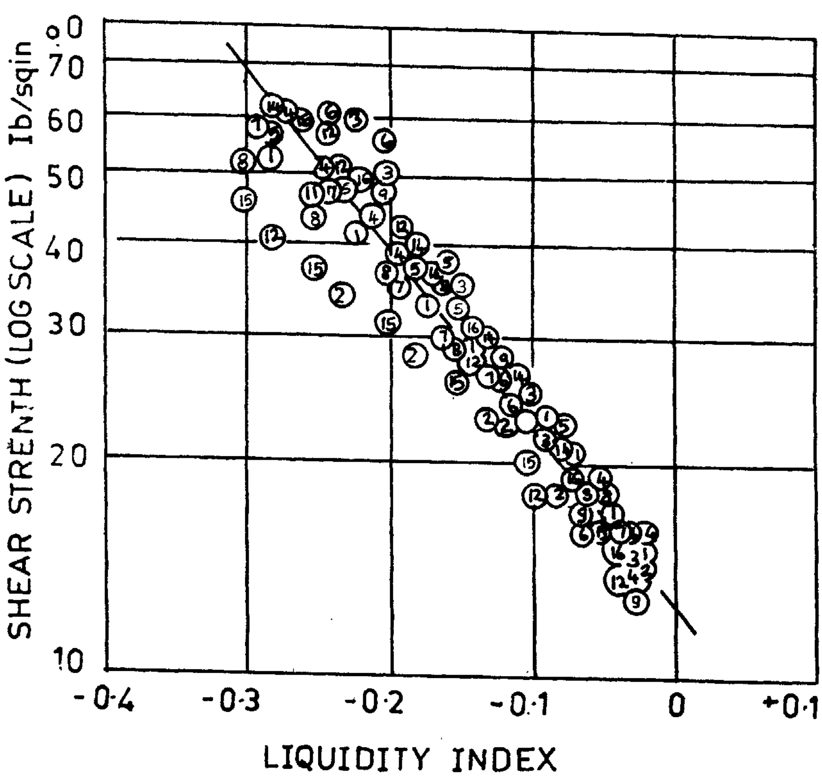

FIG 3 Shear strength *versus* liquidity index

compression test equipment to the site and testing undisturbed and remoulded samples from various depths. The sensitivity of Indian black cotton soils is therefore close to unity.

TABLE I

Location	Depth (ft.)	Shear strength undisturbed (psi)	Shear strength emoulded (psi)
Powerkhera	4	19	21
	8	21	17
	12	20	18
Indore	4	17	16
	8	22	23
	12	29	29

Differential free swell test was used to get a comparative idea of the swelling potential of the soil. The normal free swell test suggested by Holtz and Gibbs[7] consists of gently pouring 10 cc of oven dried soil, passing 36 BS sieve (Particle size less than 0.42mm), into distilled water and noting the increase in volume of the soil. The main drawback of the method is lack of uniformity in packing and long time required for the soil to come to constant volume in a soaked state. The differential free swell test developed by RDSO[8] overcomes this drawback. In this method, 10g of two oven-dried samples of soil are soaked, one in distilled water and the other in kerosine oil or some other non-polar liquid, their respective volumes being measured. The difference in volume expressed as a percentage

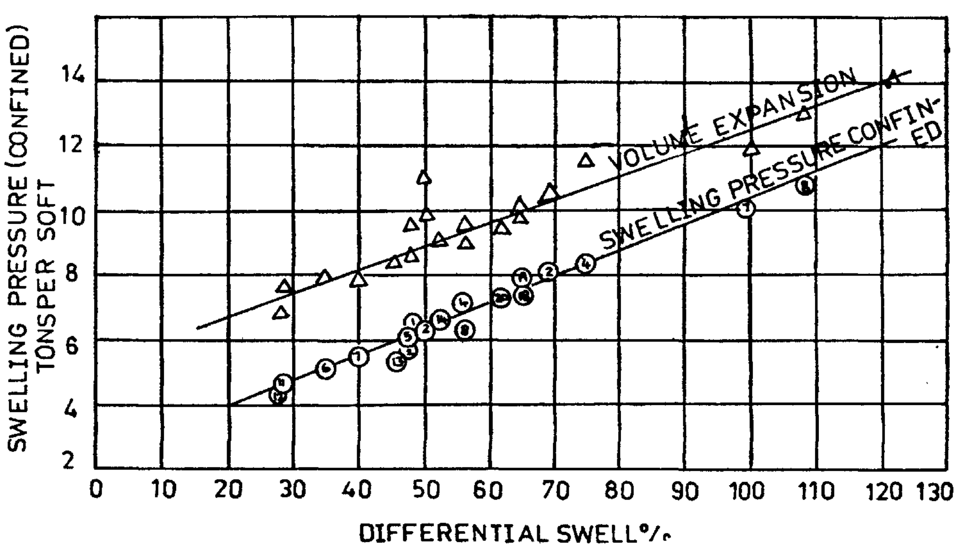

FIG 4 Relation between differential swell, and swelling pressure and volume expansion of black cotton soil samples

of the volume in kerosine oil gives the differential free swell. The values were found to range from 28 to 122. A curve was plotted between the differential free swell on one hand and volume expansion and swelling pressure on the other. A straight line relationship was observed (Fig. 4).

