Indian Geotechnical Journal, 18(4), 1988

Performance of Instrumented Underreamed Pile Foundation Supporting a Single Storey Structure in Expansive Soil

by

Chandra Prakash*

I. F. Wardle**

Ramesh Chandra*

G.R. Balodhi*

G. Price**

Introduction

Design of piles in expansive soils should normally meet the following requirements:

(*a*) The pile should have adequate capacity to carry the structural loads.

(*b*) It should have adequate strength to withstand the uplift forces induced on account of soil heave.

(*c*) The displacement of the pile due to net effects of uplift forces and downward loads should be within permissible limits.

However, the guidelines available for design of underreamed piles in expansive soils of India [IS:2911 (Part III)—1980, Sharma *et al.*, 1978] do not include any information to analyse the piles for satisfying the latter two requirements. Further no data is available on measured load transfer through these piles which provide direct input to improve upon design methodology.

The present recommendation regarding the use of minimum 3.5 m deep underreamed piles in deep deposits of expansive soil is based on field measurements of ground movements, carried out in fifties, which revealed the negligible vertical soil movements (heave) at this depth. However, subsequent measurements by Gupta *et al.*, (1983) revealed that the heave reduced to negligible value at about 5 m depth. The rate of reduction could be defined by the exponential curve. At a depth of 3.75 m

*Central Building Research Institute, Roorkee, India

**Building Research Establishment, Garston, Watford, U.K.

(*The modified manuscript of this paper was received in July*, 1988 *and is open for discussion till the end of June*, 1989)

(3/4 of that to zero heave), the heave was only about 10% of the surface value of 55 to 65 mm. Since a certain amount of movement is acceptable in foundations, Gupta *et al.*, recommended that the underreamed piles should be taken to a depth at which 10% of maximum heave occurred. Poulos and Davis (1973 and 1980) based on elastic analysis found that the most efficient means of reducing pile movements resulting on account of swelling of sub-soil was by using either underreamed piles founded at or just below the base of the swelling zone (active zone) or uniform diameter piles of length about twice the depth of active zone. Thus in order to provide information on measured load transfers and to evaluate performance of 3.5 m deep underreamed piles, a detailed field study was carried out on full scale instrumented piles in a black cotton-soil deposit having swelling zone more than 3.5 m deep (CBRI-BRE Project Report, 1986; Bhandari *et al.* 1987, Price *et al.* 1988).

In view of the fact that the behaviour of piles is different under actual structure, two of the single underreamed piles supporting a single room at same site, were also instrumented to investigate the net effect of structural loads and the load induced in toe piles on account of swelling of sub-soil. The current paper presents the results of measured load transfers through these piles and movement of foundations. A comparison between measured and the predicated values by the method based on elastic analysis (Poulos and Davis 1980) is also given.

Test Site and Soil Data

The study was carried out in black cotton soil at a site within the campus of Shri Govindram Seksaria Institute of Technology and Science at Indore in Madhya Pradesh. In the region, the soil shrinks chiefly between March and middle of June and it tends to swell during the rainy season which normally falls between June and October. The layout plan of the site showing the location of room and its details are given in Fig. 1. During summer, cracks of a polygonal pattern were observed at the site, suggesting a high potential for expansiveness.

The sub-soil characteristics alongwith averaged out properties of detailed sub-soil investigations carried out in March and September-October (towards end of rainy season) at the site adjacent to the room are given in Fig. 2. The sub-soil consisted of blackish silty clay upto 2.7 to 2.9 m depth followed by yellowish clay with nodules upto 5 m depth explored. No water table was observed upto this depth. According to the Indian Standard Classification the soil falls under the group of CH (fine grained soils of high compressibility). The variation in moisture content between dry and wet weather was inappreciable below 4.5 m. This suggests the depth of the swelling zone (active zone) of about 4.5m which is in agreement with the value of 5m observed by Gupta *et al.*, (1983) close by to the test site.

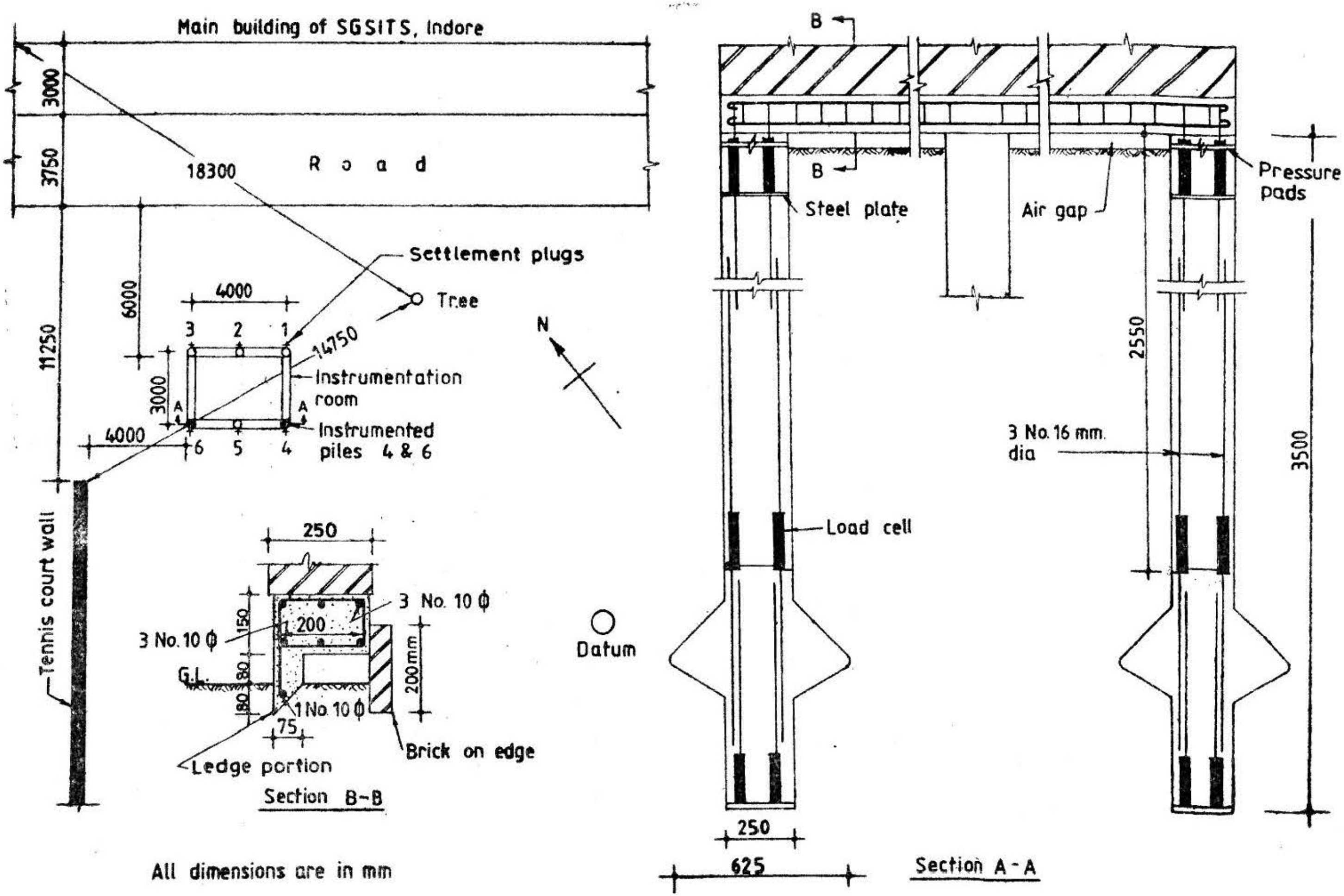

FIGURE 1 Details of Room

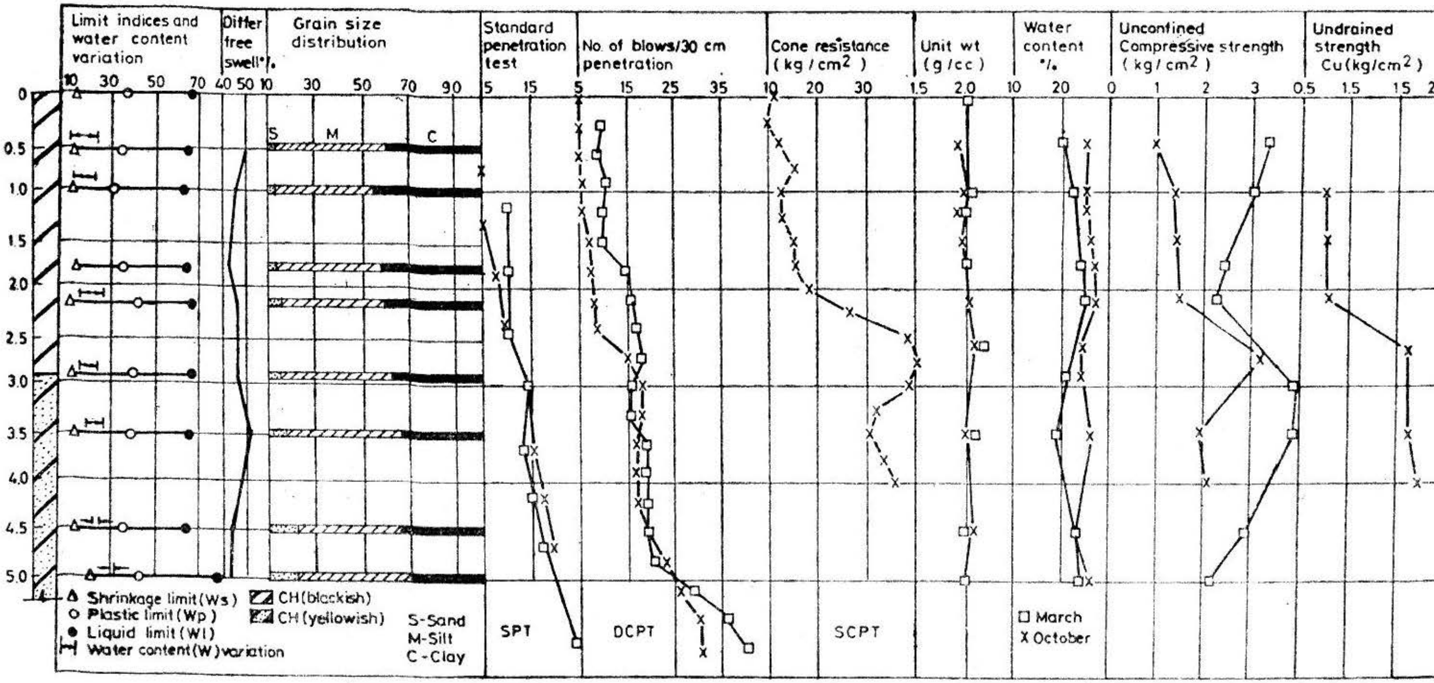

FIGURE 2 Sub-Soil Characteristics at the Test Site

The 'differential free swell' or 'free swell index' values given in Fig. 2 were determined in accordance with IS:2720 (Part XL—1977).

Based o the values of limit indices, differential free swell, clay content and its activity, mineralogical composition and the percent swelling potential as determined by various correlations available in the literature (Seed *et al.*, 1962, Ranganatham and Stayanarayana 1965, Nayak and Christensen 1979, Vijayvergia and Ghazzaly 1973, Brackley 1975, O'Neill and Ghazzaly 1977, Bandyopadhyaya 1981), the soil was designated of high to very high expensiveness. Using the values of swelling potential, total surface heave was calculated considering reduction in heave with depth as given by Vender Merwe (1964), Vijayvergia and Ghazzaly (1973), and Gupta *et al.*, (1983). The depth of active zone was taken as 4.5m. The values obtained by the methods given by Vijayvergia and Ghazzaly and Gupta *et al.*, using percentage swell calculated by the relationship given by Bandyopadhyaya were 63 mm and 71 mm respectively. These were quite close to the values of maximum heave of 55 to 65 mm measured by Gupta *et al.*, adjacent to the site and also as reported by Mohan and Jain (1958). Later in this paper the maximum value of heave of 60 mm as measured is used to calculate the net effect of uplift forces and structural loads in the piles.