Foundation Design and Construction

Foundation of buildings and other structures on Indian Black cotton soils have been a matter of great concern to the engineers and builders. It is heart-rending to see some of the structures cracking badly within a year after their construction.

Untill 1955, the normal methods for constructions of foundations on black soils were —

(1) Provision of reinforced concrete bands at plinth and lintel level.

(2) Provision of reinforced concrete raft or beam to support the superstructure.

(3) Removing the black soil entirely or to a considerable depth and backfilling the trench with cohesionless soils.

Method (1) was not very effective and method (2) was costly. Method (3) is also uneconomical where depth of black soil exceeds 1m.

The main reason leading to the failure of foundations in black soils is the differential movement of the structure with uneven ground movements due to alternate swelling and shrinkage of the soil. The vertical ground movements decrease with depth and are negligible at a particular depth. Investigations were carried out by CBRI to locate this depth in two different regions with deep layers of black soils. Test sites were established at both these places and each of the sites had a row of G.I. pipes anchored at different depths from 6 in (15cm) to

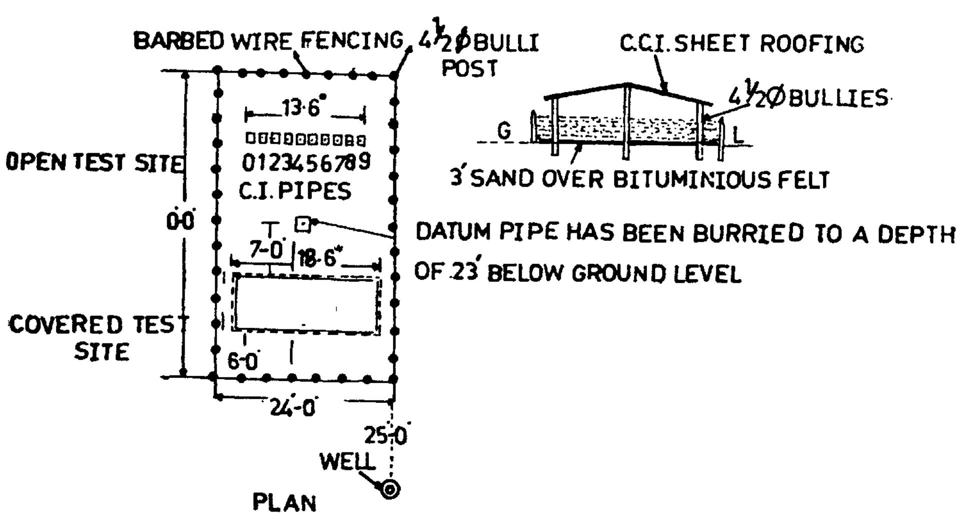

Fig 5 Plan of the test site

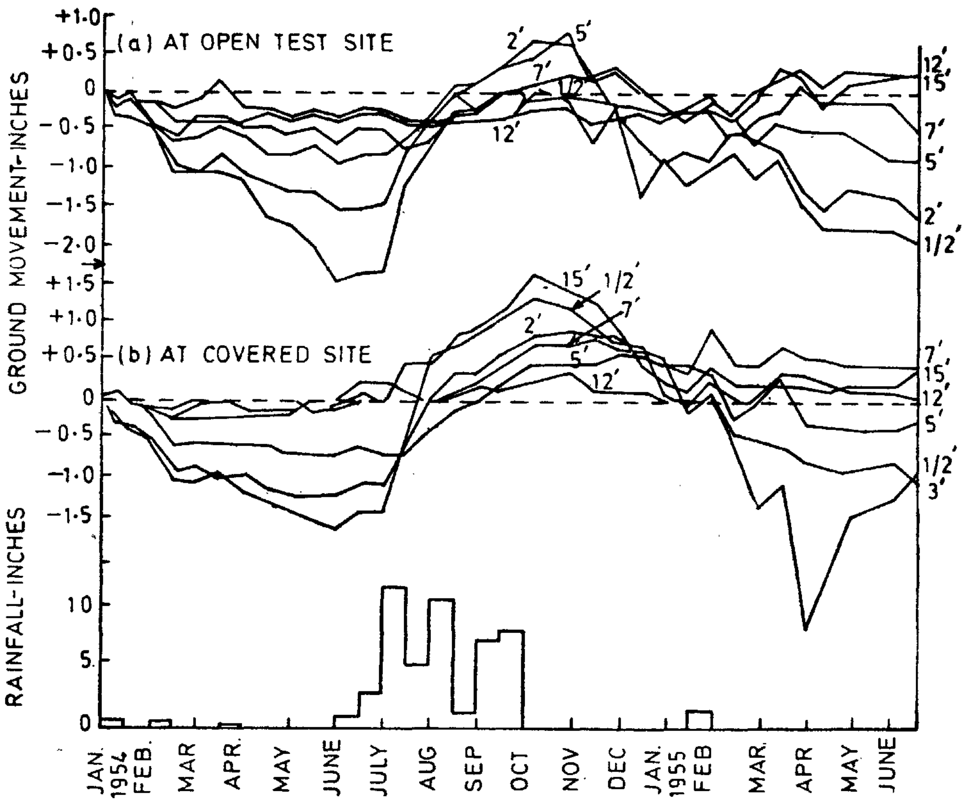

FIG 6 Record of vertical ground movement at Hoshangabad

15 ft. (4.5m) deep. There were two such rows at each site, one in the open and the other under a shed and protected by bituminous felt to simulate the cover produced by a building (Fig. 5). Fortnightly levels were recorded of each pipe over a period of 13 to 18 month and these have been plotted in Fig. 6 and 7. From a study of the curves it would be seen that 12 feet (3.5m) is the depth at which the vertical ground movements are inappreciable.