Both the static and dynamic penetration tests reflected that the strata were more uniform during October than in March. This would be expected since the rains caused a uniform wetting of the top layers of the soil. While the variation in moisture contents of both the periods was found negligible only at 4.5 m depth and onward, the penetration resistance showed hardly any variation beyond 2.5 m depth. Also an increase in strength with depth was reflected by the penetration resistance, more predominantly during October than in March. In top 2 m unconfined compressive strength reduced considerably during October as compared to March reflecting the effect of wetting. The static penetration resistance during October and unconfined compressive strength during both the periods revealed a stiffer layer between 2.25 m to 3.25 m. The natural and dry unit weights were fairly constant with depth during both the seasons. The values of undraied shear strength (c_u) obtained in the laboratory during October agreed well with those determined using the relationship, $c_u = q_c/20$ (q_c = static cone penetration test resistance as suggested by various workers (Sanglerat 1972). The values of elastic modulus E obtained from stress-strain plots (secant modulus) for unconfined compressive strength and undrained tests carried out in October were very similar, ranging 35 to 45 kg/cm^2 upto a depth of 2 m, 70 to 80 kg/cm^2 at 2.6m and 45 to 50 kg/cm^2 from 3.5 to 4.5 m depth. Using the static cone penetration resistance, the values of E as obtained from the correlations summarised by Sanglerat were 35-50 kg/cm^2 for the top 2 m, 100-150 kg/cm^2 for the next one metre and 75-100 kg/cm^2 to 4.5 m depth.

Details of Room

The room is 3 m × 4 m centre to centre in plan with ceiling height of 3 m (Fig. 1), founded on six 25 cm diameter, 3.5 m deep single underreamed (underreamed diameter 62.5 cm) piles. A grade beam 20 cm×20 cm linked the piles together and supported 25 cm thick brick masonry walls of the room. The roof is of flat RCC slab. The door is provided at the far right hand corner of south-east wall and the window is in the centre of south-west wall. A view of the finished strucure is shown in Fig. 3.

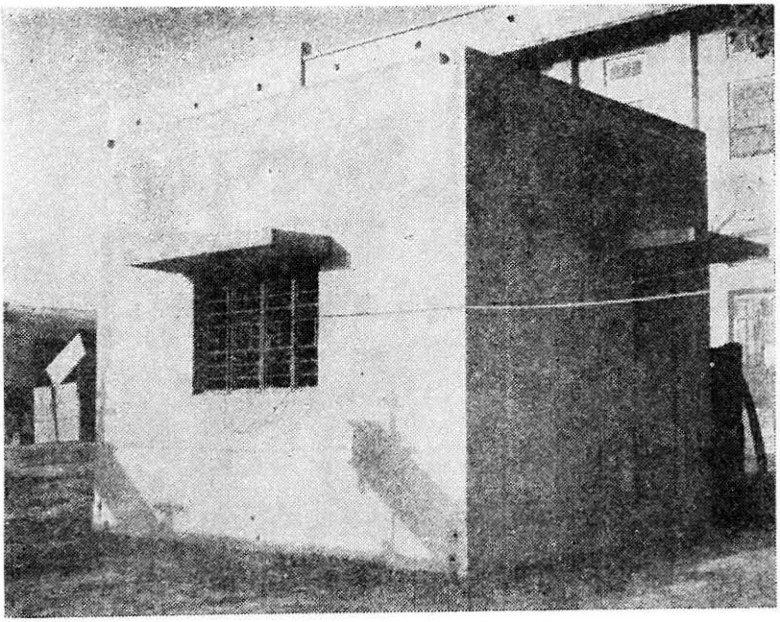

FIGURE 3 View of Instrumentation Room

The piles were constructed in accordance with IS:2911 (Part III)—1980 and Sharma *et al.*, (1978). The concrete used was of 1:2:4 nominal mix which corresponded to M 15 (cube strength 15 N/mm^2) grade.

Instrumentation of Foundations

Two piles, No. 4 and No. 6 (Fig. 1) were instrumented with load cells at three levels, one at toe, one just above the underream and one just at the junction of the pile and beam. For monitoring the foundation movements, six plugs were also fixed in the beam towards top edge just above each pile. These movements were monitored with respect to a datum installed at a depth of 5.5 m about 15 m away form the room.

The load cells were designed and fabricated using three vibrating wire type load sensing units. The sensing units, capable of monitoring

200 kN axial loads both compression and tension, were steel tubes of 200 mm long fitted internally with vibrating wire gauges. The loads in the piles at the level of load cells were transferred wholly to the sensing units through reinforcing steel only. This was achieved by creating a discontinuity in the concrete by means of a 3 mm thick soft membrane formed across the pile shaft through which only sensing units bridged with reinforcement. The gap formed by the soft membrane provided adequate working clearance for the sensing units to deform *i.e.*, assuming a working strain of 1000 micro strain, the elastic shortening of 200 mm long unit is only 0.2 mm. Each load cell was built as a separate unit with its own length of reinforcement at top and bottom. The length of reinforcement was designed to overlap in the shaft of pile to provide required bond and continuity for the pile. Small diameter inflatable tubes were used to seal the bore hole at each load cell position. The details of construction of load cells and their installation in the piles were similar to that reported by Prakash *et al.* (1987).