Another test site 25m × 35m was set up after a lapse of about 15 years in a black cotton soil area where the top 2.7m was black silty clay overlying yellow clay which extended to 5m and beyond.[11] A number of surface movement indicators, depth gauges and *in situ* swelling pressure measuring instruments were installed and measurement were carried out over a period of 3 years (Fig. 8). Maximum value of ground movement at the surface was found to be 65mm which decayed to a neglible value at about 5m depth. Bulk of the movement was found to occur within the top 2m and 10 per cent of maximum occurred at nearly 3/4 of the depth of negligible heave. A depth of 5 × 3/4 = 3.75m could therefore be taken as a safe depth for foundations. *In situ* swelling pressure studies were carried out at depths varying from 0.5m to 2.5m using individual beam set up. (Fig. 9). In order to accelerate swelling, the area was continuously

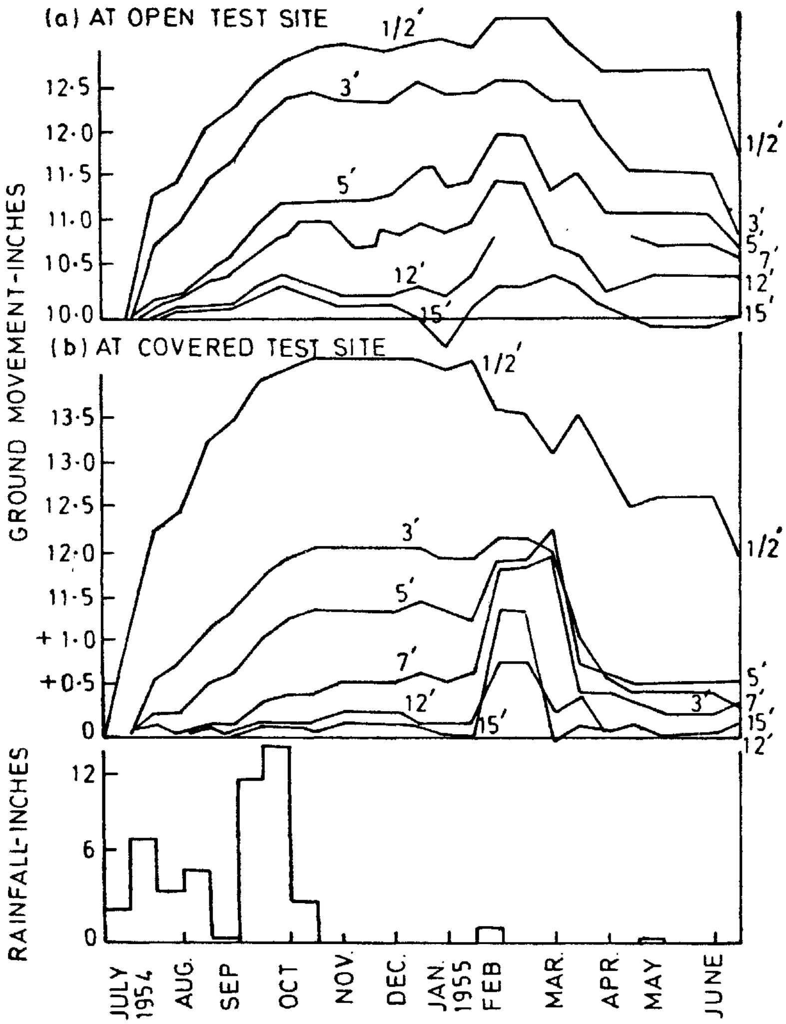

FIG 7 Record of vertical ground movement at Indore

flooded for a period of five month. Out of a total six swelling pressure set-ups two measured *in situ* swelling pressures by accelerated tests. Swelling pressure values (6_s) as high as 5.5kg/cm^2 was observed with depth (D) which could be expressed by the following relationship $6_s = 8 - \delta\ 1.6D$. Having found 3.50–3.75m as depth of inappreciable ground movement, a design was developed of under-reamed pile foundations with the belled out portion anchored at a depth of 3.5m or earlier if water table or a stable strata was encountered. The boring for the pile was carried out by a spiral auger and its bottom was under-reamed to about 2½ times the shaft diameter. For this purpose a portable, hand-operated under-reaming tool was designed (Fig. 10). The tool is simple to fabricate in a workshop. It consists of an assembly of four steel blades fixed around a central