After casting of instrumented and other piles in January 1983, the grade beam (Fig. 1) was laid without losing much time. For fixing plugs, two hexagonal nuts of 25 mm diameter welded together were embedded in the beam towards the top face of beam at desired locations at the time of concreting. After 14 days, the walls were raised and construction completed in March, 1983.

Monitoring of Data

The readings of load cells were taken since the pile installation (27 January, 1983) by a frequency counter meter which monitors time for 100 cycles of the signal. The load in a particular sensing unit is calculated by the change in time period and using calibration factor for that sensing unit. The algebraic sum of load in three sensing units provided the load at that load cell level. The observations were continued at regular intervals till August, 1985. In order to investigate the effect of embedment of ledge projection of beam on outer edge and brick on edge on inside face, the soil around and beneath the beam adjacent to pile No. 6 was removed during April 1984, *i.e.*, prior to second rainy season. Throughout the observaton time all the load cells responded very satisfactorily.

The displacement of foundations was monitored with respect to a datum by simple water tube level and normal levelling instrument. The water tube level used consisted of a water reservoir at one end connected by a 12 mm tube to a sight glass mounted on frame against a scale. The smallest division of the scale was of one milimetre. The level in the reservoir moves negligibily compared to the level in the narrow tube, so difference in level between points were measured directly on the scale. The interval of dispalcement observations was more than that of the load cell observations and these were taken upto the beginning of November, 1984.

Results and Discussions

Load Distribution

The recorded loads by each of the load cells in pile No. 4 and pile No. 6 with respect to time are given in Fig. 4. Initially soon after the laying of beam and prior to superstructure construction, the top and shaft load cells in both the piles showed the tensile loads. The probable factor responsible for these is the expansion of soil adjacent to pile and ledge portion of beam on account of increase in moisture, migrated from the piles. As the walls started building up, these load cells showed compressive loads in the piles as expected. At the end of construction the top load cells in pile No. 4 and pile No. 6 recorded 38 kN and 34 kN load respectivley. Since the window is half way between the two piles and the door remote from both, the load on each pile could be expected to be similar as observed. Calculated from the dead load of the component parts of the structure, the expected loads on the piles were 47 kN (No. 4) and 52 kN (No. 6).

If these estimates of construction loads are correct then about 9 to 18 kN of load was carried by the grade beam adjacent to the respective piles.

During the summer as the soil dried out and shrank, all the three load cells in both the piles showed an increase in compressive loads. The increase in load in top load cells confirmed that some of the construction load had been carried by the beam, and as the soil shrank from under, the load was transferred to the piles. It would appear that more shrinkage occurred around pile No. 4 (more load was transferred to it) possibly due to the tree. The maximum load carried by the piles would suggest that the building was slightly heavier than estimated. During peak summer (middle of May to June 1983), there was reduction in load transfer through shaft with a consequent increase in load transfer through underream portion indicating partial separation of pile shafts with the soil.

During rainy season, from July 1983 onwards the development of uplift forces on account of swelling of sub-soil was recorded in both the piles. This clearly demonstrated that even through there was sustained downward load from superstructure on the piles, these were subjected to net tensile loads. The maximum tensile loads were recorded at second load cell level *i.e.*, 2.6 m depth, about 0.76 times the length of pile. This is in agreement with the depth at which the maximum tensile loads occurred in unloaded experimental piles during rainy season (Bhandari *et al.*, 1987). Out of the two piles, the maximum tensile load, being just over 40 kN was recorded in pile No. 6. The less tensile load recorded in the pile No. 4 was probably on account of tree which reduced the soil swell around this pile.