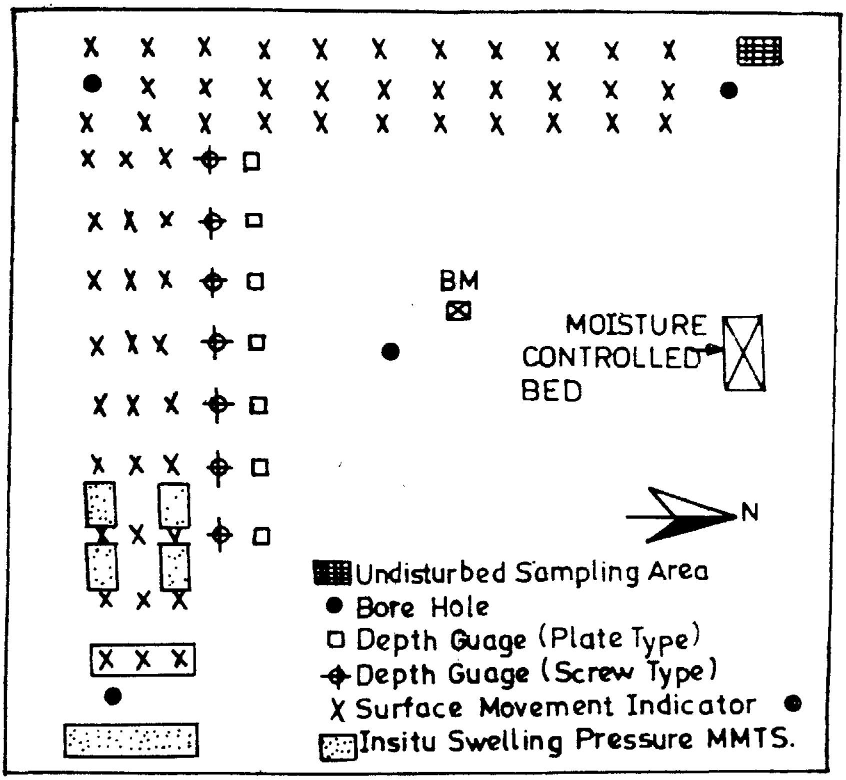

FIG 8 Site for field experiment showing layout of measuring instruments

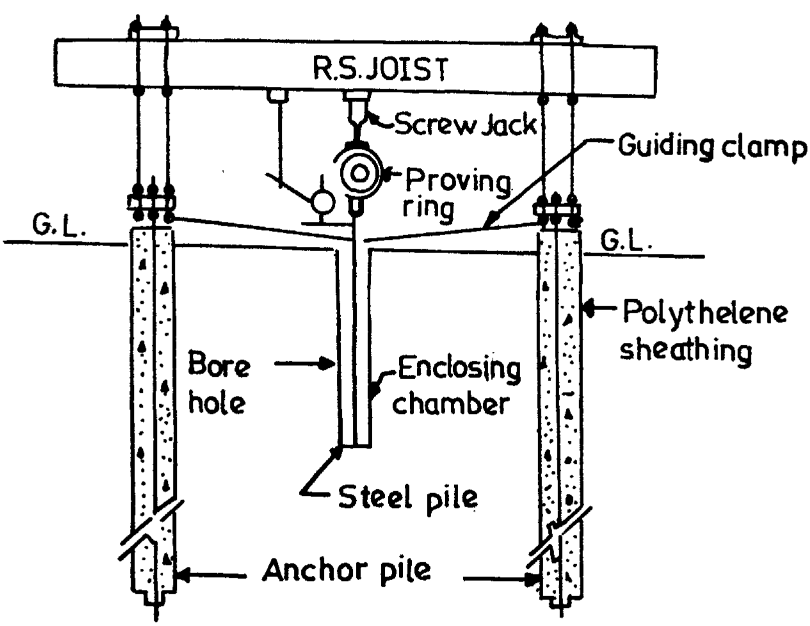

FIG 9 Swelling pressure measurement using individual beam set up

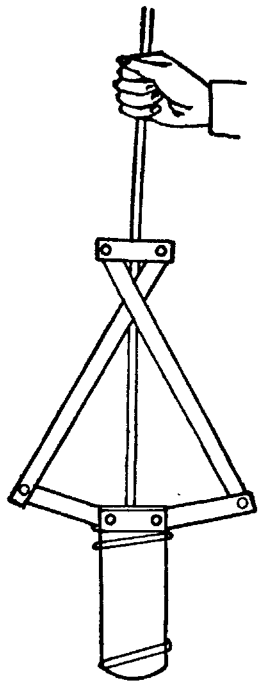

FIG 10 Under-reaming Tool

shaft and a bucket to hold the cut soil. The blades widen out as the shaft is pressed downwards and by rotating them the bore hole is widened. The shaft has a number of holes and a pit[12] inserted in a particular hole controls the maximum diameter of the bell.