Subsequently, after rainy season when sub-soil started drying up,

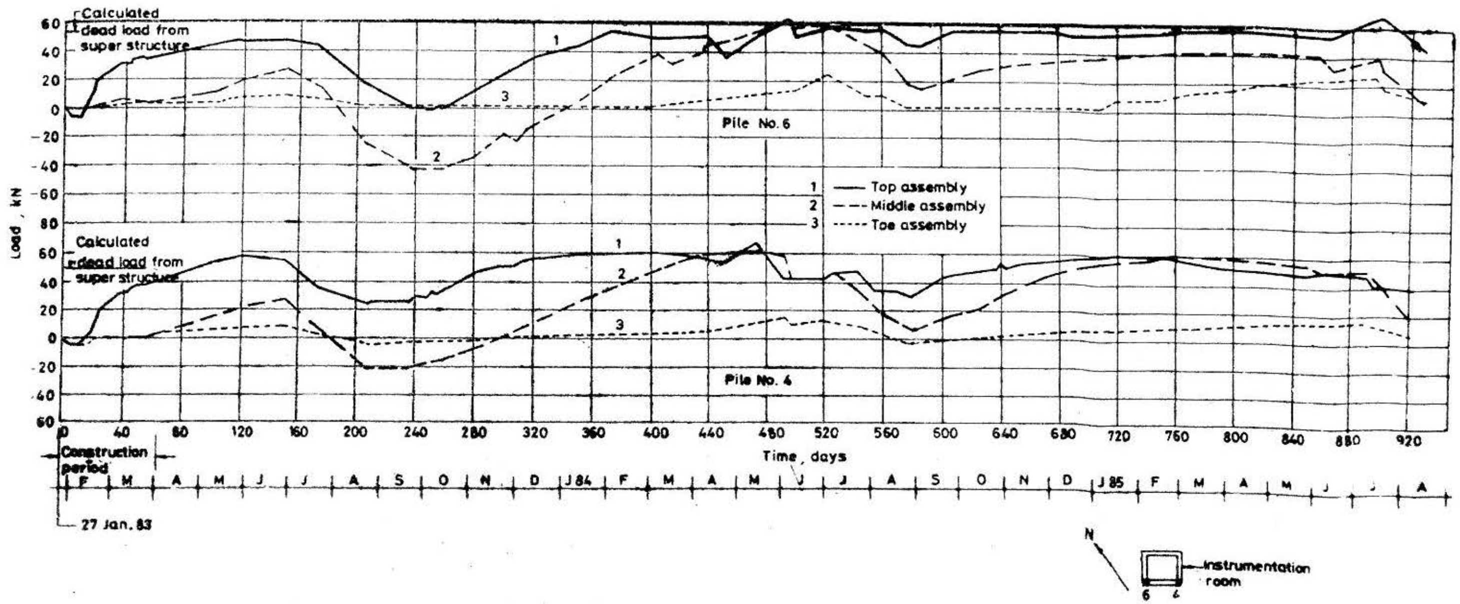

FIGURE 4 Load Distributions vs Time in Pile Nos. 4 and 6

the load cells showed compressive loads. Finally, during summer 1984, the recorded distribution showed that there was almost no load transfer through the shaft portion of pile above underream to the soil, indicating separation of pile shafts with the soil. This suggests that in calculating pile capacity under downward load condition, about 2 m of shaft should be ignored for contributing friction and the pile should be designed for downward loads for this condition.

During the second rainy season (1984) while the uplift loads developed in the pile No. 4 were almost equal to those observed during first rainy season (1983), no net tensile loads were recorded. This is because the point at which maximum uplift load occurred carried a greater compressive load at the start of the swelling of sub-soil this time.

In pile No. 6 during this season, the induced uplift loads work out to about 2/3 of the uolift loads which occurred during first rainy season (1983), reflecting the effect of removal of soil below and around the ledge projection of beam and removal of brick on edge from inner face of beam this time. It indicates that about 1/3 of the measured uplift load (about 13 kN) was induced by the beam-soil contact, suggesting that reinforcement on both faces of the beam is necessary. Also if the beams cast, remain unloaded during rainy season these may be subjected to tensile loads and movements both due to pile's movements and beam movements and thus should be designed for these forces. Alternatively if the piles have to remain unloaded during rainy season, the beam should not be laid over these.

The pattern of load distribution through pile No. 4 during third dry season (November to June 1985) was identical to that observed during second dry season in 1984. The top section of pile carried hardly any load and most of the load was carried by the underream. However, pile No. 6 showed different load disribution, indicatng more load transfer through toe during peak summer (May-June 1985) in comparison to the top section of pile and underream. Based on these observations it is difficult to arrive at any possible reason for this behaviour. There may be a possibility of shrinking of soil in deeper depths around this pile or some thing went wrong with the load cells.

Both the piles showed that these were subjected to net tensile loads during first cycle of rainy season, maximum being 43 kN for pile No. 6. Corresponding to this load, the tensile stress for a pile of 250 mm shaft diameter with 0.5% mild steel reinforcement (being used at present) works out to 0.83 N/mm^2 against permissible value of 2.0 N/mm^2 for the grade of concrete used in the piles in accordance with IS:456—1978. Thus the current practice of using about 0.5% reinforcement of cross sectional area of pile appears to be adequate.