The pile diameter varies from 20 to 50cm depending on the load it is expected to carry. For small works, in out of the way places, manual operation is usually preferred but for larger diameter and deeper piles, a mechanised boring rig is normally necessary.

The piles are provided at appropriate locations keeping in view the layout of the building and the load carrying capacity of the piles. As far as possible all piles are uniformly loaded and the spacing so adjusted as to keep the door and window openings mid-way between two piles. A typical layout of a residential building and details of piles is given in Fig. 11(a) and a section through the pile foundation is given in Fig. 11(b). It would be seen that the grade beam is kept 3 in (8cm) clear of the ground so that the soil does not heave against it. These beams, carrying the masonry superstructure are designed for a bending moment of $WL/50$ to allow for panel action in the masonry. The shuttering supporting the beams is therefore not removed for about a week.

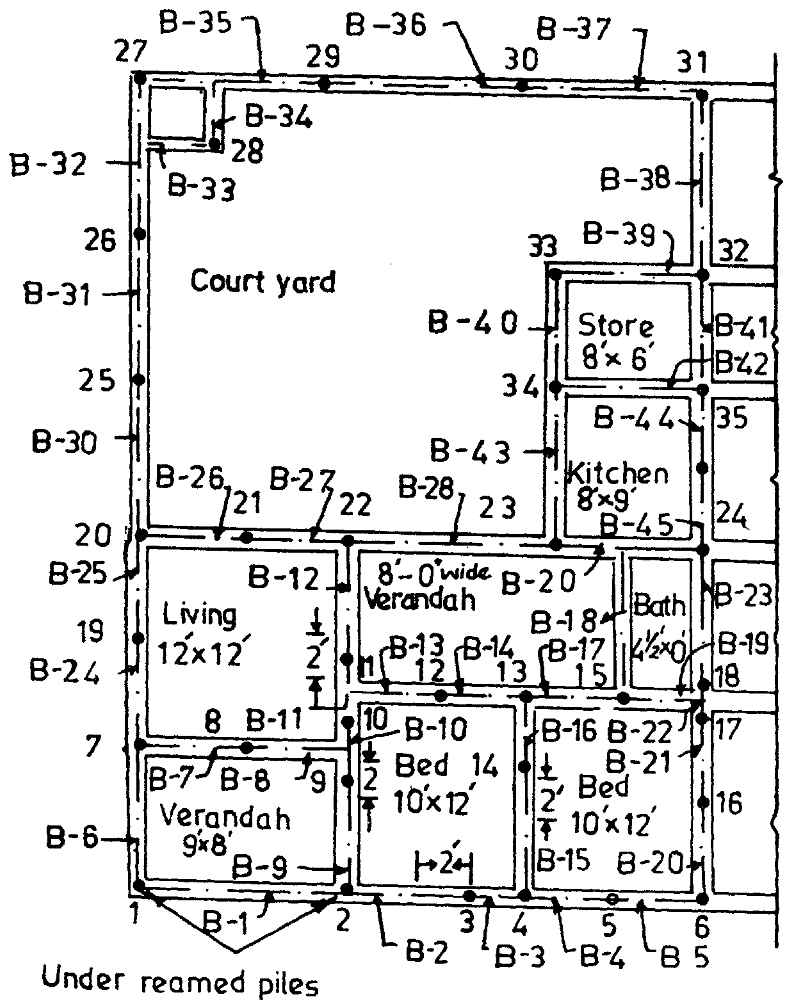

FIG 11 (a)

The bearing capacity (Qu) of under-reamed piles can be determined by the following expression normally used for bored piles,

$$Qu = Ap\, N\, Cp + \alpha\, \bar{C}\, As \qquad \text{(i)}$$

where, Ap = area of the pile tip (under-reamed base of the pile)

N = bearing capacity factor (may be taken 9)

Cp = undisturbed shearing strength of the soil at the bearing level

α = reduction factor for bored piles

$\bar{C}$ = average undisturbed shearing strength of the soil along the pile length