Movement of Foundations

The recorded movements of foundation of the room with respect to time are shown in Fig. 5. The foundation showed uplift movement, maximum being between 6 and 12 mm recorded during first rainy season (1983). It indicates that the effect of uplift forces on account of swelling was more than the downward load as also reflected by the load cell observations. The maximum differential movement over wall span of 3 m works out to 5 mm, giving angular distortion 1 in 600. Also no structural damage was noticed. Thus the movements can be considered within safe limits. During the second season reduced movements were recorded. It woud appear that as the load transferred to the underream, less movements can be expected. No structural damage has been seen on the building to date after 60 months of construction. But some distress has been detected on the floor within the building. The floor, was laid directly on the prepared soil which was independent of pile foundations. These observations suggest that in most of the cases 3.5 m deep underreamed piles may prove a safe alternative even if the depth of active zone is more than 3.5 m.

Comparison of Observed and Predicted Behaviour

The procedure based on elastic analysis as reported by Poulos and Davis (1980) was used to predict uplift loads and movements imposed on the piles. For these calculations, piles were considered to be resting in the swelling zone and more weightage was given to the values of soil modulus and pile-soil interface strength upto 2 m depth in averaging out these values, The values of the parameters used were: depth of active zone Z', 4.5 m elastic moudulus E_s, 5000 kNlm^2 (constant with depth), Poisson's ratio, 0.4, pile-soil interface strength T_a, 40 kN/m^2 (constant with depth) and maximum heave of soil at surface y, 60 mm. Elastic modulus for pile concrete was taken 22076 N/mm^2.

The procedure for prediction involves, first the calculation of maximum uplift load and movement that is likely to occur if there was zero load on pile head. Subsequently, for working out net effects of this uplift load and downward load, the load distribution corresponding to downward load and calculated settlement corresponding to it, were superimposed.

Initially the uplift load PFS, that would occur if full adhesion was mobilised along the whole shaft, was calculated using following relationship;