As = Surface area of the pile shaft

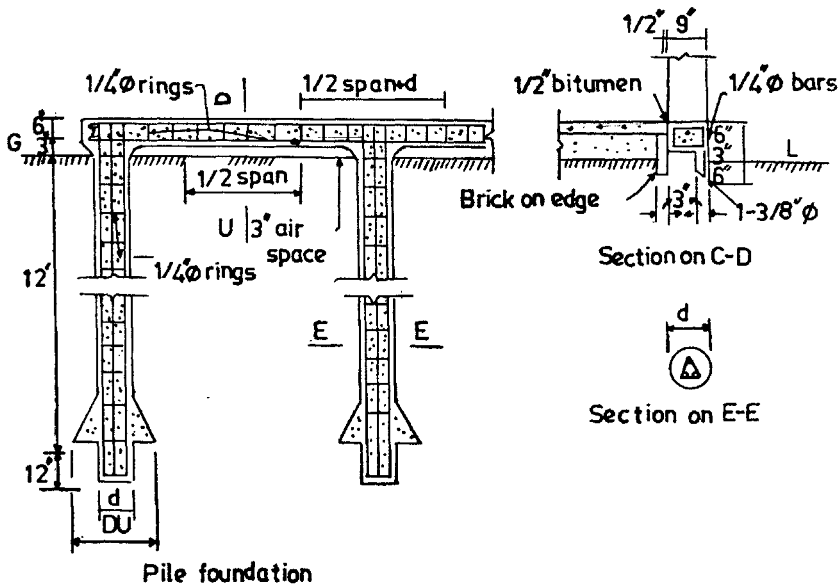

FIG 11 (b)

The value of reduction factor (α) may be taken as 0.5. This has been determined by carrying out a series of loading and pull out tests on *cast-in-situ* bored concrete piles in black cotton soils at four different sites in India.[10]

The adoption of under-reamed pile in expansive soils has resulted in economy to the extent of 30-60 per cent when compared to traditional strip footings. An additional advantage is that the process is quick and no extra excavation or backfilling is required. This provides better and more uniform conditions for floor finishes adjacent to the walls.

A full scale load test[12] was carried out on a 7.6 m long and 45 cm stem diameter multi-under-reamed pile, having four bulbs, each 112.5 cm diameter, in a deep layer of black cotton soils, having undrained cohesion varying from 0.9 to 1.45kg/cm^2, liquid limit from 65 to 70 and plasticity index from 35 to 50. Twelve under-reamed piles, arranged in a circle, were used as anchors for the loading frame designed to carry a load of 400 tonnes. A sectional view of the test set up is shown in Fig. 12. Load carrying capacity of the test pile as worked out from table *Appendix-A*, was found to be 290 tonnes. The computed value, using bearing capacity formula (1), gave a value of 328 tonnes. The load settlement curve of the test pile is shown in Fig. 13. The pile could only be tested upto 300 tonnes at which the settlement was only 14 mm. Further testing was not possible due to yielding of the anchor piles. It is clear from the load test that the ultimate capacity of the pile is more than 300 tonnes and could be close to the computed value of 328 tonnes.