$$PFS = \int_0^L T_a \, \pi \, D \, dz \qquad \ldots(1)$$

where L = Length of pile

D = Shaft diameter of pile

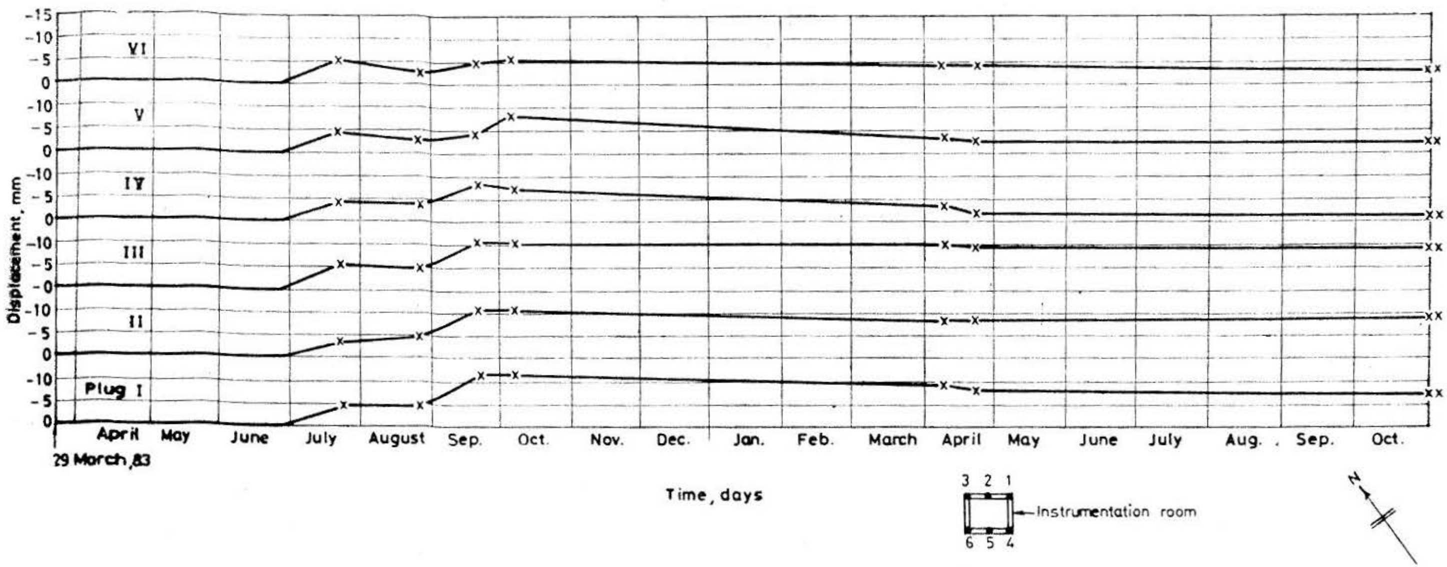

FIGURE 5 Displacement of Foundation vs Time

The ratio of P_{max}/PFS (P_{max} = maximum developed uplift load) was read corresponding to the value of $(y.Es)/(D.Ta)$ from the curves given for $Z'/L = 1$ and $Du/D = 2$ (Du = underreamed dameter) for constant pile-soil shear strength with depth. From the ratio P_{max}/PFS and calculated value of PFS, P_{max} was determined. Similarly, for the known value of y/D reading S_y/y (S_y = uplift movement of pile) from the curve corresponding to $Z'/L = 1$ and $Du/D = 2$ for constant pile-soil shear strength with depth, the value of S_y was worked out. Since the charts given by Poulos and Davis do not provide any curve for $Du/D = 2.5$, the values corresponding to $Du/D = 2$ were taken. These are expected to provide the estimate on conservative side.

The net uplift load considering the effect of downward axial load was determined by deducting the observed load at second load cell level (the level of maximum uplift load) just prior to rainy season (31 May 1983) from the calculated value of P_{max} as obtained above. For estimating net pile movement, the settlement S_d, corresponding to the load recorded by top load cell P_d, just prior to rainy season on the above date was first worked out using the following relationship,

$$S_d = \frac{P_d\, I}{E_s\, D} \qquad \ldots(2)$$

Where I = influence factor, obtained from various charts given by Poulos and Davis, its value was 0.097 for the present case.

The net pile movement was then determined by deducting this value from the calculated value of uplift movement, S_y as described above.

The predicted net values of uplift loads and movements are compared with the monitored values in Table 1.

TABLE 1

Monitored and Predicted Uplift Loads and Movements

Pile No.	Net Uplift Load (kN)		Net Uplift Movement (mm)	
	Predicted	Monitored	Predicted	Monitored
4	58	22	14	8
6	64	43	15	5

The predicted values are on higher side in comparison to the recorded values. The difference between the two may be attributed to various factors *e.g.* validity of assumptions in development of curves used, basis of their selection for calculations, election of various soil parmeters, superimposition of the effect of two loads etc., considered in predicting the values. However, to be on conservative side the method may be used till more data and a better method are available.

Conclusions

Based on the present investigation which is first of its kind in India providing direct measurement of load transfer through short bored underreamed piles supporting single storey structure in expansive soils alongwith the foundation movements, the following conclusions can be drawn:

(*i*) Underreamed piles with a beam at top designed and constructed in accordance with IS : 2911 (part—III)—1980, loaded prior to peak summer and rainy season by constructing a single storey structure over these, showed development of net tensile loads and uplift movements during first rainy season on account of swelling of sub-soil, suggesting that the piles in addition to the downward loads should also be designed for these loads. During second rainy season though the swelling affected load distributions, no net tensile loads were recorded.

(*ii*) The beam-soil contact in the form of embedment of ledge portion of beam and brick on edge on inner face of beam also contributes towards the development of tensile forces suggesting that the beam should be provided with reinforcement on both the faces.

(*iii*) After first cycle of seasons, the major portion of load transfer was found through underream and toe portion of piles, suggesting to ignore at least top 2 m of shaft for calculation of friction in place of 1.2 m suggested at present. Also the governing condition for design of piles under downward loads seems to be the summer *season when soil shrinks* maximum.

(*iv*) The maximum uplift loads measured suggest that the minimum reinforcement of about 0.5% of pile coss sectional area being used at present in underreamed pile appears to be sufficient.

(*v*) No structural damage has been noticed on room to date after 60 months of construction. Most of the movements (6 to 12 mm uplift) including differental movement, occurred during the first yearly cycle, the maximum being 5 mm over a wall span of 3 m, are within permissible limits. These observations suggest that in most of the cases 3.5 m deep underreamed piles may provide a safe foundation system even if the depth of swelling zone (active zone) is more than 3.5 m.