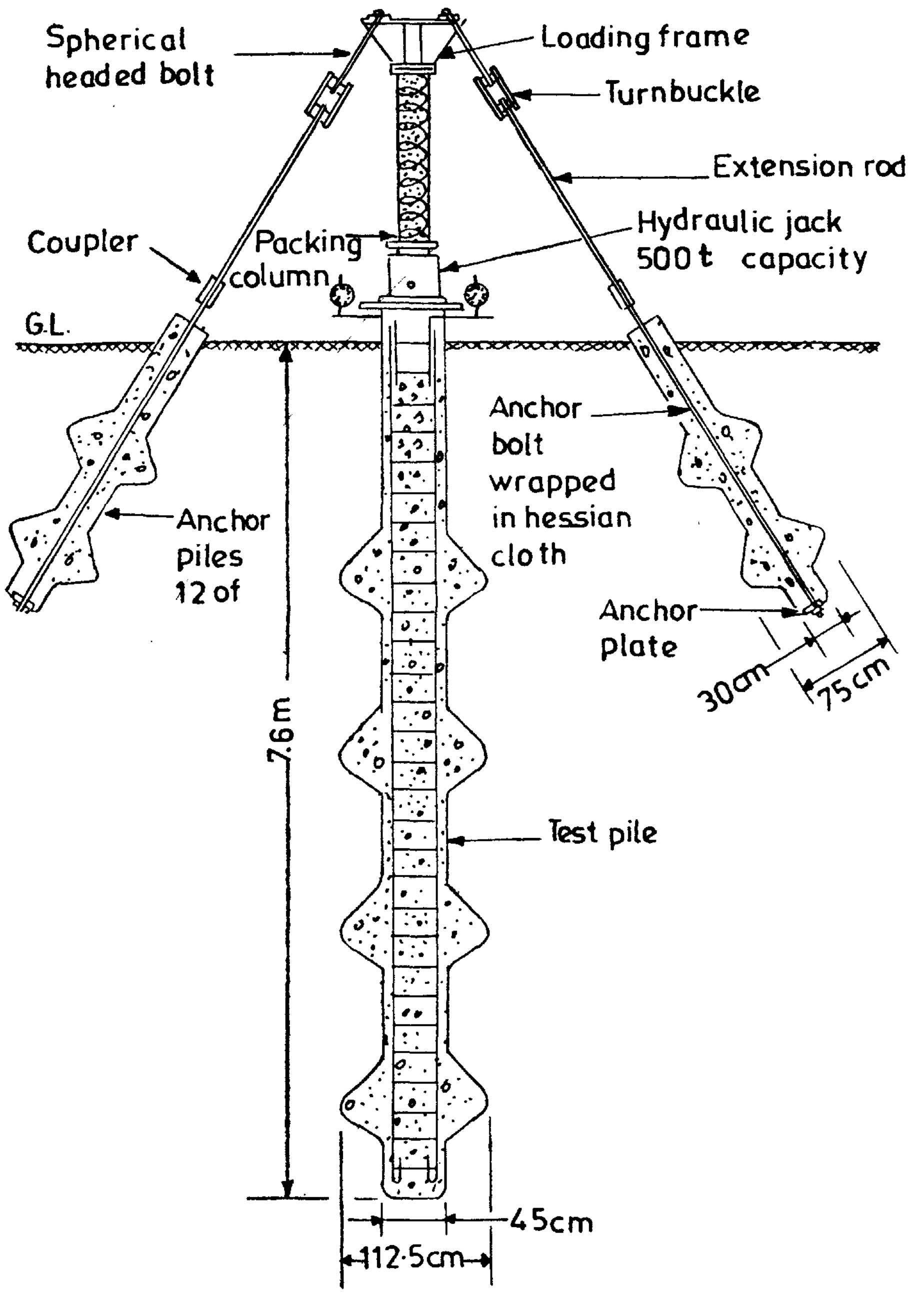

Fig 12 Sectional view of the pile load test set-up

Concluding Remarks

The use of under-reamed piles in India is now an accepted practice in black cotton soil areas. It is no longer restricted to expansive soils or buildings alone

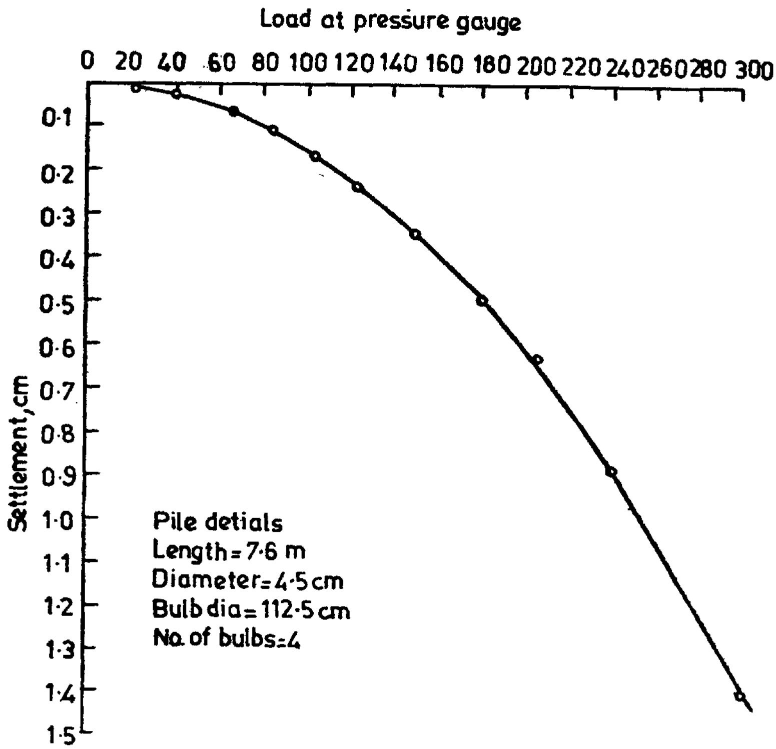

FIG 13 Load settlement curve

but is used in other types soil conditions such as filled up grounds and poor soils overlying firm strata, foundations subjected to uplifts and thrusts such as transmission line towers, underground tanks etc. A private developer has even adopted them for normal grounds in view of their speedy construction. An Indian Standard code of practice [IS : 2911 (Part III) 1980] on under-reamed piles has been brought out to help the designer and a safe load table provided in the code has been given in *Appendix-A*.

It can therefore be safely claimed that under-reamed pile foundations have provided a foolproof, economical and quick to construct solution for foundation in expansive clays in India.

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Appendix A

Extract from Indian Standard Code of Practice for Under-reamed Piles–IS : 2911 (Part III)–1980

TABLE

Safe load for vertical bored cast in situ *under-reamed piles in sandy and clayey soils including black cotton soils*

Size		Length		Mild Steel Reinforcement			Compression				Safe Loads in Uplift Resistance				Lateral Thrust	
Diameter of pile	Under-reamed diameter	Single under-reamed	Double under-reamed	Longitudinal reinforcement		Rings spacing of 6mm dia rings	Sinlge under reamed	Double under-reamed	Increase per 30cm length	Decrease per 30cm length	Single under-reamed	Double under-reamed	Increase per 30cm length	Decrease per 30cm length	Single under-reamed	Double under-reamed
cm	cm	m	m	No.	Dia mm	cm	*t*	*t*	*t*	*t*	*t*	*t*	*t*	*t*	*t*	*t*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
20	50	3.5	3.5	3	10	18	8	12	0.9	0.7	4	6	0.65	0.55	1.0	1.2
25	62.5	3.5	3.5	4	10	22	12	18	1.15	0.9	6	9	0.85	0.70	1.5	1.8
30	75	3.5	3.5	4	12	25	16	24	1.4	1.1	8	12	1.05	0.85	2.0	2.4
37.5	94	3.5	3.75	5	12	30	24	36	1.8	1.4	12	18	1.35	1.10	3.0	3.4
40	100	3.5	4.0	6	12	30	28	42	1.9	1.5	14	21	1.45	1.15	3.4	4.0
45	112.5	3.5	4.5	7	12	30	35	52.5	2.15	1.7	17.5	25.75	1.60	1.30	4.0	4.8
50	125	3.5	5.0	9	12	30	42	42	63	1.9	21	31.5	1.80	1.45	4.5	5.4

1. The safe bearing, uplift and lateral loads for under-reamed piles given in the Table on p. 28 apply to both medium compact ($10 < N < 30$) sandy soils and clayey soils of medium ($4 < N < 8$) consistency including expansive soils. The values are for piles with bulb diameter equal to two-and-a-half times the shaft diameter.
   The columns (3) and (4) of Table provide the minimum pile lengths for single and double under-reamed piles, respectively, in deep deposit of expansive soils. Also the length given for 375 mm diameter double under-reamed piles and more in other soils are minimum. The values given for double under-reamed piles in columns (9) and (13) are only applicable in expansive soils. The reinforcement shown is mild steel and it is adequate for loads in compression and lateral thrusts (Columns (8), (9), (16) and (17)). For up lift (Columns (12) and (13)), requisite amount of steel should be provided. In expansive soils, the reinforcement shown in Table is adequate to take upward drag due to heaving up of the soil. The concrete considered is M 15.
2. Safe loads of piles of lengths different from those shown in Table can be obtained considering the decrease or increase as from Columns 10, 11, 14 and 15 for the specific case.
3. Safe loads for piles with more than two bulbs in expansive soils and more than one bulb in all other soils (including non-expansive clayey soils) can be worked out from Table by adding 50 per cent of the loads shown in columns (8) or (12) for each additional bulb to the values given in these columns. The additional capacity for increased length required to accommodate bulbs should be obtained from columns (10) and (14).
4. Values given in columns (16) and (17) for lateral thrusts shall not be increased or decreased for change in pile lengths. Also for multi-under-reamed piles the values shall not increase than those given in column (17). For longer and/or mult-under reamed piles higher lateral thrusts may be adopted after establishing from field load tests.
5. For dense sandy ($N \geqslant 30$) and stiff clayey ($N \geqslant 8$) soils, the safe loads in compression and uplift obtained from Table may be increased by 25 per cent. The lateral thrust values should not be increased unless the stability and strength of top soil (strata upto a depth of about three times the pile shaft diameter) is ascertained and found adequate. For piles in loose ($4 < N \leqslant 10$) sandy and soft ($2 < N \leqslant 4$) clayey soils, the safe loads should be taken as 0.75 times the values shown in the Table. For every loose ($N \leqslant 4$) sandy and very soft ($N < 2$) clayey soils the values given in the Table should be reduced by 50 per cent.
6. The safe loads obtained from Table, should be reduced by 25 per cent if the pile bore holes are full of subsoil water or drilling mud during concreting.
7. The safe loads in uplift and compression given in Table or obtained in accordance with 2 to 6 should be reduced by 15 per cent for piles with bulb of twice the stem diameter. But no such reduction is required for lateral loads shown in Table.
8. The safe loads in Table and the recommendations made to obtain safe load in different cases (2 to 8) are based on extensive pile load tests obtained may be taken equal to two-thirds the loads corresponding to deflection of 12mm for loads in compression and uplift. The deflections corresponding to respective safe loads will be about 6mm and 4mm. The deflection at safe lateral load will be about 4mm. The values given in Table will be normally on conservative side. For working out ultimate compressive and uplift loads, if defined as loads corresponding to 25mm deflection on load-deflection curve, the value obtained from Table can be doubled. In case of lateral thrust twice the values in Table should be considered corresponding to deflection of 12mm only.
9. For piles/subjected to external moments and/or larger lateral loads than those given in Table the pile should be designed properly and requisite amount of steel should be provided.